Agenda Item 11.
February 27, 2019
Prepared by Terry Lauritsen
Water Utilities

I. SUBJECT, ATTACHMENTS, AND BACKGROUND

Discuss and take action to approve the final report for a Water Reuse Feasibility Study completed through a Bureau of Reclamation grant to augment Bartlesville's Long Term Water Supply.

Attachments:

Title XVI Feasibility Study Report – Augment Bartlesville Water Supply with Drought-Resilient Reclaimed Water

II. STAFF COMMENTS AND ANALYSIS

Over the last 12 years, the City has been experiencing consistent flows into the wastewater treatment plant that have exceeded the plants rated design capacity. Due to the frequency of these flows, a facility plan was initiated in 2004 and completed in 2010 to investigate options to either expand the current treatment plant or build a secondary treatment plant south of town. Based on the City's growth patterns, comparable capital costs and long-term desire to move the treatment plant away from populated areas, Council selected the secondary treatment plant south of town option.

After the completion of this facility plan (2010), the state legislature adopted the Water for 2060 Law, which has a goal to consume no more fresh water in 2060 than consumed in 2012 and tasks the Oklahoma Department of Environment Quality to develop regulations and encourage water reuse. Bartlesville has a desirable layout regarding the location of the existing wastewater treatment plant and a potable raw water pump station on the Caney River, which is shown below.

Since water reuse was not considered in the 2010 facility plan, Council approved a contract with Tetra Tech in May 2016 to investigate the feasibility of reuse and update the costs for the two options developed through the 2010 facility plan. In August 2017, the results from the study were presented to Council, which concluded that water reuse is feasible up to 4 million gallons per day and the capital costs to build a secondary plant south of town was $65MM, while the cost to expand the existing treatment plant was $49.4MM. In September 2017, Council selected the option to expand the existing treatment plant and authorized staff to pursue water reuse through this option.

In January 2017, the City applied for a $150,000 grant through the Bureau of Reclamation's (BOR) WaterSMART program to pursue a feasibility study for water reuse. In October 2017 the City was awarded the grant. While some of the tasks necessary for the feasibility study were completed through Tetra Tech's May 2016 contract, Council approved an amendment to the 2016 contract in October 2017 for additional analysis and sampling necessary for regulatory approval to complete this feasibility study.

The final report for the feasibility study is complete, which is attached, and will be presented to Council at its March 4th meeting.

III. RECOMMENDED ACTION

Staff recommends approval of the final report for the water reuse feasibility study.

Title XVI Feasibility Study Report

Augment Bartlesville Water Supply with Drought-Resilient Reclaimed Water

PRESENTED TO

Bureau of Reclamation
Oklahoma-Texas Area Office
5316 Highway 290 West, Suite 110
Austin, TX 78735

PRESENTED BY

City of Bartlesville
401 S. Johnstone Avenue
Bartlesville, OK 74003

JANUARY 2019

PREPARED BY

TETRA TECH

S2 ENGINEERING PLLC

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Reviewed By: (Prime Consultant)

LICENSED PROFESSIONAL ENGINEER
JOSHUA CHRISTOPHER PAZ
29103
OKLAHOMA

2/20/19

Project Manager
Prime Consultant
Tetra Tech, Inc.
7645 E 63rd Street, Suite 301
Tulsa, OK 74133
OK CA 2388, Exp 06/19

Sub Consultant:

REGISTERED PROFESSIONAL ENGINEER
SRINIVASAN SUNDARAMOORTHY
16260
OKLAHOMA

2/20/19

S2 Engineering PLLC
7633 E 63rd Place, Suite 300
Tulsa, OK 74133
OK CA 5077, Exp. 06/19

MS/ABBREVIATIONS

AFY	Acre-Feet per Year
BMA	Bartlesville Municipal Authority
CRWPS	Caney River Raw Water Pump Station
CWWTP	Chickasaw Wastewater Treatment Plant (City of Bartlesville)
DEQ	Department of Environmental Quality (Oklahoma)
DPR	Direct Potable Reuse
EA	Environmental Assessment
FOA	Funding Opportunity Announcement
FS	Feasibility Study
FY	Fiscal Year
IPR	Indirect Potable Reuse
MG, MGD	Million Gallons, Million Gallons per Day
MIB	2-Methylisoborneol
O&M	Operation & Maintenance
OCWP	Oklahoma Comprehensive Water Plan
ODEQ	Oklahoma Department of Environmental Quality
OPDES	Oklahoma Pollutant Discharge Elimination System (Permit)
OWQS	Oklahoma Water Quality Standards
OWRB	Oklahoma Water Resources Board
PAS	Planning Assistance to States
QA/QC	Quality Assurance / Quality Control
RWD	Rural Water District
T&O	Taste and Odor
TMDL	Total maximum Daily Load
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USEPA	U.S. Environmental Protection Agency
USFW	U.S. Fish and Wildlife Service
WLA	Waste Load Allocation
WQ	Water Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

1.0 INTRODUCTION

1.1 NON-FEDERAL SPONSOR

The City of Bartlesville is the non-federal project sponsor. The City of Bartlesville is an incorporated municipality in Oklahoma with a 2015 census population of approximately 36,596. Bartlesville operates and maintains its own water and wastewater utilities. In addition to serving within its city limits, the Bartlesville water system serves the surrounding communities of Washington County Rural Water District (RWD) #2, Washington County RWD #5, Osage County RWD #1, Town of Ochelata, Town of Ramona, City of Dewey, Strike Axe Water system, and the Bar Dew water system.

The proposed project is a critical component of the overall long-term plan to address Bartlesville's water supply needs. Bartlesville has the backing and support of its elected officials, local community leaders, and the stakeholders to pursue this study. Funding has already been appropriated by the Bartlesville City Council to pursue this study.

1.2 DESCRIPTION OF STUDY AREA

The Bartlesville service area is approximately 282 square miles covering part of Washington County, Osage County, and Nowata County, and serves as the major regional water supplier within the watershed basin.

The City of Bartlesville is an incorporated municipality in Oklahoma with a 2015 census population of approximately 36,596. Bartlesville operates and maintains its own water and wastewater utilities and its water service area is shown in Figure 1-1. In addition to serving within its city limits, the Bartlesville water system serves the surrounding communities of Washington County Rural Water District (RWD) #2, Washington County RWD #5, Osage County RWD #1, Town of Ochelata, Town of Ramona, City of Dewey, Strike Axe Water system, and the Bar Dew water system.

1.3 EXISTING CONDITION

Bartlesville's primary source of raw water supply is Hulah Lake (see Figure 1-2**)**. Hulah Lake is a federally owned lake originally completed in 1951 for flood control, water supply, low flow regulation, and conservation purposes. Bartlesville has a water right of 13,819 acre-feet (12.4 MGD). Raw water from Hulah Lake is pumped to discharge into Lake Hudson which is a city-owned lake. Due to its size, Lake Hudson is insufficient for water supply yield on its own and is considered part of the Hulah/Hudson water supply system.

Bartlesville also has water rights on the Caney River, which served as the original raw water supply for Bartlesville prior to the development of the Hulah/Hudson lake system. In the late 1920s a low water dam was constructed on the Caney River to create a small impoundment within the river from which to draw the raw water. Bartlesville continues to operate a 1940-era raw water pump station on the Caney

River within this impoundment and uses the Caney River as a secondary source. Due to intermittent water quality and seasonal flow variations, this water source is unreliable.

The current raw water supply portfolio available to Bartlesville is as summarized below:

- Surface Water Sources:
  - Hulah Lake. Bartlesville has 13,819 acre-feet (12.4 MGD) of water rights. There are no more water rights available at this Federally owned lake. Based on historic and projected silting and sediment deposits, the projected dependable yield from Hulah is 6.4 MGD through year 2035 and 4.4 MGD by year 2055.
  - Hudson Lake. Bartlesville has 6,000 acre-feet (5.4 MGD) of water rights which represent all the water rights available at this City-owned lake. Due to the size of the lake and limited watershed, there is no appreciable yield associated with the lake, and it is considered part of the Hulah Lake water supply system. Therefore, for practical reasons, water rights from Hudson Lake are not considered separate but included within available water rights from Hulah Lake.
  - Caney River. Bartlesville has 6,000 acre-feet (5.4 MGD) of water rights and operates a 1940-era pump station on the Caney River. The intermittent water quality and seasonal flow variation in the river makes this source non-dependable.
- Ground Water Sources: There are no known dependable ground water supplies within the watershed with adequate quantity or quality for potable use.
- Reclaimed Water Sources: Reclamation of wastewater effluent from Bartlesville's wastewater treatment plant is a potential option to include in the water supply portfolio which is the focus of this feasibility study.

1.4 FEASIBILITY STUDY

This study evaluates the feasibility of utilizing the reclaimed effluent from Bartlesville's existing wastewater treatment plant to augment Bartlesville's water supply by discharging the reclaimed effluent approximately 7 miles upstream of the existing Caney River raw water intake. Raw water from the Caney River intake will be pumped to Bartlesville's existing water treatment plant for treatment to comply with the Safe Drinking Water Act and ODEQ standards and regulations to meet or exceed potable water quality standards.

The proposed reclaimed water augmentation will benefit all service areas of the Bartlesville water supply system.

Figure 1-1 Bartlesville Water Service Area

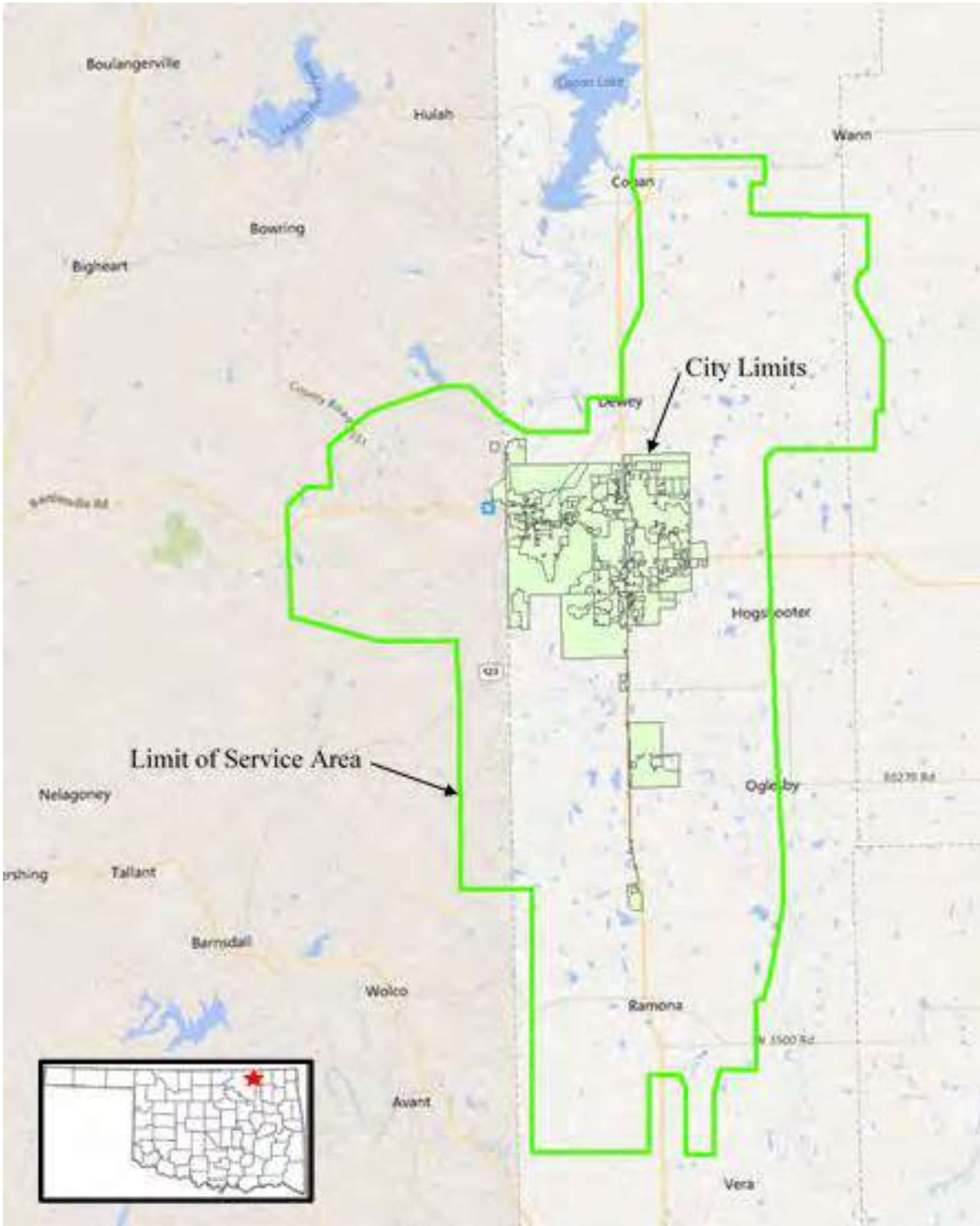
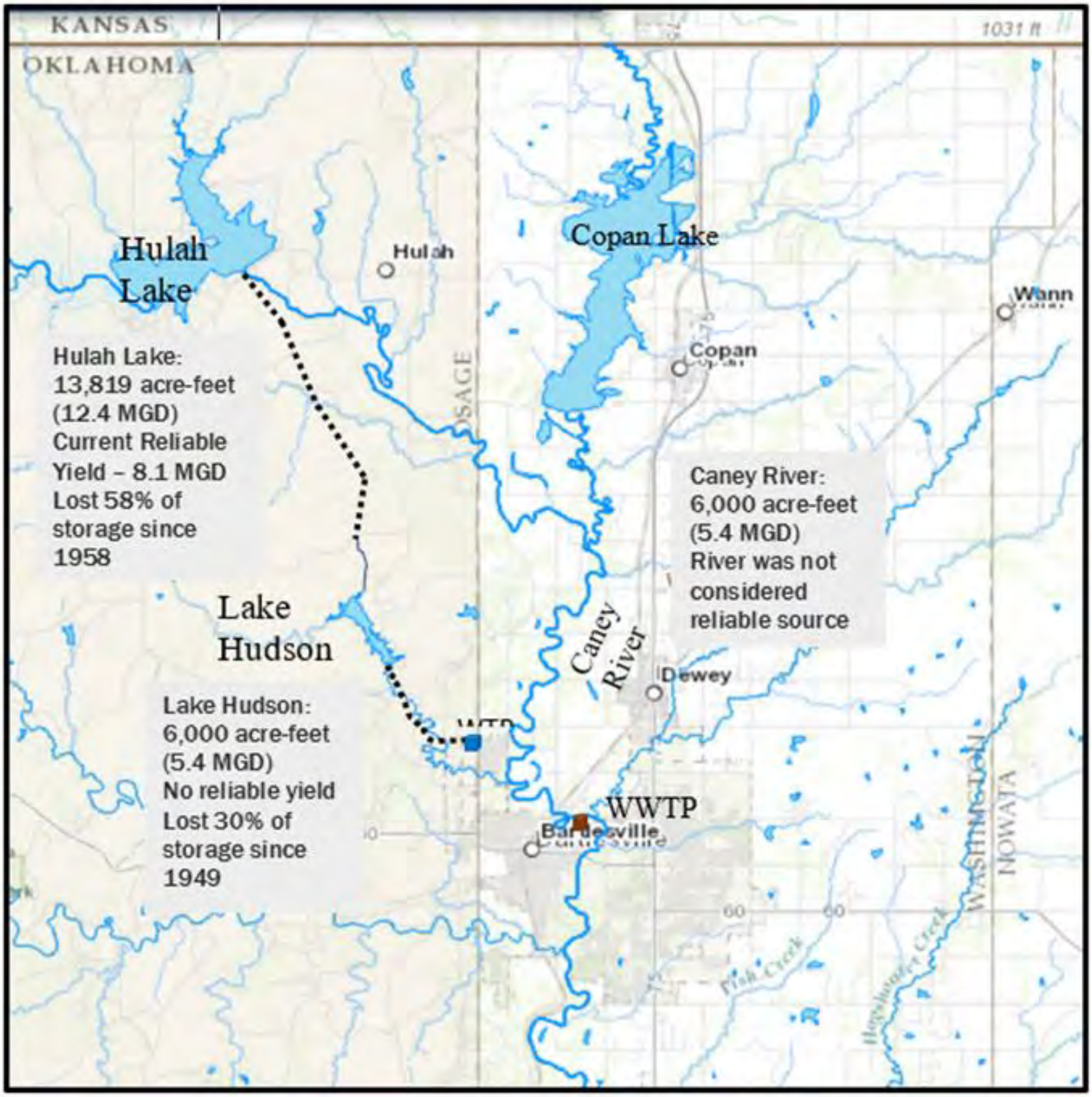


Figure 1-2 Bartlesville's Existing Water Supply Sources



2.0 STATEMENT OF PROBLEMS AND NEEDS

2.1 STATEMENT OF PROBLEM

Bartlesville's primary source of raw water supply is Hulah Lake. Hulah Lake is a federally owned lake originally completed in 1951 for flood control, water supply, low flow regulation, and conservation purposes. Bartlesville has a water right of 13,819 acre-feet (12.4 MGD). Raw water from Hulah Lake is pumped to discharge into Lake Hudson which is a city-owned lake. Due to its size, Lake Hudson is insufficient for water supply yield on its own and is considered part of the Hulah/Hudson water supply system.

Bartlesville also has water rights on the Caney River, which served as the original raw water supply for Bartlesville prior to the development of the Hulah/Hudson lake system. In the late 1920s a low water dam was constructed on the Caney River to create a small impoundment within the river from which to draw the raw water. Due to intermittent water quality and seasonal flow variations, this water source is unreliable. Water is pumped from the river to the Ted D. Lockin Water Treatment Plant. Bartlesville uses the Caney River as a secondary source.

The severe drought of 2001-2002 was a wake-up call regarding the long-term viability and dependability of Bartlesville's primary source of water - Hulah Lake. This drought resulted in Hulah Lake storage being reduced by approximately 68%. The drought prompted several actions to be taken to secure more reliable sources of water in the future. Bartlesville adopted the 2002 Drought Contingency Plan and began seeking new alternative sources of water to secure long-term water supply portfolios.

In late 2002, the City Council created the Water Resource Committee - a 15-member committee ranging from City Council members, City staff, business, and service leaders as well as federal legislative liaisons. The committee was tasked with identifying a long-term water supply source.

In 2002, a Volumetric Survey of Hulah Lake was completed by the Texas Water Development Board for the United States Corps of Engineers (USCOE), Tulsa District. The summary comparison from this survey showed that the active pool storage capacity was 22,553 acre-feet in 2002 compared to 33,390 acre-feet in 1958, an approximately 32.5% reduction due to sedimentation and silting.

In 2004 the USCOE, Tulsa District, completed a study that evaluated the cost of bringing water from other surface water sources within Oklahoma including federal lakes, state lakes, and Natural Resources Conservation Service (NRCS) lakes. The options required construction in excess of 30 miles of pump and pipeline infrastructure and required securing new water rights from already stressed resources.

During 2006-2007, Bartlesville partnered with the USCOE, Tulsa District, and the Oklahoma Water Resources Board (OWRB) and completed the water supply alternatives study under the Planning Assistance to States (PAS) program.

- Phase I of the study evaluated the current and projected water demand through 2055 and re-evaluated the dependable yield from Hulah reservoir based on historic and projected silting and sediment deposits in the lake. The Phase I evaluation concluded that Bartlesville's dependable yield from Hulah will decrease from the original yield of 12.4 MGD in 1951 to 6.4 MGD through year 2035 and to 4.4 MGD by year 2055.
- Phase II of the study focused on three primary alternatives: (1) purchasing remaining water storage rights at Copan Lake and purchasing additional storage rights through the reallocation of flood storage to water supply at Copan Lake and Hulah Reservoir; (2) development of a new reservoir (called Sand Lake) on Sand Creek in Osage County, Oklahoma; (3) use of Kaw Lake water supply storage, and development of a pipeline to the city's Hudson Lake.
    - The Phase II study concluded that alternate (1) was the most viable option to satisfy the City's raw water needs through year 2055. The study recommended the City purchase new water supply agreements through the USCOE for Hulah and Copan Lake as follows:
        - Three new agreements at Copan Lake consisting of 1) remaining water supply originally authorized with the lake's construction, 2) new storage reallocated from water quality, and 3) new storage reallocated from reallocating 5 percent of the flood control and raising lake water surface by 1.99 feet.
        - Two new agreements at Hulah Lake consisting of 1) new storage reallocated from water quality and 2) new storage reallocated from reallocating 5 percent of the flood control and raising the lake surface by 3.67 feet.
        - However, the flood control storage reallocation for Copan Lake and Hulah lake requires regulatory clearance and approval, downstream flood damage mitigation through the purchase of property or easements, and mitigation to upstream recreational and cultural resources.
- In 2013 Bartlesville initiated the Copan Raw Water Conveyance Study. The goal of this study was to establish a plan for the raw water conveyance facilities to convey raw water from Copan Lake to Bartlesville's existing Ted D. Lockin Water Treatment Plant (WTP).

There are various uncontrollable factors that threaten the long-term viability of the City's existing water sources, as well as the ones identified through the PAS study. The historic silting and sediments flow into the lake will continue to decrease the dependable yield from Hulah reservoir as well as Copan Lake. Hulah is a state designated nutrient limited watershed that will have unspecified long-term impact on water quality. Both surface water resources are dependent on rainfall and runoff that are prone to seasonal variations, severe drought, and long-term climate change.

Availability of other regional surface water sources are limited and will require a substantial amount of new infrastructure improvement in pipeline, intake, and pump station costs; require securing of new water rights; and involve considerable regulatory approval. These new sources will also be subject to a similar level of uncertainties associated with drought, regional and global climate change, and other environmental factors.

There are no known dependable ground water supplies within the watershed with adequate quantity or quality for potable use.

Bartlesville is a regional water supplier, and the water demand for the region is expected to grow. Based on the 2006-13 studies, the projected average water demand for the Bartlesville service area is projected to grow to 9 MGD under an average growth scenario and 10.8 MGD under an optimistic growth scenario by the year 2065 as discussed later in the report. The current supply portfolio will experience a supply gap in the next 10 to 15 years, and this is even without consideration of impact from climate change and/or impact from drought.

2.2 PROJECT NEED

Bartlesville is in Basin 76 of the Middle Arkansas Watershed Planning region as published by the 2012 Oklahoma Comprehensive Water Plan (2012 OCWP), see Figure 2-1. This watershed region primarily relies on surface water supplies and there are no dependable ground water sources available for Bartlesville. The 2012 OCWP identified a water supply gap in this basin by 2020 and beyond, even without considering the potential impacts from global warming and climate change. Bartlesville is the major water supplier in this watershed.

Figure 2-1 Middle Arkansas Watershed

Historically, water system master planning in Oklahoma looked at the City's wastewater system as a separate and stand-alone system to the City's water supply portfolios; the use of reclaimed water was in its infancy in Oklahoma. However, Oklahoma's "Water for 2060" goal under the 2012 Oklahoma Comprehensive Water Plan (OCWP) changed the paradigm. The goal of this law is to consume no more fresh water in the year 2060 than is consumed statewide in the year 2012 while continuing to grow the populations and economy. Reclamation of a portion of the effluent from the city-owned wastewater treatment to augment the Caney River supplies is an attractive option to expand the current water supply portfolio for Bartlesville, fill the water supply gap within the basin, address the regional watershed needs, extend the existing supply use, and enhance the drought resiliency.

Bartlesville strongly believes that reclaimed water from the existing wastewater treatment plant is an integral part of the future water supply portfolio and believes that a strategic use of reclaimed water will extend the city's water supply by as much as 25 years based on current projections. For example, augmenting with approximately 2 MGD of reclaimed water will extend the water supply by 13 years, and a 4 MGD augmentation will extend it by approximately 25 years.

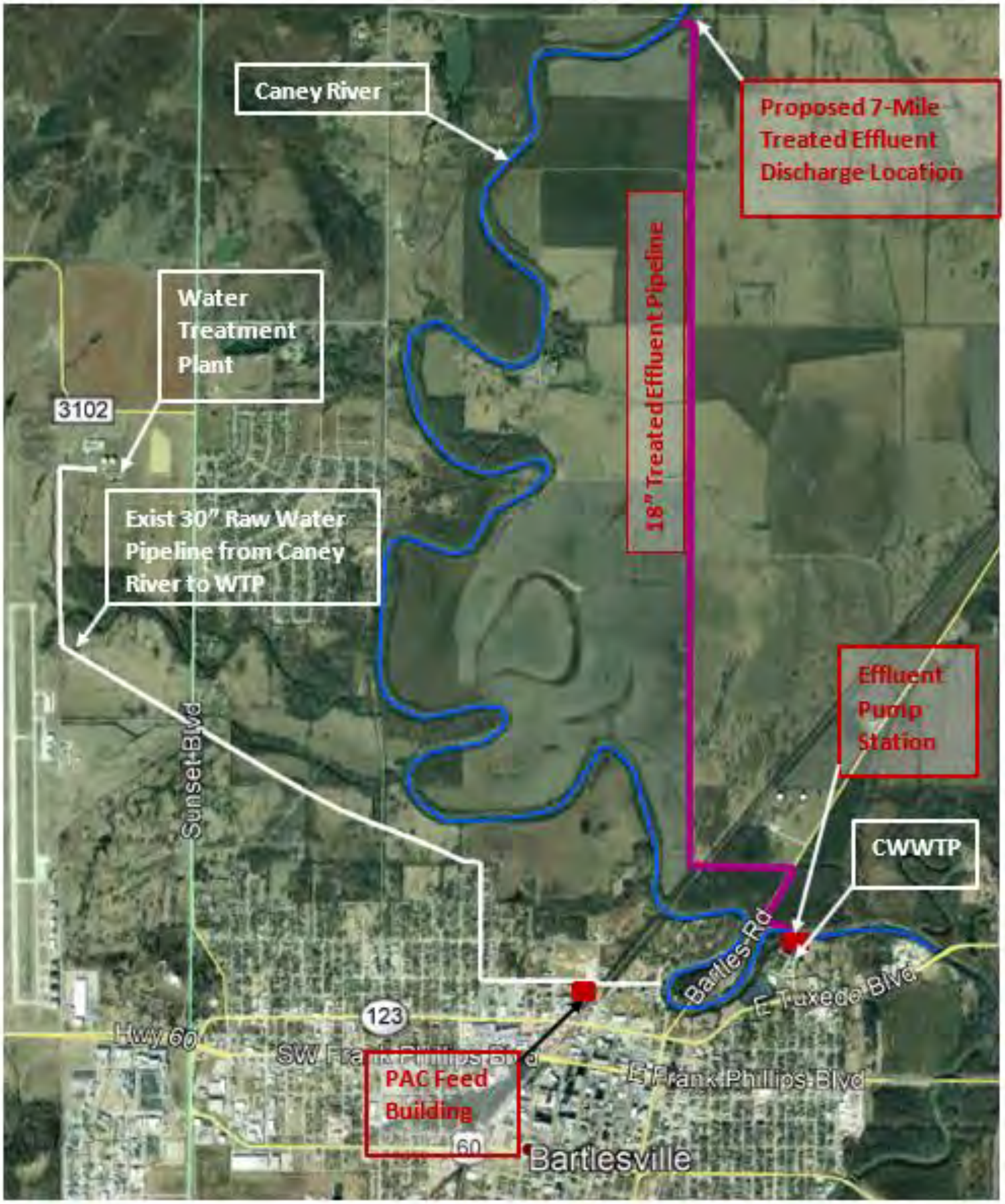
The existing location of the wastewater treatment plant with respect to the existing Caney River raw water intake provides a unique opportunity for Bartlesville to utilize the reclaimed water in an innovative way. Reclaimed water is readily available at the city-owned Chickasaw Wastewater Treatment Plant (CWWTP), which has a permitted capacity of 7 MGD. As the CWWTP is expanded to meet future growth, additional reclaimed water will be available as well. The proposed project will evaluate the following alternatives individually or in combination thereof:

1. Reclaim effluent from the CWWTP, treat it to a level suitable to maintain and/or improve Caney River water quality standards, and discharge approximately 5 to 7 miles upstream of the existing Caney River raw water intake to augment the flow in the Caney River. Reclaimed effluent will provide a drought-resilient supply to Caney River flow upstream of the raw water intake.

2. Identify and develop specific non-potable use for the reclaimed water within the CWWTP to offset the potable use currently practiced at CWWTP.

The goal and the focus of this feasibility study is to (1) demonstrate and document the technical feasibility, (2) evaluate the impacts to environmental and cultural resources, (3) develop present worth life-cycle costs for funding, designing and implementing of the preferred alternatives, and address other Title XVI Water Reclamation and Reuse Program goals and objectives. Figure 2-2 shows the proposed project concept.

Figure 2-2 Proposed Title XVI Project



2.3 CURRENT AND PROJECTED WATER DEMAND AND SUPPLIES

2.3.1 Projected Water Demand

Under the Planning Assistance State (PAS) Program, in 2007 the U.S. Army Corps of Engineers (USCOE), Tulsa District, completed a water study titled, Bartlesville Water Supply and Conveyance Study (PAS Study). The PAS Study was completed in two phases. Phase I evaluated the current and projected water demands in relationship to the existing water supply for the study period 2005 to 2055. Phase II evaluated three alternatives to provide additional water supply for Bartlesville and Washington County.

The projections provided in the PAS Study were reviewed to confirm the water demand projections for the City of Bartlesville and its satellite customers for use in this feasibility study.

The PAS Study determined future net water needs for the City of Bartlesville (net meaning the water needs not including the usage at the WTP) and the surrounding communities, rural water systems, and other areas to which the city provides water. The study used actual usage data for 2005 as the base year and estimated the future demand based on different growth scenarios Washington County may experience over a 50-year planning period. Since the City of Bartlesville supplies water to approximately 99% of the residents in Washington County, the study forecast was based on Washington County data. This study used existing population, housing, and employment data to project future population and water supply needs.

Three water demand scenarios were presented in the report. The “Baseline Projection” scenario for population growth was based on historical growth and weather pattern trends experienced in the study area. The “Baseline Projection” population of Washington County is 53,000 by the year 2055. The “High Projection” scenario utilized current growth trends in Bartlesville and resulted in a higher growth population estimate of 73,169 by the year 2055. The third scenario, called the “Mid Projection,” was the average of the “Baseline Projection” and “High Projection” growth scenarios. The “Mid Projection” population estimate for Washington County in 2055 was 63,000.

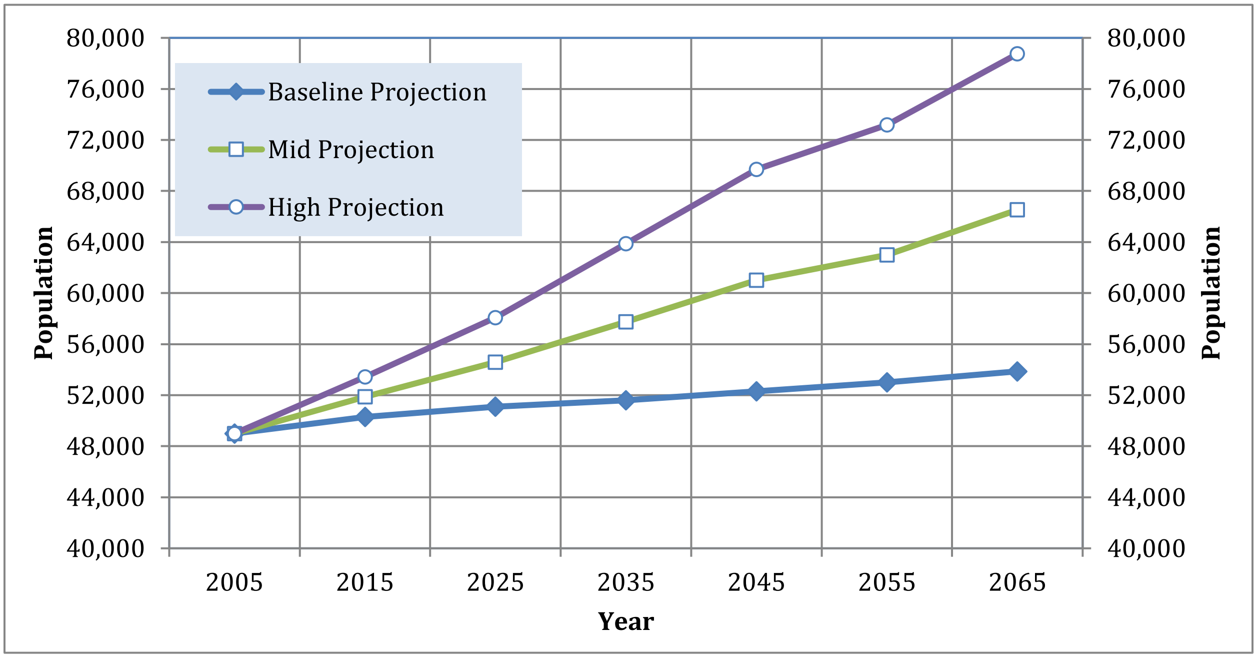
Table 2-1 is a summary of the population projections from the PAS Study. The PAS Study used a 50-year study period from 2005 to 2055. The data shown for 2065 was extrapolated based on linear trend-line projections. Figure 2-3 graphically shows the population projections for Washington County.

Table 2-1 Population Projection for Washington County

Population Projections for Washington County							
Scenario	2005	2015	2025	2035	2045	2055	2065[1]
Baseline	48,996	50,300	51,100	51,600	52,300	53,000	53,868
Mid Growth	48,996	51,870	54,590	57,740	61,000	63,000	66,525
High Growth	48,996	53,436	58,065	63,877	69,685	73,169	78,747

[1] Projection for 2065 based on Trend-line

Figure 2-3 Population Projection for Washington County



Based on the PAS Study, the City of Bartlesville agreed to utilize the Mid Projection and the High Projection growth scenarios in planning for long-term water supply needs. The Baseline Projection was an underestimate. This assertion was supported by the 2010, US Census which became available after the PAS Study was completed. The census results indicated that the 2010 population of Washington County was 50,976, which is more than what the Baseline Projection had estimated, but less than the Mid Projection.

The 2007 PAS Study utilized the population projections as the key parameter in projecting the residential water demand for Washington County. The demands for commercial, industrial, and public use categories were made using the current and projected employment data.

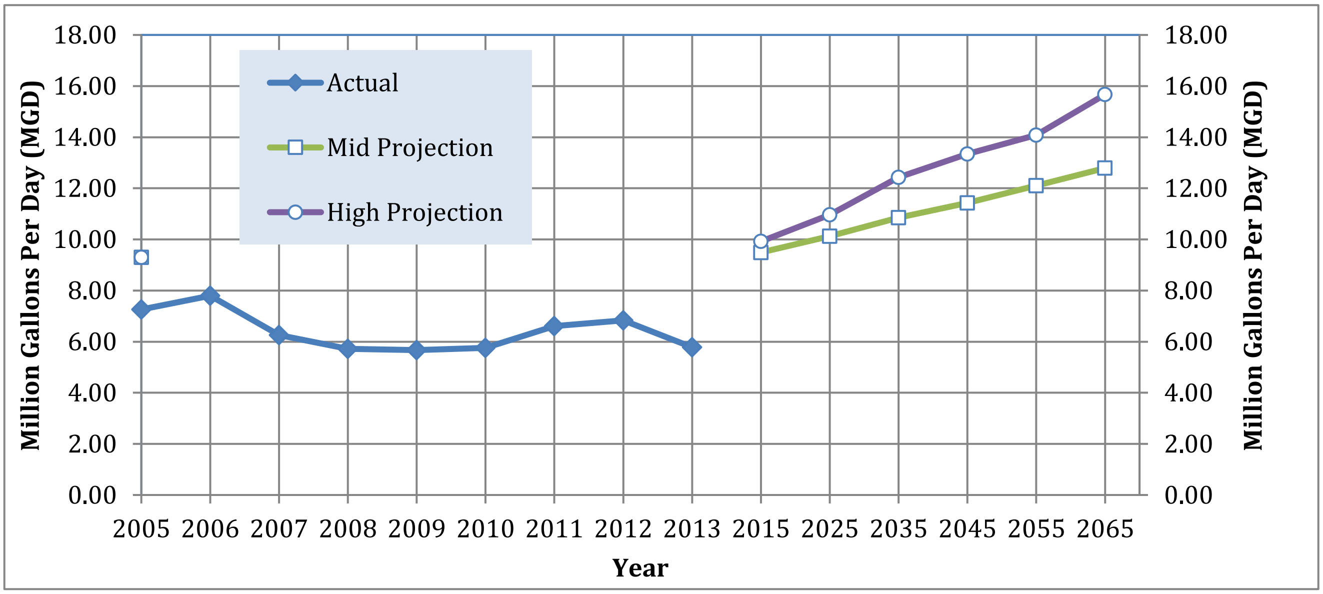
Table 2-2 summarizes the total average day water demands for the Baseline Projection and High Projection from the 2007 PAS Study. The 2065 projections were developed for this study using a linear trend-line extrapolation. The Mid Projections are the average of the other two projections. Figure 2-4 graphically shows the projected average water demand along with the actual total raw water usage for the period 2005 through 2013. As shown on the graph, the projected demands are conservative as compared to the 2005-2013 actual usage. The 2007 PAS Study used 2005 as the base year and a 2005 usage of 9.3 MGD. The actual 2005 usage was 7.26 MGD for Bartlesville and its other customers. Also, the PAS Study included amounts of unmetered/unaccounted water varying from 1.2 MGD to 1.9 MGD for the scenarios.

Table 2-2 2007 PAS Water Demand Projections

2007 PAS Study Demand Projections[3]							
Scenario	2005	2015	2025	2035	2045	2055	2065
Baseline Projection[1]	9.30	9.80	9.90	10.10	10.20	10.50	10.71
Mid Projection[1,2]	9.30	10.30	10.90	11.60	12.15	12.80	13.45
High Projection[1]	9.30	10.70	11.70	13.10	14.10	14.80	16.19

Notes:
[1] In 2007 Bartlesville decided to use the Mid Projection and the High Projection scenarios for long-term planning
[2] Mid Projections are an average of the Baseline and High Projections
[3] 2065 projections were made by Tetra Tech using the linear trend-line extrapolation

Figure 2-4 Bartlesville Service Area Water Demand Projection (2007 PAS Study)



For this study, the following adjustments are proposed to the projections made in the PAS Study. First, the actual historical usage for 2015 was 5.7 MGD compared to 10.30 MGD projected in the PAS study. An adjustment factor of 5.12 MGD was applied to projection. Secondly, the demand projections represent what the treatment plant must deliver (in treated water) to the distribution system. To account for the treated water uses within the treatment plant, the historical raw water to treated water ratio (5%) will be used to estimate the total raw water supply needs. An average factor of 1.05 is used to convert the projected treated water demands to the raw water supply need. These adjustments are reflected in Table 2-3. Therefore, for this study a projected demand of 9.00 MGD (Average Demand Projection) to 10.80 MGD (Optimistic Demand Projection) is proposed for planning for the long-term (2065) water supply needs.

Table 2-3 Bartlesville Service Area Adjusted Water Demand Projections

Adjusted Water Demand Projections						
Scenario	2015[1]	2025	2035	2045	2055	2065
Mid Projection	10.30	10.90	11.60	12.15	12.80	13.45
Adjustment to PAS Study Baseline	-5.12	-5.12	-5.12	-5.12	-5.12	-5.12
RW/FW Ratio Adjustment, 5%	0.52	0.55	0.58	0.61	0.64	0.67
Adjusted Mid Growth Projection	**5.70**	**6.33**	**7.06**	**7.64**	**8.32**	**9.00**
High Projection[1]		**7.60**	**8.47**	**9.17**	**9.98**	**10.80**
Notes:[1]2015 is historical data. [2]High Projection is approximately 120% of Mid Projection						

2.3.2 Water Supply

The current raw water supply portfolio available to Bartlesville is as summarized below:

- Surface Water Sources:
    - Hulah Lake. Bartlesville has 13,819 acre-feet (12.4 MGD) of water rights. There are no more water rights available at this Federally owned lake. Based on historic and projected silting and sediment deposits, the projected dependable yield from Hulah is 6.4 MGD through year 2035 and 4.4 MGD by year 2055.
    - Hudson Lake. Bartlesville has 6,000 acre-feet (5.4 MGD) of water rights which represent all the water rights available at this City-owned lake. Due to the size of the lake and limited watershed, there is no appreciable yield associated with the lake, and it is considered part of the Hulah Lake water supply system. Therefore, for practical reasons, water rights from Hudson Lake are not considered separate but included within available water rights from Hulah Lake.
    - Caney River. Bartlesville has 6,000 acre-feet (5.4 MGD) of water rights and operates a 1940-era pump station on the Caney River. The intermittent water quality and seasonal flow variation in the river makes this source non-dependable.
- Ground Water Sources: There are no known dependable ground water supplies within the watershed with adequate quantity or quality for potable use.
- Refer to Figure 1-2 for the existing water supply sources.

Figure 2-5 shows the projected water demand with existing water supply portfolio. As shown, the projected water demand will exceed the current water supply portfolio capacity between 2025 and 2030 depending on either the average or optimistic demand projections.

Figure 2-5 Water Demand and Supply (Existing Portfolio)

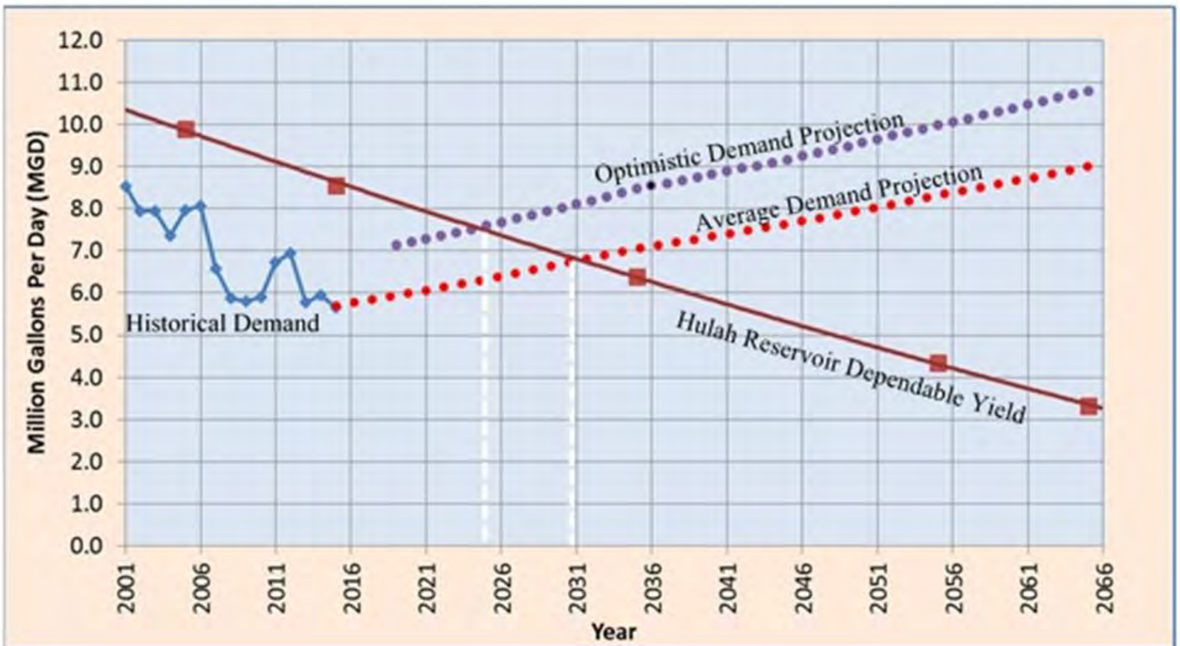
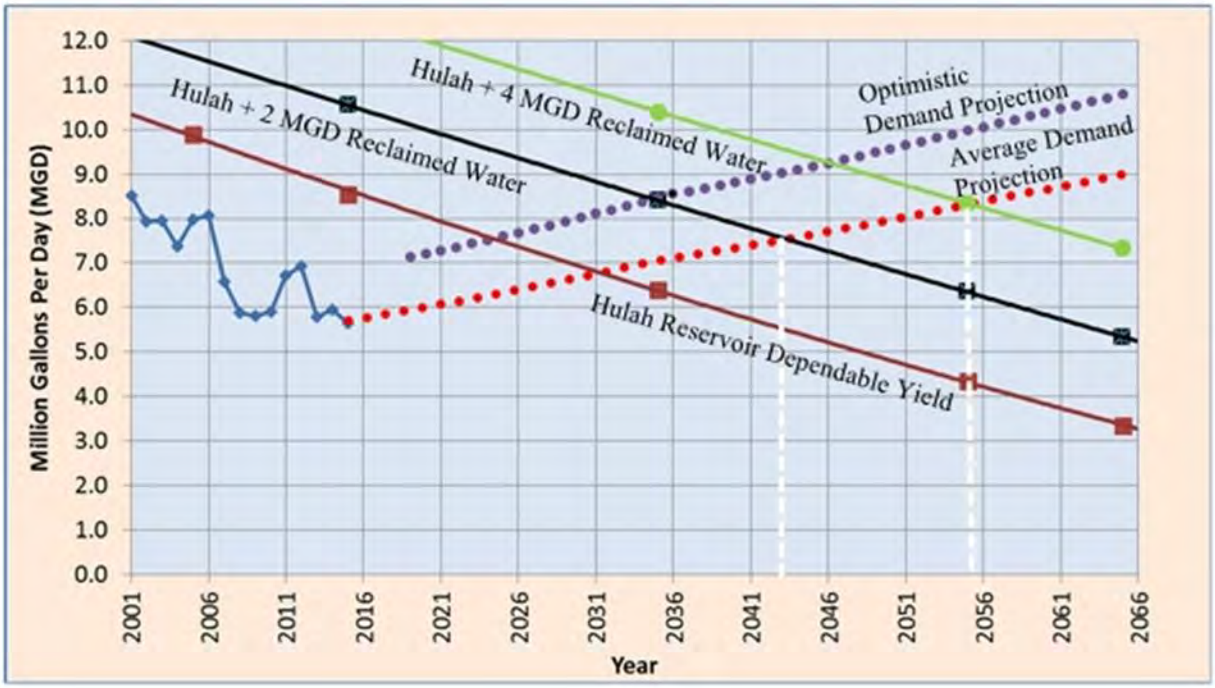


Figure 2-6 shows the water demand and supply with the proposed project where up to 4 MGD of reclaimed water will be used to augment the existing water supply. Bartlesville should be able to meet the projected demand through 2043 to 2055 depending on either the average or optimistic demand projections.

In addition, the proposed project will be drought-resilient since the wastewater flow is not materially impacted by drought or climate change impacts. The proposed project provides lead time (approximately 25 years) for Bartlesville to continue working towards other alternatives recommended in the PAS study, namely, acquiring additional water rights from Hulah and Copan Lakes, and planning and constructing the pipeline and pump station infrastructure necessary to convey the new sources to the Bartlesville service area.

Figure 2-6 Water Demand and Supply (Proposed Project)



2.4 WATER QUALITY CONCERNS

Currently, treated effluent from the existing wastewater treatment plant is discharged to the Caney River approximately ¼ mile downstream of the Caney River Raw Water Intake that Bartlesville maintains. The proposed project will reclaim a portion of the treated effluent from the wastewater treatment plant and will transport it to a location on the Caney River approximately 5 to 7 river miles upstream of the Caney River Raw Water Intake. The reclaimed water will co-mingle with the Caney River flow and travel via the environmental buffer before reaching the raw water intake from where the augmented water supply will be pumped to the Bartlesville Ted Lockin Water Treatment Plant where it will be completely treated to potable water quality to comply with the federal Safe Drinking Water Act requirements.

Effluent from the wastewater treatment plant is subject to the Oklahoma Pollution Discharge Elimination System (OPDES) permit and ODEQ rules and regulations and will comply with the federal Clean Water Act.

In Oklahoma, regulations governing the Indirect Potable Reuse (IPR) are addressed in DEQ regulations, Title 252:628 - Indirect Potable Reuse for Surface Water Augmentation. This rule addresses new discharges of treated municipal wastewater to existing Public Water Supply (PWS) surface waterbodies for the purpose of augmenting the existing volume of water available for PWS purposes. These rules apply to discharges to both sensitive water supplies and other reservoirs designated with the Public and Private Water Supply beneficial use in the Oklahoma Water Quality Standards (OWQS), or upstream of such reservoirs.

However, these rules do not apply to discharge of treated municipal effluent to existing streams and rivers, which is the case for the proposed project. For the proposed project, DEQ has determined that based on the proposed 5- to 7-mile discharge location and other factors, the proposed discharge of reclaimed water is not considered IPR. DEQ will consider the proposed project as a point source discharge to Caney River, and the discharge will be subject to OPDES permit requirements. As part of this feasibility study, Bartlesville has completed a waste load allocation (WLA) study and submitted to DEQ for review and approval. The findings of the WLA are that the advanced secondary level of treatment achieved at the wastewater treatment plant will meet the wasteload allocation set for Caney River, and confirmed the assimilative capacity of Caney River for this discharge.

Bartlesville has partnered with Oklahoma Geological Survey/University of Oklahoma to conduct a Constituents of Emerging Concerns (CEC) study along Caney River. The goal of the study is to benchmark the CEC within the Caney River segment as well as the Hudson/Hulah water supply to set a comparison for public information and for future monitoring and control. Samples are collected at six different locations quarterly over a one-year period. This study is scheduled to be completed by early 2019.

Existing Caney River supply experiences periodic taste and odor (T&O) episodes due to summer algae bloom in the river. As part of the proposed project, a powder activated carbon (PAC) feed system will be included to the Caney River Intake to remove T&O compounds.

In summary, the proposed project will meet the Oklahoma Water Quality Standards (OWQS), and the proposed powder activated carbon feed system at the raw water intake will address the seasonal and sporadic taste and odor issues from the Caney River water source.

3.0 WATER RECLAMATION AND REUSE OPPORTUNITIES

3.1 USE FOR RECLAIMED WATER

The existing Chickasaw Wastewater Treatment Plant has an average permitted capacity of 7.0 MGD. The objective of the proposed project under this feasibility study is to reclaim up to 4 MGD of the treated effluent and use it to augment the Caney River water supply. As discussed earlier, the reclaimed water will meet all DEQ rules and treatment requirements and will comply with the Oklahoma Water Quality Standards (OWQS).

In addition, there is a moderate use for the reclaimed water within the existing wastewater treatment plant for non-potable use such as spray water for the headwork screens, gravity belt thickener, chemical mixing and dilution, and plant-wide wash water purposes. The demand for reclaimed water within the existing wastewater treatment plant is estimated to total approximately 0.072 MGD. The effluent quality will meet the DEQ regulation (Category 6 Reuse Water) for use within the plant. No additional treatment will be required. The City of Bartlesville plans to implement reclaimed water reuse within the existing wastewater treatment plant as part of a future plant upgrade.

In summary, up to approximately 4.0 MGD of reclaimed water will be used to augment the Caney River water supply to serve the long-term water supply needs for Bartlesville. The level of treatment already achieved at the existing Chickasaw Wastewater Treatment Plant is sufficient to meet the water quality standards stipulated by the Oklahoma Department of Environmental Quality (ODEQ). The Caney River is subject to seasonal and sporadic taste and odor (T&O) episodes due to summer algae growth, which will be addressed by a new powder activated carbon (PAC) feed system at the intake.

3.2 MARKET AVAILABILITY TO UTILIZE RECLAIMED WATER

Traditionally, other uses applicable to reclaimed water include: non-contact landscape/golf course irrigation (purple pipe distribution system), cooling water, and other industrial uses. Such use will also require additional treatment specific to the end user need. Currently, Bartlesville has not identified any significant opportunity for such uses.

3.3 CONSIDERATIONS THAT MAY PREVENT IMPLEMENTATION OF THE REUSE PROJECT

The proposed project is consistent with the federal and Oklahoma water resources initiatives, federal and state regulations, and the Oklahoma Comprehensive Water Plan. Use of reclaimed water will reduce the competition and reliance on new freshwater sources and will help to fill the supply gap within the Bartlesville water service area.

With any project of this magnitude, there are limiting factors that could negatively impact and potentially prevent or delay the project implementation. The following factors could impact the implementation of the project:

- The proposed secondary discharge to Caney River is subject to OPDES permit requirements under the jurisdiction of DEQ. Any delay in obtaining the permit could impact the project schedule. To alleviate the potential concern, Bartlesville has been in close coordination with the DEQ and on November 13, 2018, received technical review approval.

- Public Opposition. Public and stakeholder acceptance will be critical to the success of the project. Bartlesville has the support and backing of the City Council, Citizen Oversight Committee, local leaders, and the regional stakeholders. Bartlesville has presented the project in public settings and received positive feedback. The City of Bartlesville has received letters in support of the proposed Feasibility Study from the Bartlesville Regional Chamber of Commerce, The Bartlesville Development Authority, the Bartlesville Fire Department, the City of Dewey, and other wholesale customers.

- The proposed project will involve constructing a pipeline requiring new easements as well as river, highway and railroad crossing permits. Bartlesville has been in contact with impacted property owners and will closely coordinate with the permitting entities to mitigate any issues that may delay the project.

3.4 REGULATORY AGENCIES JURISDICTION OVER THE PROJECT AREA

The Oklahoma Department of Environmental Quality (DEQ) is the primary state agency with authority over the project. DEQ has established rules and regulations for obtaining a discharge permit for the proposed secondary discharge to the Caney River for the reclaimed water as well as the required permit to construct the proposed infrastructures. DEQ issued a technical review and approval for the secondary discharge on November13, 2018, and submitted a request to the Environmental Protection Agency (EPA) review. Subsequent to the EPA review, DEQ will finalize the issuance of the OPDES permit for the proposed project.

The U.S. Army Corps of Engineers (USCOE) and U.S. Fish and Wildlife will have jurisdiction related to any potential environmental assessment and review for the project. The Oklahoma Water Quality Standards (OWQS) establish the environmental regulations to protect the fish and wildlife propagation in the Caney River. The proposed project will meet the Oklahoma Water Quality Standards (OWQS).

At the local level, Washington County Commissioners will have authority to grant the use of the public right-of-way for the proposed pipeline as well as for roadway crossing permits.

3.5 DESCRIPTION OF POTENTIAL SOURCES OF WATER TO BE RECLAIMED

The proposed reclaimed water is the treated effluent from the Bartlesville-owned Chickasaw Wastewater Treatment Plant (CWWTP). All wastewater flows generated in the Bartlesville service area are transported to CWWTP for treatment before discharge to the Caney River. The plant utilizes an advanced secondary treatment using the activated sludge process. Sludge generated from the plant is anaerobically digested for stabilization and pathogen reduction. Digested and treated sludge is beneficially used by land application on agricultural sites under a sludge disposal permit issued by DEQ.

The discharge from the CWWTP to Caney River is covered under OPDES permit number OK0030333. The discharge limits contained in the permit are summarized in Table 3-1. The plant currently has a permit design annual average day capacity of 7.0 MGD. The waste load allocation study completed under this feasibility study project provided the technical information necessary for regulatory approval by DEQ. DEQ has issued a technical review and approval for the proposed secondary discharge.

Table 3-1 Chickasaw Wastewater Treatment Plant OPDES Permit

SUMMARY OF THE EXISTING OPDES PERMIT CHICKASAW WASTEWATER TREATMENT PLANT					
	Discharge Limitations			**Monitoring Requirements**	
Effluent Characteristic	**Mass[1]**	**Concentration**		**Frequency**	**Sample Type**
	Monthly Average (lbs/day)	**Monthly Average (mg/L)**	**Weekly Average (mg/L)**		
BOD_5	583.8	10	15	3/week	12 hr Comp
TSS	875.7	15	22.5	3/week	12 hr Comp
Ammonia	116.8	2	3	3/week	12 hr Comp
Fecal Coliform (org/100 mL)	--	200	--	3/week	Grab
% Fecal Coliform exceeding 400 org/100 mL	--	--	10%	3/week	Grab
Total Chlorine Residual	--	< 0.1		Daily	Grab
1. Base flow for calculating the mass limits:		7.00	MGD		

Figure 3-1 shows the existing plant process schematics. Figure 3-2 shows the existing plant site plan.

Figure 3-1 Existing Chickasaw WWTP Process Schematic

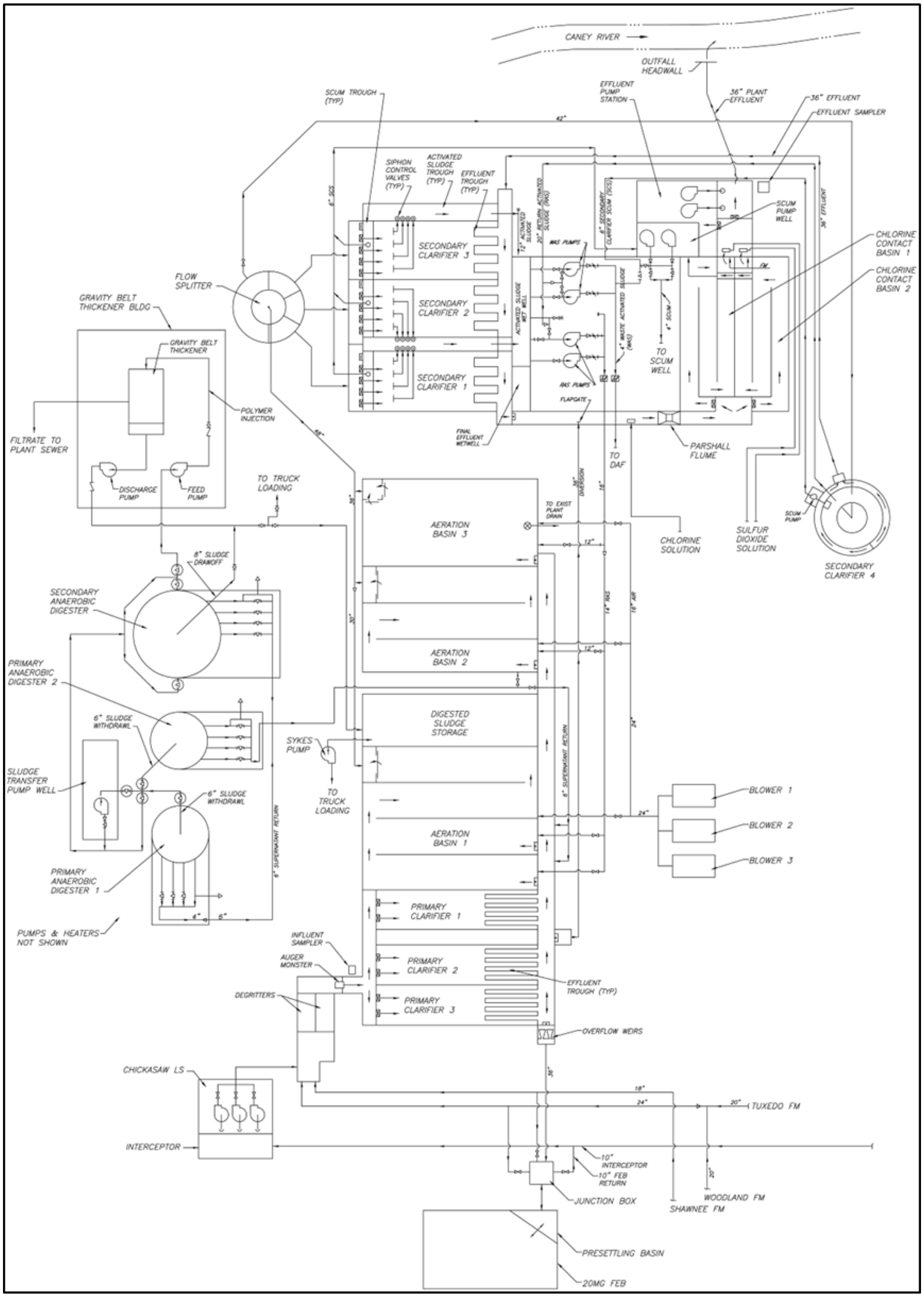
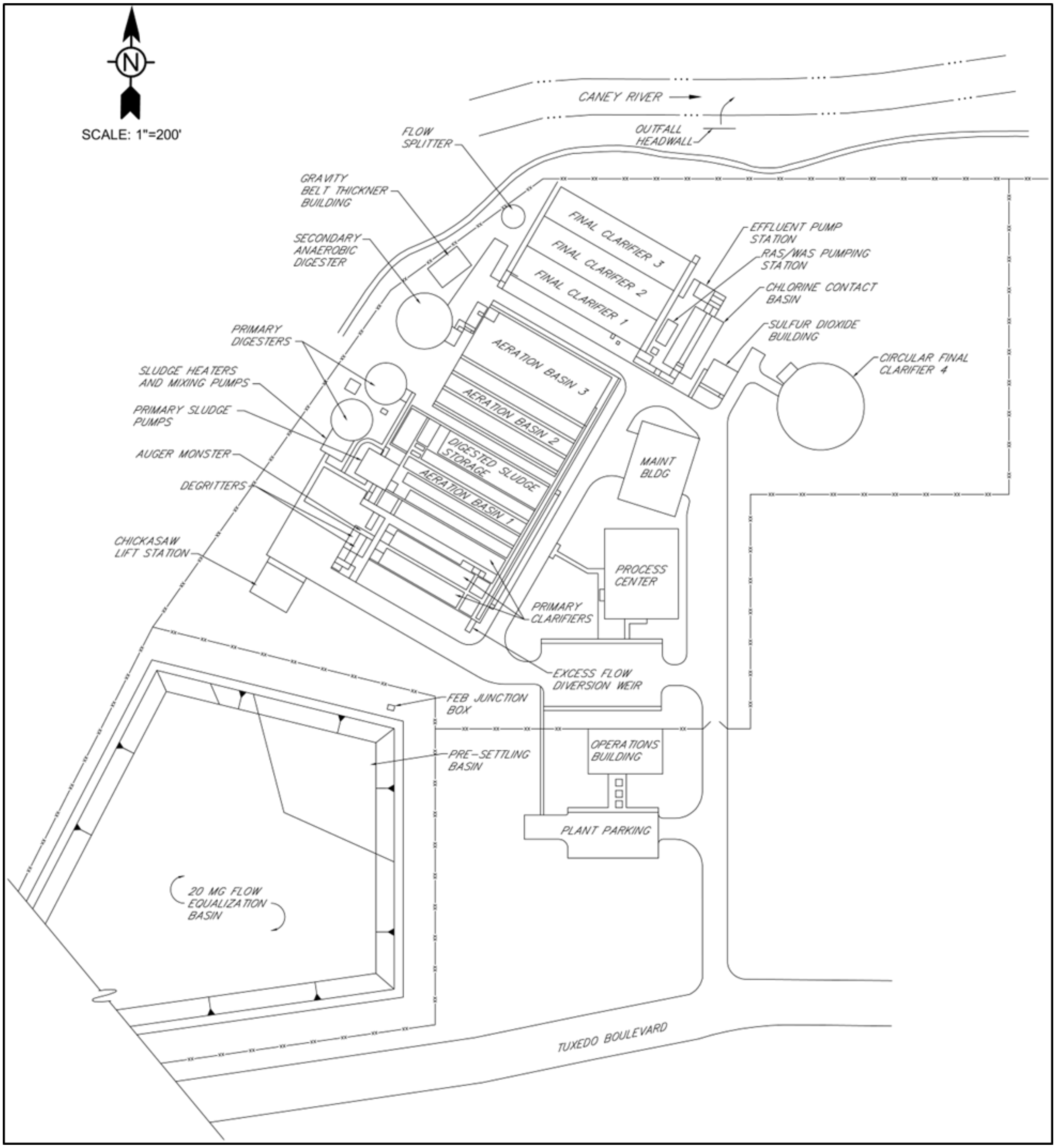


Figure 3-2 Existing Chickasaw WWTP Site Plan



3.6 DESCRIPTION OF CURRENT REUSE TAKING PLACE IN THE STUDY AREA

The concept of reclaiming treated effluent is a relatively new paradigm in Oklahoma. Currently, there is no known reuse taking place in the study area. The proposed project will be an innovative concept to reclaim the treated effluent to augment raw water supply for Bartlesville.

3.7 DESCRIPTION OF CURRENT AND PROJECTED WASTEWATER AND DISPOSAL OPTION OTHER THAN THE PROPOSED TITLE XVI PROJECT

The current practice of discharging treated effluent to the Caney River will continue under the DEQ authorized OPDES permit. The proposed project will reclaim a portion of the treated effluent (up to 4 MGD) and discharge to Caney River at a location approximately 5 to 7 miles upstream. The new discharge will be authorized under a new discharge permit from DEQ as part of the proposed project. There are no other changes to existing wastewater and disposal practices anticipated.

3.8 SUMMARY OF WATER RECLAMATION AND REUSE TECHNOLOGY CURRENTLY IN USE IN THIS AREA

With the exception of one commercial car wash that recycles a portion of their water, the Bartlesville study area does not practice reclamation and reuse. The proposed project will be the first major water reclamation and reuse opportunity in the study area.

4.0 DESCRIPTION OF ALTERNATIVES

4.1 ALTERNATIVES CONSIDERED

Since the severe drought of 2001-02, the City of Bartlesville has completed multiple studies focusing on new surface water impoundment with the most comprehensive being the U.S. Army Corps of Engineers Planning Assistance to States (PAS) study. This study titled, "Bartlesville Water Supply and Conveyance Study," was completed in December 2007. This study screened various options and worked with the City of Bartlesville to develop alternatives that have the highest potential to most effectively meet the city's water supply needs. The alternatives considered in this study included the following:

- Alternative 1- No Action. This option assumed no change in existing water supply sources.
- Alternative 2- Implement New Water Supply Agreements at Hulah and Copan Lakes.
- Alternative 3- Reallocate Flood Pool at Hulah and Copan Lakes to Water Supply.
- Alternative 4- New Private (Municipal) Sand Lake with Pipeline to Hudson Lake.
- Alternative 5- Purchase Water Supply Storage from Kaw Reservoir with Pipeline to Hudson Lake.

The PAS study did not consider the Caney River supply or reclaiming the treated effluent to augment Caney River supply as part of Bartlesville's water supply portfolio. Alternative 6 included in this feasibility study is as follows:

- Alternative 6- Reclaim up to 4 MGD of Treated Effluent as Drought- Resilient Water Supply.

4.2 NON-FEDERAL FUNDING CONDITIONS

The City of Bartlesville is the non-federal sponsor of the project and is committed to implementing the proposed reuse project regardless of the outcome of the Federal funding for the proposed project. Bartlesville will utilize water and sewer utility rates to pay for the non-federal portion of the project and implemented a 5-year incremental rate increase starting 2016 to pay for construction and implementation costs.

4.3 SPECIFIC OBJECTIVES FOR ALL ALTERNATIVES

Specific objectives for all alternatives, including the Title XVI Project, are as follows:

- Resilient water supply from drought and potential impacts from climate change.
- Economically viable.
- Compliance with environmental and regulatory approval requirements.
- Augment Bartlesville's short-term and long-term water supply needs.
- Stakeholder acceptance.

4.4 DESCRIPTION OF ALTERNATIVES

The alternatives considered in the study include the following:

- Alternative 1- No Action. This option assumed no change in existing water supply sources.
- Alternative 2- Implement New Water Supply Agreements at Hulah and Copan Lakes.
- Alternative 3- Reallocate Flood Pool at Hulah and Copan Lakes to Water Supply.
- Alternative 4- New Private Sand Lake with Pipeline to Hudson Lake.
- Alternative 5- Purchase Water Supply Storage from Kaw Reservoir with Pipeline to Hudson Lake.
- Alternative 6- Reclaim up to 4 MGD of Treated Effluent as Drought- Resilient Water Supply.

Each of the alternatives are discussed in more detail in the following section.

4.4.1 Alternative 1- No Action

This alternative will maintain the current practice of using Hulah/Hudson water supply as the primary and single source of water supply. The 2006 Hulah and Copan Reallocation Study completed by the U.S. Army Corps of Engineers (USACE) estimated that the City of Bartlesville has 6.4 MGD of dependable yield from this source through 2035 using historical data for the 50-year drought of record and the 2002 sediment survey data. Based on the 2002 sediment survey and assuming the trend continues, the USACE projected the dependable yield to decline from 6.4 MGD in year 2035 to 4.4 MGD in year 2055. This will not be adequate to meet the projected water demand of 9.0 MGD to 10.8 MGD by 2065.

This alternative does not meet the project objectives of meeting Bartlesville's long-term water supply needs. Therefore, the No Action alternative was not pursued and eliminated from further analysis.

4.4.2 Alternative 2- Implement New Water Supply Agreements at Hulah and Copan Lakes

This alternative evaluated the water supply yield availability through year 2055 assuming that new supply agreements proposed in the 2006 Hulah Copan Reallocation Study (by USACE) are implemented. The new water supply agreement approved by Headquarters USACE would provide the following:

- Hulah Lake:
  - 1,230 acre-feet (0.82 MGD) of new storage reallocated from water quality to water supply pool at Hulah Lake.
  - The existing pipeline infrastructure will be used to convey the additional water supply from Hulah.
- Copan Lake:
  - 2,185 acre-feet (0.97 MGD) of originally authorized water supply at Copan Lake.

- 10.305 acre-feet (4.57 MGD) of new storage reallocated from water quality to water supply at Copan Lake.
- Construct new infrastructure to include intake, pump station, and pipeline to convey the new source to the Bartlesville water treatment plant.

These three new agreements provide a total of 13,720 acre-feet (6.4 MGD); this, combined with the current Hulah water right contract of 12.74 MGD, will provide a total of 19.14 MGD in water supply. However, because of the continued sedimentation in Hulah and Copan Lakes, the available water supply storage will continue to decrease resulting in reduced water supply yield. The PAS study estimated the yield available at Hulah and Copan Lakes would total 6.85 MGD by year 2055 which will not meet the demand projection of 9.0 MGD to 10.8 MGD by 2065.

This alternative does not meet the project objectives of Bartlesville's long-term water supply; thus, this alternative was not pursued and eliminated from further analysis.

4.4.3 Alternative 3- Reallocate Flood Pool at Hulah and Copan Lakes to Water Supply

This alternative evaluated the potential water availability from a future reallocation of the flood pool to water supply at both Hulah and Copan Lakes. This alternative evaluated reallocating approximately 1%, 2.5%, 5% and 10% of the flood control storage at both Hulah and Copan Lakes. The study found that reallocation of 5% of the flood control pool at both lakes would provide a total yield of 16.76 MGD, 8.33 MGD from Hulah Lake and 8.43 MGD from Copan Lake. This alternative would increase the conservation pool at Hulah from elevation 733.0 to 736.67, an increase of 3.67 feet. For Copan Lake, the conservation pool would increase from elevation 710.0 to 711.99, an increase of 1.99 feet.

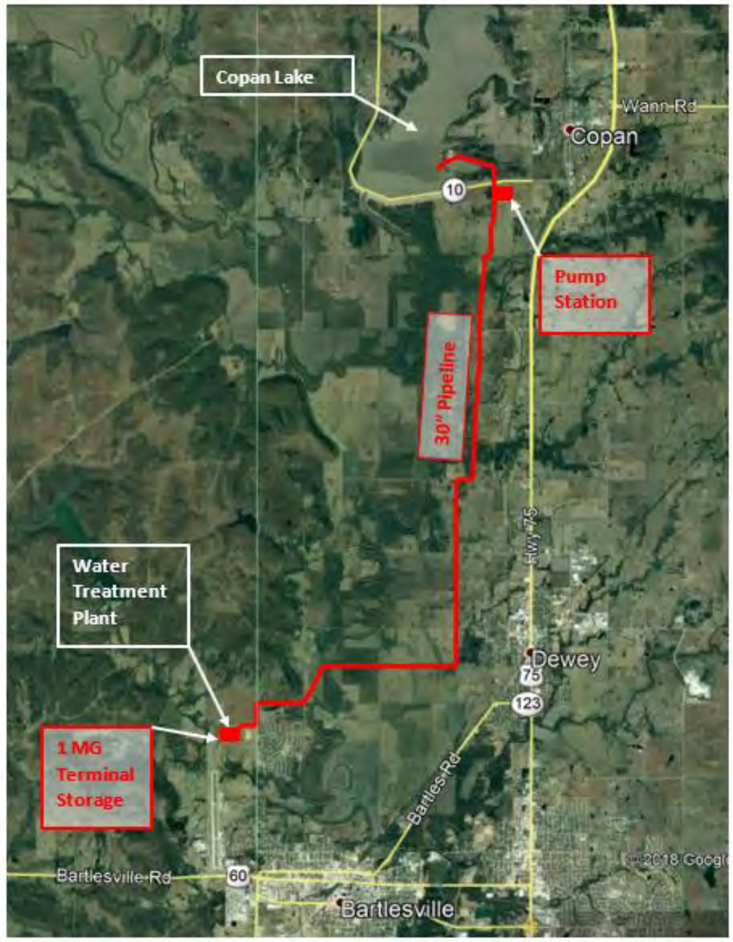
There is already existing infrastructure consisting of two 24-inch pipelines from Hulah Lake to Hudson Lake which is adequate to convey 8.33 MGD from Hulah. From Hudson Lake, the flow is by gravity via existing 30- and 36-inch pipes to the Ted Lockin Water Treatment Plant.

However, there is no existing intake or pipeline infrastructure at Copan Lake. The 2007 PAS study evaluated the option of conveying flow from Copan to Hudson Lake and proposed an intake structure at Copan and a 30-inch pipeline from Copan to Hudson Lake. The pipeline alignment was evaluated in the 2014 Tetra Tech study. Figure 4-1 shows the proposed pipeline alignment from Copan Lake to the water treatment plant based on this study's findings.

Following is the infrastructure summary for this alternative.

- 57,900 LF of 30-inch from Copan Lake to the WTP.
- New intake structure at Copan Lake.
- New raw water pump station with 900 HP pumping capacity for peak flow.
- Terminal storage tank at WTP = 1 MG.
- Easement assumed: 57,900 LF at 25-feet-wide easement = 31 acres

Figure 4-1 Pipeline from Copan Lake to Water Plant



Hulah and Copan Lakes are Federally owned. Reallocating the flood control pool will require additional water supply contracts with the US Army Corps of Engineers and mitigation activities to account for impacts to existing cultural and environmental facilities around the lake. The 2007 PAS study estimated this cost to be approximately $27.2 million for the additional water supply cost and $2.3 million for the cultural and environmental mitigation costs.

The capital cost for the purchase of new water storage was developed in the 2007 PAS study, and the cost is escalated to year 2018 using the Engineering News-Record (ENR) construction cost index. Cost for pipeline and pump station was developed in the 2014 Tetra Tech study, and the cost is escalated to year 2018 using ENR CCI. The capital costs are summarized in Table 4-1.

Table 4-1 Alternative 3-Capital Cost Estimate

Item Description	Quantity	Unit	Unit Price	Year 2007 / 2014	Year 2018
Purchase New Storage				Year 2007 ENR CCI = 7967	Year 2018 ENR CCI=11043
Purchase New Storage from Govt				$27,200,000	$37,701,800
Upstream Mitigation				$2,300,000	$3,188,100
Pipeline & Pump Station				Year 2014 ENR CCI = 9807	Year 2018 ENR CCI=11043
Land/Easement	31	Acres	$1,850	$57,350	$64,600
Acquisition	1	LS	$12,000	$12,000	$13,600
Pipeline	57,900	LF	$233	$13,490,700	$15,191,000
Highway Bore	1	EA	$105,000	$105,000	$118,300
Pump Station	900	HP	$3,512	$3,160,800	$3,559,200
Terminal Storage 1 MG Tank at WTP	1	EA	$750,000	$750,000	$844,600
Intake Structure & Shoreline Valve Vault	1	LS	$4,600,000	$4,600,000	$5,179,800
Engineering & SIOH	16%	LS		$3,548,136	$3,995,400
Contingency	25%			$6,430,997	$7,241,600
TOTAL				**$61,654,983**	**$77,098,000**

4.4.4 Alternative 4- New Private (Municipal) Sand Lake with Pipeline to Hudson Lake

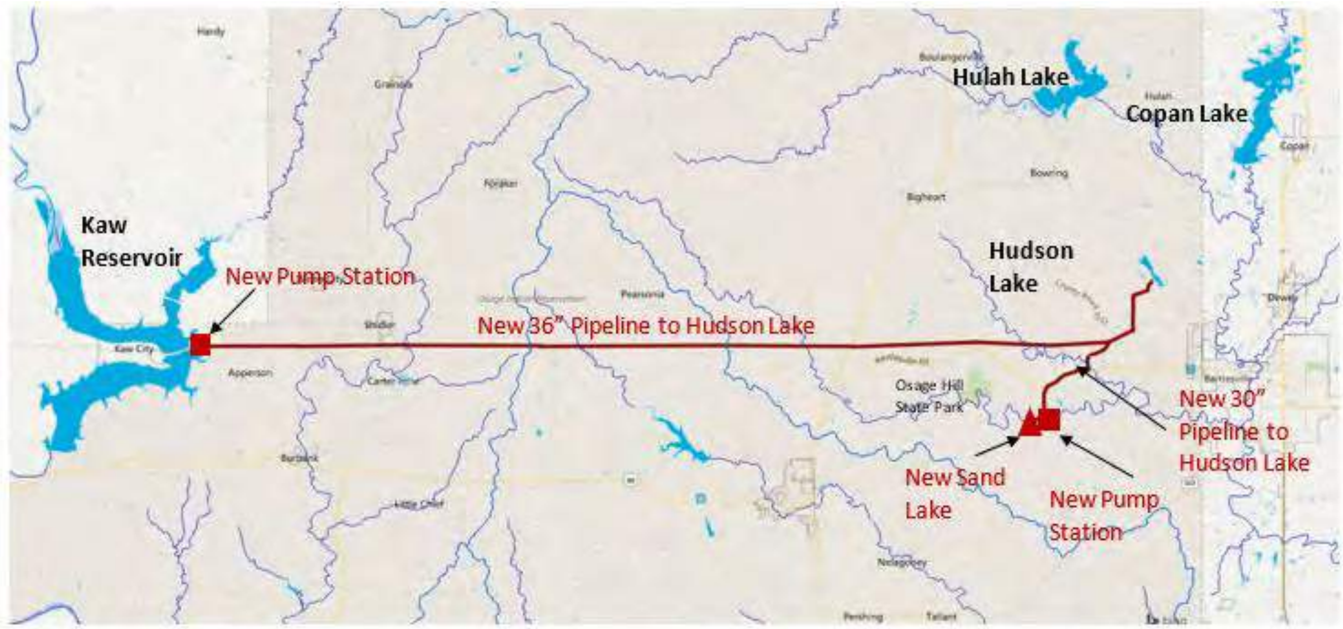
The 2007 PAS study evaluated this option based on information from a previous 1984 Reconnaissance Report. The reconnaissance report identified Sand Lake as a potential reservoir located on Sand Creek at mile 19.1 (upstream from the confluence of Caney River). This location was originally a federally authorized reservoir, but it was subsequently deauthorized. This alternative now assumes no federal authorization. The PAS study assumed the location to be at or near the deauthorized Sand Lake site. The location is about 8.5 miles west and 1.5 miles south of Bartlesville on Sand Creek in Osage County, just upstream of the Town of Okesa. The yield at the authorized site was projected to be about 12 MGD.

The study estimated a pump station and a 36-inch pipeline to transfer raw water from Sand Lake to Lake Hudson. The pipeline is assumed to run northeast from the dam site, then follow US-60 for several miles. The pipeline will leave the highway alignment and run northeast, then north to an arm of Hudson Lake. Figure 4-2 shows the proposed Sand Lake location and the pipeline alignment.

The Infrastructure Summary from the 2007 PAS Study is as follows:

- 49,915 LF of 36-inch from Sand Lake to Hudson Lake
- Intake structure at Sand Lake
- Raw water pump station 1,745 HP total for peak flow
- Easement assumed: 49,915 LF at 25-feet-wide easement = 29 acres

Figure 4-2 Alternatives 4 and 5- Pipelines from Kaw Reservoir and New Sand Lake



The 2007 PAS study developed construction costs for the infrastructure for this alternative. The 2007 PAS study also identified the following factors which may negatively impact the implementation of this alternative:

- The proposed Sand Lake would likely flood some portion of the Osage Hills State Park which requires further evaluation and future mitigation measures.
- The proposed Sand Lake would likely flood a significant portion of the Boy Scout Camp located upstream which requires further evaluation and future mitigation measures.
- The proposed Sand Lake is within Osage County and within the Osage Reservation subject to mineral rights. Future study would be required to establish mineral rights cost associated with the new lake.

The capital cost for the purchase of new water storage was developed in the 2007 PAS study, and the cost is escalated to year 2018 using the ENR construction cost index. The capital costs are summarized in Table 4-2.

Table 4-2 Alternative 4- Capital Cost Estimate

Item Description	Quantity	Unit	Unit Price	Year 2007	Year 2018
Pipeline & Pump Station				Year 2007 ENR CCI = 7967	Year 2018 ENR CCI=11043
Land/Easement	29	Acres	$1,500	$43,500	$60,300
Acquisition	1	LS	$10,000	$10,000	$13,900
Pipeline	49,915	LF	$216	$10,781,640	$14,944,400
Highway Bore	1	EA	$70,000	$70,000	$97,100
Pump Station	1,745	HP	$2,855	$4,981,975	$6,905,500
Intake Structure	1	LS	$620,000	$620,000	$859,400
Engineering	1	LS	$1,645,000	$1,645,000	$2,280,200
S.I.O.H (Supervision, Inspection & Overhead)	1	LS	$987,000	$987,000	$1,368,100
Contingency	25%			4,784,779	$6,632,200
SUBTOTAL				$23,923,894	$33,161,100
Sand Lake & Land				Year 2007 ENR CCI = 7967	Year 2018 ENR CCI=11043
Land Cost	4,300	Acres	$1,500	$6,450,000	$8,940,300
Residential Relocation	4	EA	$200,000	$800,000	$1,108,900
Land Acquisition	1	LS	$400,000	$400,000	$554,500
Infrastructure & Facility Relocation	1	LS	$3,875,000	$3,875,000	$5,371,200
Dam/Equipment/Building	1	LS	$22,866,000	$22,866,000	$31,694,400
Recreation	1	LS	$1,500,000	$1,500,000	$2,079,200
Engineering	1	LS	$2,824,000	$2,824,000	$3,914,400
S.I.O.H (Supervision, Inspection & Overhead)	1	LS	$1,694,000	$1,694,000	$2,348,100
Contingency	25%			$10,102,250	$14,002,700
SUBTOTAL				$50,511,250	$70,013,700
TOTAL				**$74,435,144**	**$103,174,800**

4.4.5 Alternative 5- Purchase Water Supply Storage from Kaw Reservoir with Pipeline to Hudson Lake

This alternative will purchase water supply from the Kaw Reservoir and pump raw water from Kaw Reservoir to Hudson Lake. The 2007 PAS study evaluated this option which is summarized here. The new Kaw pipeline would begin from a new water intake structure in Kaw Reservoir just off the southeast bank of the Highway 11 (SH-11) crossing. The transmission line would then traverse an east/west section line alignment parallel and just south of SH-11 to Shidler. From there the pipeline will share/parallel a high voltage power transmission line. This alignment runs cross country to the east until crossing Highway 99 where it then parallels Highway 60 to near the location of Lake Hudson.

The study estimated a pump station and a 36-inch pipeline to transfer raw water from Kaw Reservoir to Lake Hudson. Figure 4-2 shows the proposed pipeline from Kaw Reservoir to Lake Hudson.

The Infrastructure Summary from the 2007 PAS Study is as follows:

- 238,266 of 36-inch from Kaw Reservoir to Hudson Lake
- Intake structure at Kaw Reservoir
- Raw water pump station 2,575 HP total for peak flow
- Easement assumed: 238,266 LF at 25-feet-wide easement = 137 acres

The capital cost for the purchase of new water storage was developed in the 2007 PAS study, and the cost is escalated to year 2018 using Engineers News Record (ENR) construction cost index. The capital costs are summarized in Table 4-3.

Table 4-3 Alternative 5- Capital Cost Estimate

Item Description	**Quantity**	**Unit**	**Unit Price**	**Year 2007**	**Year 2018**
Pipeline & Pump Station				Year 2007 ENR CCI = 7967	Year 2018 ENR CCI=11043
Land/Easement	137	Acres	$1,500	$205,500	$284,900
Acquisition	1	LS	$40,000	$40,000	$55,500
Pipeline	238,266	LF	$216	$51,465,456	$71,335,900
Highway Bore	2	EA	$70,000	$140,000	$194,100
Pump Station	2,547	HP	$2,855	$7,271,685	$10,079,300
Intake Structure	1	LS	$620,000	$620,000	$859,400
Engineering	1	LS	$5,950,000	$5,950,000	$8,247,300
SIOH (Supervision, Inspection & Overhead)	1	LS	$3,570,000	$3,570,000	$4,948,400
Contingency	25%			$17,315,660	$24,001,200
TOTAL				**$86,578,301**	**$120,006,000**

4.4.6 Alternative 6- Reclaim up to 4 MGD of Treated Effluent as Drought- Resilient Water Supply

The existing Chickasaw Wastewater Treatment Plant (CWWTP) has an average permitted capacity of 7.0 MGD. This alternative proposes to reclaim up to 4 MGD of the treated effluent and use it to augment the Caney River water supply.

Bartlesville currently maintains and operates the Caney River Raw Water Pump Station (CRWPS) located approximately 0.25 miles upstream of the CWWTP. Bartlesville already holds 5.4 MGD in water rights from the River. The Caney River flow is in part dependent upon the upstream release from Hulah and Copan Lakes and is unreliable due to impacts from drought and climate change. The current CWWTP discharge is located downstream of the CRWPS. The proposed project will divert up to 4 MGD of treated effluent from CWWTP and discharge it approximately 7 miles upstream of the CRWPS. This would augment the Caney River flow and provide a drought-resilient, long-term water supply for Bartlesville.

Modifying the current effluent discharge location will require compliance with the Oklahoma Water Quality Standards (OWQS) and regulatory approval from the Oklahoma Department of Environmental Quality (DEQ). For regulatory approval, DEQ determined that the proposed discharge must comply with the water quality standards for the Caney River. The following summarizes the various efforts completed to date to secure technical approval from DEQ.

In August 2016, Tetra Tech completed the report titled, *Caney River QUAL2K Scoping Model near Bartlesville*. The purpose of this report was to utilize a desktop model analysis (using EPA QUAL2K model) to provide a preliminary evaluation of the impact of the proposed effluent discharge location. This study confirmed assimilative capacity of the Caney River for the new discharge location; however, a more detailed field monitoring and modeling efforts were recommended to more accurately establish the assimilative capacity and reduce uncertainty of key modeling parameters. This report was presented to and concurred by DEQ.

In July 2017, Tetra Tech completed the report titled *Monitoring Study Plan, Caney River TMDL Study for Chickasaw Wastewater Treatment Plant*. The study plan described the monitoring and modeling protocols to gather necessary field data for use in the subsequent wasteload allocation study required by DEQ. This plan was approved by DEQ in July 2017.

Field samplings were collected during two different flow regimes in the Caney River. Sampling for various water quality parameters was obtained for the 21-mile segment of the Caney River. The first sampling was completed September 6-11, 2017, at a river flow of 24.3 cubic feet per second (cfs) which represents summer low flow conditions. The second sampling was completed October 2-6, 2017, at a river flow of 96.7 cfs which represents median flow in the river. The field sampling data was used to calibrate and corroborate the Caney River wasteload allocation model. A report titled, *Bartlesville WLA*

Studies- Caney River Monitoring and Modeling Report, was prepared by Tera Tech and submitted to DEQ in April 2018.

From April 2018 to November 2018, Bartlesville and Tetra Tech coordinated and worked with DEQ to further refine the report findings and respond to DEQ's comments. The wasteload allocation study concluded that the proposed discharge will meet the Oklahoma water quality standards for the Caney River at a discharge flow of up to 4.1 MGD. Additionally, the study found the upstream discharge location of either 5 or 7 miles upstream of the existing intake structure will be suitable to meet the water quality standards. Bartlesville proposes to use the 7-mile upstream location in part due to input received from local stakeholders and residents who live near the proposed pipeline alignment and the discharge location. The proposed discharge limits for the new discharge are as follows:

- Year around: 10.0 mg/l, BOD5; 1.0 mg/l, NH3-N; 6.0 mg/l, DO; 15.0 mg/l.
  - The existing Chickasaw Wastewater Treatment Plant is designed to achieve this level of effluent treatment, and therefore, additional advanced treatment is not necessary.
- No discharge during April 1 until June 15.
  - Oklahoma water quality standards require a higher (6 mg/l versus 5 mg/l during summer) dissolved oxygen (DO) during the spring season to promote early life fish and wild life propagation. Bartlesville proposes no discharge during the spring season to meet the standard. During spring all the effluent will be discharged at the current discharge location under existing discharge permit.

DEQ concurred with the report findings and issued technical approval on November 13, 2018. This approval completes the major regulatory milestone for the proposed project. DEQ submitted the request to EPA for review on November 13, 2018. The next phase of the regulatory approval is the completion of a formal review by EPA and issuance of a final permit. These efforts will be completed as part of the construction phase of the project in 2019.

The proposed Title XVI Project concept is shown on Figure 2-2 . The infrastructure proposed for the project are an effluent pump station, effluent pipeline, and a powder activated carbon (PAC) feed system as summarized below:

Effluent Pipeline. Approximately 18,500 linear feet of 18-inch effluent pipeline is proposed to convey up to 4 MGD effluent from the wastewater treatment plant to the 7-mile upstream discharge location. The proposed alignment is relatively flat with a ground relief of within 10 feet except where it crossed the Caney River. At the end of the discharge location, a natural cascade aeration structure will be provided to naturally add a minimum of 6 mg/l dissolved oxygen prior to discharge to Caney River. Adequate erosion control measures will be incorporated in and around the discharge location.

Effluent Pump Station. A new effluent station will be located within the wastewater treatment plant boundary and adjacent to the existing chlorine contact basin. Two 4 MGD vertical turbine pumps (1-

duty, 1-standby) with variable frequency drives will be provided to allow discharging of flow from 2 MGD to 4 MGD to meet varying demand. A review of the proposed pipeline alignment shows relatively flat terrain. The pump station elevation is approximately 667, and the discharge elevation at the 7-mile upstream location is approximately 677 feet. The proposed vertical turbine pumps would be 75% efficient. Each pump will be approximately 65-HP.

Powder Activated Carbon (PAC) Feed Facility. Caney River water supply experiences occasional episodes of taste and odor issues as a result of algae activities especially during the warm weather period. Taste and odor are primarily attributed to elevated levels of Geosmin and MIB (2-Methyl isoborneol) as a result of algal activities. Use of PAC is an effective strategy to mitigate the taste and odor episodes. A new 1000-square-foot building with a PAC feed facility will be located near the existing 30-inch raw water pipeline near the Caney raw water intake. This location is ideal to inject PAC into the 30-inch raw waterlines. Based on prior experience, the use of the PAC feed system should be necessary for a 4 to 6 month period to address sporadic algal bloom.

The estimated construction cost for the alternative is summarized in Table 4-4.

Table 4-4 Alternative 6 (Title XVI Project)- Capital Cost Estimate

Item Description	Quantity	Unit	Unit Price	Total (Year 2018)
Pipeline & Pump Station				
Land/Easement	11	Acres	$1,850	$20,350
Acquisition	1	LS	$5,000	$5,000
Pipeline	18,500	LF	$258	$4,773,000
Highway, RR & Caney Crossing	3	EA	$115,000	$345,000
Discharge Structure & Erosion Control	1	EA	$155,000	$155,000
Pump Station	65	HP	$10,385	$675,025
Engineering & SIOH	16%	LS		$955,124
Contingency	25%			$1,731,162
PAC Feed System				
Chemical Building	1,000	SF	$260	$260,000
Flowmeter & Vault	1	EA	$75,000	$75,000
PAC Feed Equipment	1	EA	$250,000	$250,000
Engineering & SIOH	16%	LS		$93,600
Contingency	25%			$169,650
TOTAL				**$9,509,644**

5.0 ECONOMIC ANALYSIS

5.1 INTRODUCTION

The economic analysis provides a cost comparison of all alternatives that would satisfy the long-term water supply needs as the proposed Title XVI project. The analysis includes capital cost, maintenance cost, energy cost, replacement cost as applicable, and the life cycle present worth analysis. The estimates are developed at the planning level stage of the proposed project, and the estimate produced is expected to be at the appraisal level or better.

Capital Costs. Alternatives 1 and 2 were eliminated from analysis since they did not meet the project objectives. Alternatives 3, 4, and 5 were originally developed in the 2007 PAS Study prepared by the US Army Corps of Engineers. Capital costs from the 2007 study were adjusted for inflation using the ENR Construction Cost Index (CCI) for use in this study. The pipeline cost estimate for Alternate 3 was used from the 2014 Tetra Tech study and adjusted for escalation using ENR CCI. Cost estimate for Alternative 6 - the Title XVI project - was developed for this study. Capital cost includes 16% for engineering and supervision, inspection and overhead (SIOH) and a construction cost contingency of 25% appropriate for planning level estimates.

Maintenance Costs. The following annual maintenance costs are assumed for all the alternatives:

- Pipelines: 0.1% of the initial cost annually
- Pump Stations: 0.25% of the initial cost annually
- Lakes/Reservoirs: Assumed comparable and common to each alternative
- Powder Activated Carbon: $0.8025/lb

Energy Costs. Pumping will constitute the predominant energy cost for each alternative. The following electric rates are assumed based on current rates Bartlesville is charged:

- Energy Usage: $0.072/kWH
- Demand Charge: $6.59/kW/month (PSO Primary 246 Rate)

Replacement Cost. The assumed expected life is as follows:

- Reservoirs/Lakes: 100 years or more
- Structures/Pipeline: 75 years or more
- Pumps: 25 years

The life cycle cost is developed for a period of 25 years. Replacement cost for pumps will be assumed at $500/HP at the 25th year.

Life Cycle Cost. The following assumptions are included in the life cycle cost analysis.

- Life Cycle Period: 25 years
- Discount Rate: 3.0%
- Future Escalation: 3.0% annual

Residual Value. Residual value for the replacement at 25 years is assumed negligible for use in the life cycle cost analysis.

5.2 CAPITAL COST SUMMARY

Capital cost is summarized on

Table 5-1 in terms of total initial project cost. Alternative 6 (Title XVI Project) has the least initial cost as well and the highest ranking in terms of cost per MGD or acre-feet.

Table 5-1 Capital Cost Summary of Alternatives

CAPITAL COST SUMMARY OF ALTERNATIVES							
Alt. No	**Description**	**Total Initial Cost (Year 2018)**	**Estimated Yield (MGD)**	**Estimated Annual Yield (ac-feet)**	**Cost/MGD**	**Cost/ac-feet**	**Rank**
1	No Action- Will not meet Project Objectives- Eliminated.	Does not meet project objectives- Eliminated					
2	Implement New Water Supply Agreement at Hulah and Copan Lakes- Will not meet Project Objectives- Eliminated.						
3	Reallocate Flood Pool at Hulah and Copan Lakes to Water Supply	$77,098,000	16.8	18,820	$4,589,167	$4,097	**2**
4	New Private (Municipal) Sand Lake with Pipeline to Hudson Lake	$103,174,800	12.0	13,443	$8,597,900	$7,675	**3**
5	Purchase Water supply Storage from Kaw Reservoir with Pipeline to Hudson Lake	$120,006,000	10.5	11,706	$11,483,828	$10,251	**4**
6	Title XVI Project: Reclaim up to 4 MGD of Treated Effluent as Drought-Resilient Water Supply	$9,509,644	4.0	4,481	$2,377,411	$2,121	**1**

5.3 LIFE CYCLE COST SUMMARY

Life cycle cost is summarized on Table 5-2 in terms of present worth total cost. Alternative 6 (Title XVI Project) has the least initial cost as well and the highest ranking in terms of life cycle cost.

Table 5-2 Life Cycle Cost Summary of Alternatives

Life Cycle Cost Items	Alternatives			
	3	4	5	6
Capital Cost:				
Total Initial Cost	**$77,098,000**	**$103,174,800**	**$120,006,000**	**$9,509,644**
Annual Maintenance Cost				
Pipeline				
Pipeline Initial Cost	$21,696,375	$21,304,769	$101,290,560	$7,470,674
0.1% of Initial Cost	$21,696	$21,305	$101,291	$7,471
Pump Station				
Pump Station Initial Cost	$14,511,725	$11,856,331	$18,715,440	$1,190,720
0.25% of Initial Cost	$36,279	$29,641	$46,789	$2,977
Powder Activated Carbon (PAC)				
Usage (4 MGD x 40 mg/l x 15% of the time). LB/Yr				73,058
Annual cost (@$0.8025/LB)				$58,629
Total Annual Maintenance Cost	$57,976	$50,946	$148,079	$69,071
PW Annual Maintenance (25-Yr, 3.0% / 3.0% Escalation: Factor: 24.272)	**$1,407,186**	**$1,236,552**	**$3,594,177**	**$1,676,625**
Annual Energy Cost				
Flow Yield	8.43	12	10.5	4
Maximum HP	900	1745	2547	65
Maximum HP (Baseline 4 MGD Flow)	203	194	370	65
Average Annual kWH	1,322,415	1,265,358	2,412,297	424,203
kWH Use Charge, $/kWH	$0.072	$0.072	$0.072	$0.072
Annual Energy Use Charge	$95,214	$91,106	$173,685	$30,543
Demand Charge ($/kW/Month)	$6.59	$6.59	$6.59	$6.59
Annual Demand Charge	$11,938	$11,423	$21,777	$3,829
Total Annual Energy Cost	$107,152	$102,529	$195,462	$34,372
PW Annual Energy Cost (25-Yr, 3.0% /3.0% Escalation: Factor:24.272)	**$2,600,789**	**$2,488,575**	**$4,744,256**	**$834,279**
Replacement Cost				
Future Replacement Cost (@25-yrs, $500/HP)	$450,000	$872,500	$1,273,500	$32,500
PW Replacement Cost (25-Yr, 3.0%, Factor=0.4776)	**$214,920**	**$416,706**	**$608,224**	**$15,522**
Total PW Cost of Alternative	**$81,320,895**	**$107,316,633**	**$128,952,657**	**$12,036,070**
Ranking	**2**	**3**	**4**	**1**
Note: Alternatives 1 and 2 were eliminated and not included in the table				

5.4 NON-ECONOMIC CONSIDERATION

The proposed Title XVI project ranks the highest as demonstrated by the economic analysis. There are other non-economic factors that are difficult to quantify, and the following discussion summarizes the benefit of the proposed project in meeting the project objectives.

5.4.1 Resilient Water Supply from Drought and Potential Impacts from Climate Change

The proposed Title XVI project (Alternate 6) offers the most resiliency from drought and potential impacts from climate change. The existing Chickasaw Wastewater Treatment Plant is permitted for an effluent discharge flow of 7.0 MGD. The proposed reclamation of 4 MGD represents approximately 57% of the current effluent capacity. As Bartlesville grows, effluent flow will become increasingly available. Reclamation of treated effluent offers a drought-resilient water supply for Bartlesville.

The other three alternatives rely on surface water supplies from federally owned water supply (Hulah, Copan, and Kaw) or a future lake (Sand Lake). These surface water supplies are prone to the impacts from severe drought and the climate change effects and storage loss due to sedimentation.

5.4.2 Compliance with Environmental and Regulatory Approval

The proposed Title XVI project will require a new discharge permit from the Oklahoma DEQ. Bartlesville completed the wasteload allocation study in 2018 and obtained the technical approval from DEQ for the proposed discharge location. The construction of the effluent pump station and the reuse effluent pipeline will also require a permit to construct from the DEQ, and acceptance from state and local jurisdictions as well as impacted land owners for the pipeline alignment. The effluent pump station will be located within the property boundary of the wastewater treatment plant, and the construction will meet the standards established by DEQ. There is broad support within the Bartlesville community including the City Council, local business community, public, and the wholesale customers for the proposed project.

The development of the additional water supply from the Copan/Hulah water sources (Alternative 3) will require significant efforts and approval from the federal government. It involves raising the flood pool elevation and requires remediation of upstream cultural and natural resources that will be impacted by the normal pool increase. Additional evaluation will be necessary to more fully define the impacts and mitigation costs.

The development of Sand Lake (Alternative 4) will require multi-jurisdictional review and approval from Federal, tribal, State and local agencies. Additional investigation will be necessary to establish the impact of mineral and water rights associated with the development of this source.

The development of the Kaw Lake water supply (Alternative 5) involves more than 45 miles of cross-country pipeline across many drainage basins and Osage Reservation land. The procurement of the cross-country easement will be a challenge. Water quality from Kaw Lake is different than Hulah Lake. Co-mingling of the two waters could require additional improvements for storage and treatment at the Bartlesville treatment plant. Pilot testing and study will be necessary to better quantify the impacts.

5.4.3 Augment Bartlesville's Short-term and Long-term Water Supply Needs

The proposed Title XVI project is conducive for implementation within a short period (three years or less) providing immediate augmentation with drought-resilient water supply. This project will also be an effective part of Bartlesville's water supply portfolio to meet long-term needs.

The other three alternatives considered will provide a long-term water supply though they will not be drought-resilient. The scope of the other three alternatives will also require a longer period to implement compared to the Title XVI project.

5.4.4 Stakeholder Acceptance

Bartlesville has broad support within its community and stakeholders for the proposed Title XVI project. The proposed Title XVI project follows the Oklahoma Water for 2060 Act which has set a goal of consuming no more fresh water in 2060 than consumed statewide in 2012. The project fosters the statewide efforts towards water recycle and reuse.

6.0 PROPOSED TITLE XVI PROJECT

The proposed Title XVI project is Alternative 6, which will reclaim up to 4 MGD of the treated effluent and use it to augment the Caney River water supply based on the following justifications.

6.1 JUSTIFICATION IN TERMS OF MEETING PROJECT OBJECTIVES, DEMANDS AND COST EFFECTIVENESS

- The proposed Title XVI project is the only alternative that met all the project objectives, especially, the need to have a drought-resilient water supply.

The proposed Title XVI project scored the lowest initial capital cost as well as the present worth life cycle costs. The proposed project cost is approximately 48% less than the next viable alternative in terms of both capital and life cycle costs. Refer to

- Table 5-1 and Table 5-2 .
- The proposed Title XVI project has the fastest implementation time frame compared to the other alternatives and is conducive for implementation within a 3-year period allowing Bartlesville to realize its short-term water drought-resilient water supply needs. Other alternatives have substantially higher initial costs that will stretch Bartlesville's financial flexibility and will also require a longer implementation period.

6.2 ANALYSIS AND AFFIRMATIVE STATEMENT ON SPECIFIC ISSUES

- The proposed Title XVI project will become an effective part of the water supply portfolio for Bartlesville and will postpone the need to develop new sources to meet Bartlesville's long-term water supply needs. This provides additional time to further investigate and develop the new water sources discussed in the other alternatives.
- With the use of reclaimed water, Bartlesville will be able to lessen the reliance on the existing Hulah water supply (Federal water supply). However, this may not reduce the overall demand on existing Federal water supplies.
- The proposed project will have minimal impact on the operation of the existing wastewater treatment plant. The proposed effluent pump station will be located within the wastewater treatment facility, and its maintenance and operation will be synonymous with existing facilities in which plant operators are already performing. No reduction, postponement, or elimination of new or expanded wastewater facilities are anticipated.

7.0 ENVIRONMENTAL CONSIDERATIONS AND POTENTIAL EFFECTS

A full NEPA compliance evaluation has not been conducted for the proposed Title XVI project. Bartlesville has secured a separate USBOR Drought Resiliency Project grant for the design and construction of the proposed project. Full NEPA evaluation compliance will be done as part of the design phase of the project in 2019.

7.1 POTENTIAL IMPACT ON ENDANGERED OR THREATENED SPECIES, PUBLIC HEALTH OR SAFETY, NATURAL RESOURCES, REGULATED WATERS OF THE UNITED STATES

Clean Water Act. The treated effluent discharge to the Caney River is subject to the Clean Water Act requirements and will require an OPDES permit from the DEQ. Bartlesville has completed the wasteload allocation study and received technical approval from the DEQ for the proposed discharge location. The process to obtain a final DEQ permit is scheduled for early 2019.

Endangered or Threatened Species. Federally endangered and threatened species in Oklahoma include: American Burying Beetle, Gray Bat, Indiana Bat, Interior Least Tern, Neosho Mucket, Ouachita Rock Pocketbook, Ozark Big-eared Bat, Piping Plover, Red-cockaded Woodpecker, Scaleshell, Whooping Crane, Winged Mapleleaf, Arkansas River Shiner, Leopard Darter, Neosho Madtom, Northern Long-eared bat, Ozark Cavefish, Rabbitsfoot and Rufa Red Knot. The state threatened species is Blackside Darter.

The proposed project will include an effluent pump station, and chemical feed building that will be located within property currently owned by Bartlesville. No impacts are anticipated to any endangered or threatened species at these sites. The buried effluent line will require excavation and backfill activities and generally follow the existing rights-of-way. As part of the design phase efforts in 2019, Bartlesville will conduct a survey to identify the presence of the American Burying Beetle or bat species along the proposed alignment and will include mitigation measures, if warranted.

Biological Resources. The proposed discharge will comply with the OPDES discharge permit in compliance with Oklahoma Water Quality Standards of protection of the fish and wildlife and the biological resources in the Caney River. A preliminary review of the proposed effluent pipeline indicates no significant impact to the potential wetlands along the proposed pipeline. As part of the design phase efforts in 2019, a field reconnaissance survey and coordination with US Fish and Wildlife and the US Army Corps of Engineers will be conducted to avoid and/or mitigate any impact.

Cultural Resources. It is anticipated that the proposed project should have no or minimal impact on potential archeological sites. As part of the design phase efforts in 2019, Bartlesville will request review from the Oklahoma Archeological Survey for the proposed pipeline alignment and take appropriate mitigative measures.

7.2 POTENTIAL SIGNIFICANT ENVIRONMENTAL EFFECTS OF THE PROJECT

The project construction phase will have a temporary impact on the environment related to noise, dust control, and local traffic. The construction contract documents will mandate necessary measures from the project contractor to minimize and mitigate these temporary impacts. Bartlesville does not anticipate any other significant environmental effects from the proposed project.

7.3 STATUS OF REQUIRED FEDERAL, STATE, TRIBAL, AND/OR LOCAL ENVIRONMENTAL COMPLIANCE MEASURES

As part of the design phase of the project in 2019, Bartlesville will complete the EA for the proposed effluent pipeline in compliance with NEPA guidance. Preliminary approval for the proposed discharge was received in November 2018 from DEQ, and a formal discharge permit process is anticipated to be completed in early 2019.

The proposed project does not involve Tribal land or review. The proposed effluent pipeline alignment will require review and approval by the Washington County Commissioners. Bartlesville has the support of the county and the local leaders for the proposed project.

7.4 OTHER INFORMATION TO ASSIST WITH NEPA COMPLIANCE

There is no other specific information identified for the project.

7.5 TITLE XVI PROJECT IMPACT ON WATER SUPPLY AND WATER QUALITY

Bartlesville is a regional water supply serving the surrounding communities of Washington County Rural Water District (RWD) #2, Washington County RWD#1, Town of Ochelata, Town of Ramona, City of Dewey, Strike Axe Water system, and the Bar Dew water system. The proposed Title XVI project will not only help the citizens of Bartlesville but also the surrounding communities in providing a drought-resilient, long-term water supply.

Bartlesville is in Basin 76 of the Middle Arkansas Watershed Planning region as published by the 2012 Oklahoma Comprehensive Plan (2012 OCWP). The 2012 OCWP identified a water supply gap in this basin by 2020 even without considering the impacts from climate change and drought. The proposed Title XVI project will provide a drought resilient water source to address the water supply gap.

The effluent water quality will meet the Oklahoma Water Quality Standards of protection of the Caney River water supply. The Caney River water supply will be further treated at the Bartlesville water treatment plant in compliance with the Safe Drinking Water Act to potable water quality standards. The additional 4 MGD reclaimed water flow during the summer low flow conditions could further augment the flow and velocity in the river segment to provide additional benefits such as increased surface aeration and dissolved oxygen. Though it is difficult to quantify, the increased flow velocity and the resulting turbulence could also help to somewhat limit the algal bloom.

7.6 PUBLIC INVOLVEMENT IN THE FEASIBILITY STUDY

Bartlesville has received broad support from community leaders including the Bartlesville Chamber of Commerce, Bartlesville Development Authority, Bartlesville Fire Department, City of Dewey, Washington County RWD#2, Osage County RWD#1, and Washington County RWD#5. Bartlesville has the support of its City Council and state elected leaders. For the past two years, Bartlesville has conducted multiple public information meetings and received positive feedback in support of the project.

Bartlesville has presented the project concept in multiple council meetings that are televised for public benefit. These presentations have not received any adverse comments. As part of these feasibility study efforts, Bartlesville has reached out to the stakeholders along the proposed pipeline and received positive feedback. Bartlesville believes there will not be any significant opposition to the acceptance and implementation of the project.

7.7 POTENTIAL IMPACT ON HISTORIC PROPERTY

Preliminary walkthrough of the proposed project pipeline did not identify any historic property along the pipeline alignment. The proposed project will not impact any historic property in the project area. As part of the design phase efforts in 2019, a formal review will be requested from the Oklahoma Historical Society.

8.0 LEGAL, REGULATORY, AND INSTITUTIONAL REQUIREMENTS

8.1 WATER RIGHTS

Bartlesville already owns 5.4 MGD in water rights from the Caney River, and there is no need for additional water rights at this time. Currently, there is no additional state water rights requirements pertaining to the use of reclaimed water.

8.2 LEGAL AND INSTITUTIONAL REQUIREMENTS

The proposed project will be subject to the Oklahoma Department of Environmental Quality (DEQ) regulations and standards. The proposed project design will adhere to the DEQ standards to obtain the necessary permit to construct.

Local approval from the county commissioners will be required for the proposed pipeline along the section line county roads. Easements may be necessary from local landowners depending on the final alignment of the pipeline. A stream crossing permit from the US Army Corps of Engineers along with a railroad bore crossing permit from Watco Companies will be required for the project. These permits will be obtained as part of the design efforts in 2019.

8.3 MULTI-JURISDICTIONAL AGREEMENT

The proposed project does not involve or require multi-jurisdictional agreements or approval.

8.4 PERMITTING PROCEDURES REQUIRED FOR THE PROJECT IMPLEMENTATION

The following permits will be required for the proposed project implementation.

- OPDES permit from DEQ for the new discharge location for the proposed reclaimed effluent discharge. Bartlesville has obtained technical approval from DEQ and a formal permit process will be completed in early 2019.
- Railroad and county road crossing permits will be obtained as part of the design phase in 2019.
- Depending on the findings from the EA investigation during the design phase, Nationwide General Permit (CWA Section 404) may be required if the proposed alignment interferes with wetlands.

8.5 DISCUSSION OF ANY UNRESOLVED ISSUES

Preliminary pipeline alignment has been established for the benefit of this feasibility report. As part of the design phase in 2019, a more detailed EA will be necessary along the pipeline alignment to resolve any potential impacts to endangered species or wetlands. There are no other significant unresolved issues.

8.6 CURRENT AND FUTURE DISCHARGE REQUIREMENTS RESULTING FROM THE PROPOSED PROJECT

None.

8.7 DESCRIPTION OF RIGHTS TO DISCHARGE RESULTING FROM THE IMPLEMENTATION OF THE PROPOSED PROJECT

This does not apply to the proposed project.

9.0 PROJECT FUNDING PLAN

In 2016, the City of Bartlesville implemented a 5-year step increase to the utility rates along with a capital investment fee as shown by Ordinance 3468 in Appendix F. The purpose of the rate adjustments and capital fee were to fund the anticipated capital improvements and debt service to the water system, including the Title XVI alternate, water reuse. The City plans to finance the water reuse improvements through a loan from the Oklahoma Water Resources Board (OWRB). The rate structure and step increases implemented in 2016 are sufficient to finance the project through the OWRB. The City is in the process of coordinating the financing options through the OWRB and will finalize those decisions once the engineering design for the improvements are completed.

In addition to the utility rates currently in place, the City has been awarded a $750,000 grant through the Bureau of Reclamations WaterSMART Drought Response Program: Drought Resiliency Projects for Fiscal Year 2017-DRP-018 for the construction of the Title XVI alternate, water reuse

Appendix A

Bartlesville Water Supply and Conveyance Study

Planning Assistance to States Program

December 2007

U.S. Army Corps of Engineers

Appendix B

Caney River QUAL2K Scoping Model Near Bartlesville, Oklahoma

August 2016

Tetra Tech

Appendix C

Monitoring Study Plan, Caney River TMDL Study

July 2017

Tetra Tech

Appendix D

Bartlesville WLA Studies – Caney River Monitoring and Modeling Report

November 2018

Tetra Tech

Appendix E

Letters of Local Support

Appendix F

Funding Mechanism

Copy of City of Bartlesville Ordinance 3468

Appendix A

Bartlesville Water Supply and Conveyance Study

Planning Assistance to States Program

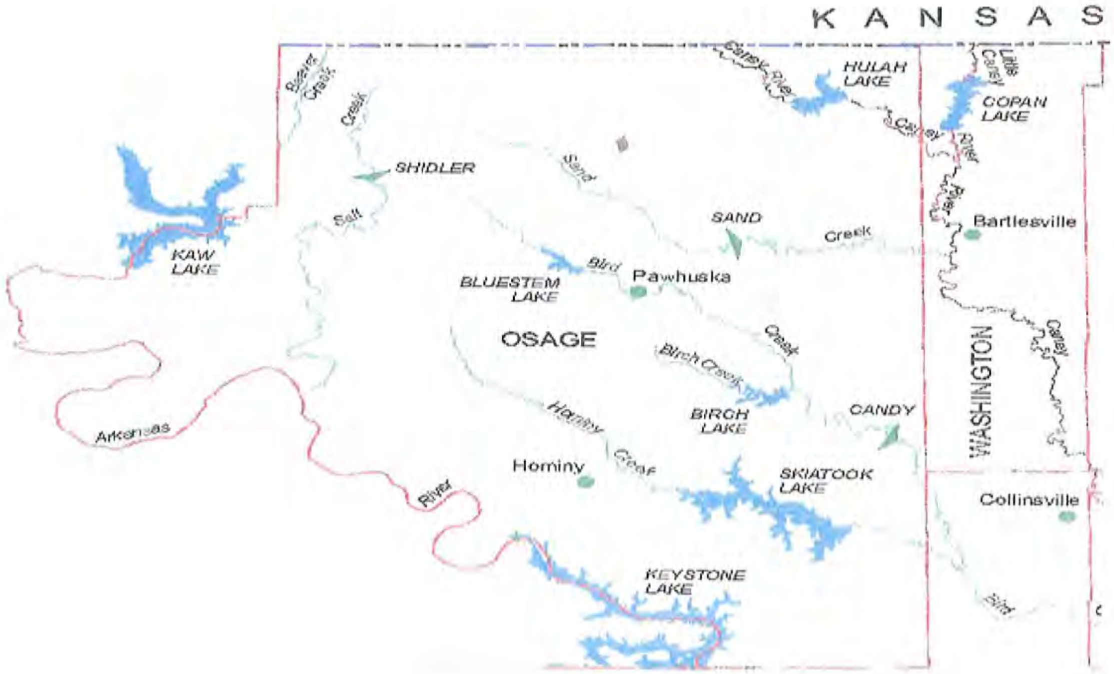
December 2007

U.S. Army Corps of Engineers

2007091

BARTLESVILLE WATER SUPPLY AND CONVEYANCE STUDY

PLANNING ASSISTANCE TO STATES PROGRAM



U.S. Army Corps of Engineers
December 2007

BARTLESVILLE WATER SUPPLY AND CONVEYANCE STUDY

PLANNING ASSISTANCE TO STATES PROGRAM

Prepared For
The Oklahoma Water Resources Board
and
The City of Bartlesville, Oklahoma

By

U.S. Army Corps of Engineers
Tulsa District
1645 S. 101st E. Ave.
Tulsa, OK 74128

December 2007

BARTLESVILLE WATER SUPPLY STUDY

EXECUTIVE SUMMARY

The Bartlesville Water Supply Study evaluated alternatives for future water supply for the City of Bartlesville and most of Washington County, Oklahoma. This study was conducted by the Corps of Engineers in partnership with the City of Bartlesville and the Oklahoma Water Resources Board (OWRB) under the Planning Assistance to States Program, Section 22 of the Water Resources Development Act of 1974, as amended.

The study area includes the City of Bartlesville, Oklahoma, and the communities and rural water districts it currently serves, essentially all of Washington County, Oklahoma. The city currently utilizes Hulah Lake as its primary water supply source. The city remains concerned about the dependability of Hulah Lake. The severe drought of 2001-2002 has caused the city to evaluate the dependability of having only one primary source of water supply, Hulah Lake, in the Caney River basin. In addition, the city believes that recent industrial growth and population increases indicate a growth potential that is not necessarily reflected in the historic trends for the city and Washington County.

Phase I of this study evaluated the current and projected water demand of the study area in relationship to the existing water supply through the study period of 2005 to 2055. Phase I found that water demand could exceed the current supply as early as 2015 and that demand for water could exceed supply by 10.45 million gallons per day (mgd) by year 2055. At that time the projected demand is expected to be 14.8 mgd. Based on the available existing water supply the estimated net water needs are 10.45 mgd, which is the basis for screening alternatives for additional water supply.

Phase II of the study focused on three primary alternatives: (1) reallocation of flood control storage at Copan and Hulah Lakes to water supply; (2) non-Federal development of Sand Lake, a proposed reservoir on Sand Creek in Osage County, Oklahoma; (3) use of Kaw Lake water supply storage and development of a pipeline to the city's Hudson Lake. The study also evaluated measures to preserve and protect Hulah and Copan Lakes.

The study evaluated the use of water quality storage reallocated as a result of the 2006 Reallocation Report and found that use of the reallocated water quality storage could defer the city's water supply problems by as much as 30 years (from 2015 to about 2045), but does not completely solve their long term needs through the entire study period.

The study evaluated several options for reallocation of storage at Hulah and Copan Lakes from flood control to water supply. The study evaluated both the quantity of storage that could be made available, the projected yield associated with that storage, and also did a brief evaluation of the potential impacts to the flood control operations of the two lakes, both of which reduce flooding in the city of Bartlesville and along the Caney River. The alternatives evaluated reallocation of between 1 percent and 10 percent of the flood control storage at the two lakes, which resulted in water supply yields estimated from about 9 mgd to about 25 mgd. The impact to the areas downstream of the lakes was assessed and the study found that reduction of the flood control storage at the lakes was not significantly different between the alternatives. The additional total flood damages lost downstream ranged from about $176,000 to $222,000 over the 50 year study period, with average annual damages increasing by $9,000 to $12,000. The study also assessed impacts to the areas upstream of the Lakes Hulah and Copan Dams; the impacts were primarily to recreational facilities and cultural and natural resources. The study found that the costs for mitigation and replacement of loss of habitat and facilities were about $600,000 to $8.6 million.

The study also evaluated the costs associated with construction of Sand Lake, a potential non-Federal water supply lake on Sand Creek in Osage County, Oklahoma. The study found that construction of Sand Lake could provide sufficient water supply to meet long term needs for the city of Bartlesville in combination with the city's existing Lake Hulah contracts. The study analyzed the costs associated with construction of Sand Lake and found that these costs are about $86.0 million.. The costs associated with potential environmental and cultural impacts of constructing the dam and pipeline and costs to address mineral rights as a result of construction of the dam were not evaluated, but could be significant.

The study also evaluated use of storage at Kaw Lake and the costs associated with construction of a pipeline from Kaw Lake to Hudson Lake. The study found that sufficient water supply storage is available at Kaw Lake to meet long term needs for the City of Bartlesville. However, the cost for storage at Kaw Lake and pipeline water conveyance costs is estimated at $105.0 million. The costs associated with environmental or cultural impacts of construction of the pipeline were not addressed.

The study also addressed opportunities to protect and extend the lives of Hulah and Copan Lakes by managing the areas upstream of the reservoir to limit sediment and nutrient loading in the lakes. The study found both sediment and nutrient loading were on-going at the historic rate.

Based on these study findings, the most economical method for the city to provide for its future water supply demands is to utilize the existing sources of water supply at Hulah and Copan Lakes with flood pool reallocation when the storage is required.

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LIST OF APPENDICES

BARTLESVILLE, OKLAHOMA
WATER SUPPLY ALTERNATIVES

1. INTRODUCTION

The U.S. Army Corps of Engineers (COE), Tulsa District, conducted this study for the Oklahoma Water Resources Board and the City of Bartlesville, Oklahoma, under the authority of Section 22 of the Water Resources Development Act of 1974, the Planning Assistance to States (PAS) Program. The study explores alternatives for supplying water for the population of Bartlesville and Washington County. Based on current projections, the city's existing water supply sources will not meet water supply demands through year 2055 for the City of Bartlesville and the area for which the city supplies water. The City currently gets 100% of their water supply from Hulah Lake, a Corps of Engineers Lake on the Caney River in Oklahoma.

Based on previously expressed needs by the City of Bartlesville, the U.S. Army Corps of Engineers evaluated alternatives to reallocate storage at Hulah Lake and at Copan Lake, also a Corps reservoir in the Caney River basin, in a report dated April 2006. The recommended alternatives would provide an additional 7.2 million gallons per day (mgd) from Hulah Lake and an additional 5.54 mgd from Copan Lake. The report states that this reallocation would provide the City of Bartlesville sufficient water supply through year 2035.

At the time the reallocation report was submitted for approval, Bartlesville officials became concerned that population projections used in the reallocation report may have been underestimated. Population growth has increased the last few years with the influx of new and expanding businesses; that growth is not captured in historic trends used for the reallocation report and was not considered during the reallocation study period. This PAS study is in response to the potential revised higher demand for water. A longer study period through year 2055 was also evaluated. The goal of the study is to provide information to the City of Bartlesville in order that they can make important strategic decisions regarding a dependable, cost efficient high quality water supply for the 21st century for citizens of Bartlesville and Washington County, Oklahoma.

The study included gathering existing water system information, evaluating existing facilities, formulating alternatives in cooperation with the Bartlesville Water Resource committee, and based on the future needs that were supplied by Mike Hall, the Water Utilities director for the City of Bartlesville. The study also included a preliminary analysis of potential environmental and cultural resources issues, flood benefits lost and related implementation costs for each water supply alternative being considered.

2. STUDY AUTHORITY

The U.S. Army Corps of Engineers, Tulsa District, (Corps) conducted the study for the Oklahoma Water Resources Board and the City of Bartlesville, Oklahoma, under authority of Section 22 of the Water Resources Development Act of 1974 (Public Law 93-251), also known as the Planning Assistance to States Program. This authority establishes cooperative assistance to states for preparation of comprehensive water plans.

Section 319 of the Water Resources Development Act of 1990 (Public Law 101-640) provides authority for cost sharing of the Planning Assistance to States Program. The cost-sharing ration for this study is 50% Federal and 50% non-Federal. A Letter Agreement between the COE, Tulsa District and the City of Bartlesville, Oklahoma, was signed on April 7th, 2006. The Letter Agreement is included as Appendix 1.

3. PURPOSE & SCOPE

This study was conducted to identify long term water supply solutions for the City of Bartlesville, Oklahoma. This study is a two-part study. Phase I evaluated water demand through year 2055 for Bartlesville and Washington County. Phase II, initiated after future water supply demand was identified, evaluated water supply alternatives to meet the identified demand. Primary water supply alternatives considered include Kaw Reservoir, a previously authorized Federal reservoir site located in Osage County (Sand Lake), and flood control reallocation alternatives at Hulah and Copan lakes.

Bartlesville is proactively planning for long term availability of its water supply. The city is currently using water from Hulah Lake, which is its sole source of available water. City officials are concerned that Hulah may be insufficient as a sole water supply source and they are also concerned about Hulah Lake dependability. Hulah Lake was built in 1951 and provides a relatively inexpensive source of water to Bartlesville. However, sediment inflows continue to reduce available water supply storage both now and in the future. The 2006 Hulah and Copan reallocation report identified water supply options that Bartlesville could execute, but city officials are concerned that those options may not provide a sufficient water supply yield beyond year 2035. The city is exploring other water supply alternatives in the event that Hulah Lake as it exists today will be insufficient as the sole water supply source for its future.,.

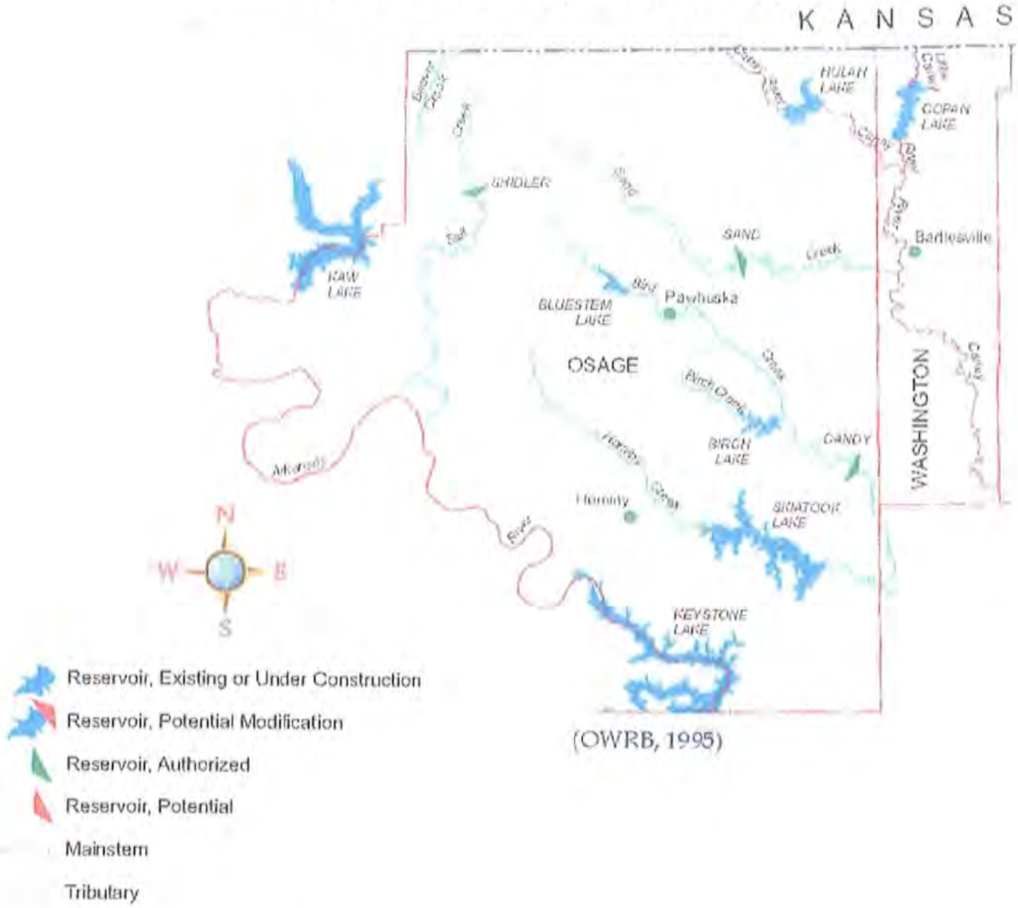
4. PROJECT LOCATION AND DESCRIPTION

The study area includes the City of Bartlesville, Washington County, and Osage County in Northeast Oklahoma. The City of Bartlesville lies in about the middle of the Caney River watershed. The Caney River, approximately 155 miles in length, rises in Elk County, Kansas, and flows in a southerly and southeasterly direction to enter the Verdigris River at river mile 78.3 in Rogers County, Oklahoma. The basin contains 2,111 square miles of drainage area, and has a channel capacity estimated to be 11,000 cfs at the mouth of the stream.

The major tributaries of the Caney River are Caney and Sand Creeks. Caney Creek, which is about 60 miles long and has a drainage area of 516 square miles, flows into the Caney River at river mile 80.5 at a point about 10 miles north of Bartlesville, Oklahoma. The channel capacity of the creek is about 5,000 cfs. Sand Creek flows into the Caney River at river mile 63.7 about 3 miles south of Bartlesville. The creek is approximately 50 miles long, with a drainage area of 240 square miles and a channel capacity of 5,500 cfs.

The study area is shown in Figure 1.

FIGURE 1 CANEY RIVER AREA WATERSHED



There have been three previously authorized Corps lakes in the Caney River Basin in Oklahoma; Hulah, Copan, and Sand Lakes. Hulah and Copan have been constructed and are in operation. Sand Lake was not constructed and was deauthorized in 1999. Hulah Lake is located on the Caney River at river mile 96.2 and controls 732 square miles of drainage area. Copan Lake is located at river mile 7.4 on the Little Caney River and controls 505 square miles of drainage area. The previously authorized Sand Lake site is located at river mile 10.1 on Sand Creek and would control approximately 187 square miles of drainage area.

5. PRIOR STUDIES AND REPORTS

Previous congressionally authorized studies pertinent to the Bartlesville Water Supply and Conveyance study are:

a. Caney River Basin, Verdigris River and Tributaries, Kansas and Oklahoma, Reconnaissance Report, September 1984. This study was designed and conducted to identify priority water resource problems in the basin and to conduct preliminary evaluations of alternatives to determine if economically feasible projects appeared to be available. This study identified water supply needs of Bartlesville as the major water resource problem in the basin.

b. Survey Report on Verdigris River and Tributaries, Oklahoma and Kansas, December 1961. Included in this report were design memorandum guidelines for Sand Lake authorization and also a summary of cost and quantity estimates. This report provided preliminary cost estimates for Federal construction of Sand Lake.

c. Hulah and Copan Lakes, Oklahoma Water Supply Reallocation Report and Water Supply Agreements and Final Environmental Assessment (EA), April 26, 2006. The Hulah Copan reallocation report reallocated water quality to water supply storage at Hulah and Copan Lakes. The April 2006 reallocation report identified an additional yield of 6.4 mgd which could be obtained from water quality storage through year 2035 from both reservoirs. The 2006 reallocation study relied on information obtained from a TetraTech, FHC report "Cost of Alternative Water Supply Sources dated August 2004. This report looked at the Cost of Alternative Water Supply Sources from Federal, State and NRCS lakes throughout Northeast and North Central Oklahoma.

d. Wholesale Water Treatment and Conveyance Study Kaw Lake Area, Oklahoma, June 2002. The study initiated data collection for three alternate plans that would provide a regional wholesale water treatment and conveyance system serving 30 communities and rural water systems in 13 counties of northern and central Oklahoma.

6. BACKGROUND

Since the early 1940's previous Bartlesville and State officials worked with the Corps of Engineers to build Hulah Lake as a federal water resource project that has provided numerous benefits to north central Oklahoma and the Bartlesville and Washington County Communities. Construction of Hulah Lake in 1951 provided a new source of water but it also provided flood reduction to the Bartlesville community that was greatly needed. Flood reduction benefits from Hulah greatly reduced downstream flooding within the Bartlesville community.

Shortly after Hulah was completed, in 1957, Bartlesville signed a water supply storage contract for 15,400 acre-feet (9.6 mgd) . Hulah Lake has provided Bartlesville with a reliable inexpensive dependable water supply yield for many years. Smaller water supply storage agreements were also signed in 1970 for 2,200 acre-feet (1.4 mgd) and 1980 for 2,100 acre-feet (1.3 mgd). Hulah Lake is Bartlesville's current sole water supply source.

In 1962, additional planning was started for future reservoirs to be built in north central Oklahoma. Sand Lake and Copan Lake were both authorized and studied for possible future construction. Copan Lake was built and became operational in 1983 and has the project purposes of flood control, water supply, water quality, recreation, and fish and wildlife. Copan provides additional flood protection to Bartlesville. Sand Lake was never built and was deauthorized in 1999.

Recently, Bartlesville's water treatment system has seen an expansion of its service area to a large percentage of communities surrounding Washington County. In 2001-2002, the region experienced a short but severe drought in the upper Caney River watershed which impacted

Hulah Lake. The 2001 drought created a question about available long term water supply especially during drought conditions. Bartlesville, the primary supplier in the area, was strained even more as surrounding communities, forced to meet more stringent water quality standards, increasingly relied on the city for water supply.

Shortly after the 2001 drought, a reallocation study was initiated to evaluate the impacts of reallocation from water quality and flood control storage to water supply at Hulah and Copan reservoirs. The reallocation report was submitted to Corps higher authority for approval in April 2006. The reallocation report identified an opportunity to reallocate from water quality storage to water supply storage and which would yield an additional 6.4 mgd. The report was approved in September of 2007 and can now be implemented by the City of Bartlesville, if the city so desires.

In 2005, Bartlesville officials requested an additional water supply evaluation through the PAS program. The city requested that the study evaluate water supply demand through 2055 and further review potential water supply alternatives. The PAS study, included two phases: Phase I analyzed the future water supply demand for Bartlesville and Washington County. Phase II then analyzed potential supply alternatives based on the identified demand.

7. STUDY SCOPE

Phase I of this study examined future water supply and water demand for the next fifty years (through 2055), and compared them to existing sources to determine Bartlesville's Net Water Needs. A collaborative effort with the City of Bartlesville was used to select the appropriate population and demand projections and, there from, the Net Water Needs before the second phase of the study. Demographic and economic variables, such as population, employment by industry, housing density, and median household income were used as a basis for projecting future water needs.

Phase II evaluated water supply yield projections for existing reservoirs and other water supply alternatives through 2055, and evaluated sediment conservation measures that could preserve existing water supply storage. Primary water supply alternatives being considered include Kaw Reservoir, previously authorized Sand Lake in Osage County, and Flood Pool reallocation alternatives at Hulah and Copan. Environmental impacts, regulatory compliance, loss of existing flood protection, availability of sufficient water supply sources to meet projected demand, and the effect that river basin size has on the dependable yield were other planning constraints that were considered. Land and legal issues for private and federal funding for new water supply reservoir alternatives was also a major constraint. Phase II also looked at potential conservation measures that addressed upstream sedimentation at Hulah and Copan Lakes and a sensitivity analysis of the future Bartlesville's existing water supply. Reliability of the water source is also an important factor to the community as well.

8. PHASE I – NET NEEDS ANALYSIS

a. Phase I

(1) Introduction Phase I of the two-phase study effort was completed in March 2007. This first phase determined future net water needs for the City of Bartlesville and the surrounding communities, rural water systems, and other areas to which the City provides water. The first phase includes an estimate of future demand for water based on different population growth scenarios Washington County could experience from 2005 to 2055, with year 2005 representing the base year. The City of Bartlesville expects population growth in the city and in Washington County to occur at a much faster rate than historic growth rates indicate. Population forecast scenarios were made for the City of Bartlesville, two rural water districts the City supplies, and Washington County. The City supplies water to approximately 99% of the residents in Washington County. Since nearly all of the water demands in Washington County are supplied by the City of Bartlesville, forecasts used for the purpose of this report are based on Washington County data.

(2) Water Demand. Estimates of the quantities of water needed in the future require the use of appropriate econometric models. These models are used to project future water use that is statistically consistent with long-term water supply planning. In order to forecast Municipal & Industrial (M&I) water demand Institute for Water Resources-Municipal and Industrial Needs (IWR-MAIN) Water Demand Management Suite, a Windows based PC software package, was used to translate existing population, housing, and employment into estimates of existing water demands for the 2005 base year. Actual water use data for year 2005 for the City of Bartlesville and included rural water districts in Washington County and the City of Dewey. Some of the included rural water districts may overlap into neighboring counties.

(3) Projection Scenarios. Three water demand scenarios were presented to the Water Resource Committee of Bartlesville. The Baseline Projection Scenario is based on historical growth and weather pattern trends experienced in the study area. Due to the fact that the population of Washington County has not increased significantly over the past ten years, the baseline water demand forecasts have not deviated from the base year by a substantial amount. The baseline projection is based on a 2055 population of 53,000 in Washington County.

The City of Bartlesville provided information on actual water use for the base year 2005. This information was disaggregated into different sectors of water use such as residential, municipal, industrial, commercial, water districts, and public schools. In addition, the City also provided information on population and housing projections from years 2000-2050. This data was then used to develop a high growth scenario for the water system that Bartlesville supplies. The City developed these growth projections based on the current level and pace of development. The water demand forecast for the high growth projection was based on a 2055 population of 73,000 and developed by the Tulsa District using the IWR-MAIN forecast system. The Oklahoma Department of Commerce (ODC) provided the demographic data of population estimates as the basis for employment and housing projections. Other sources of information include, the U.S. Department of Labor, the U.S. Census, and the Oklahoma State Climate Center, and the National Weather Service.

A third projection, called the mid projection, was interpolated from the baseline and high projections. Water demand forecasts for the mid projection population were not evaluated using IWR-MAIN but are derived as an average of the baseline and high-growth projections.

(4) Methodology. The method that was selected for forecasting residential water demand uses median household income, persons per household, housing density, marginal price of water, maximum temperature, and precipitation, to adjust per unit usage rates for residential information, but not for non-residential variables. For the non-residential sector, a model for water demand was customized using values for intercept terms, model variables, and associated coefficients and elasticities. The base year per unit water use rate is calculated from the base year water use and the number of counting units for the sub-sector. This calculated rate of use is then adjusted by the relationship between sub-sector water use and those explanatory variable selected for the sub-sector, which are selected by the user and may change over time. Year-to-year changes in water use are explained by the change in the selected explanatory variables and the counting units. Counting units derived from population projections, are the driver variables, such as employee counts, housing units, acres, etc., associated with each sub-sector.

(5) Peak Demand. Another output IWR-Main can forecast is peak water demand. Peak use for a community can vary month to month depending mainly on temperature and rainfall. Typically record peak use will occur in the hottest summer months, because this is a period where water demand significantly increases as homeowners are watering their lawns and gardens more frequently and precipitation rates are low. The system peak use may be specified in gallons per day, thousand gallons per day, or million gallons per day. The user must select the month in which the base system peak occurs and enter the peak use value. For this study, the City of Bartlesville supplied the peak use in million gallons per day which occurred in the month of July.

(6) Results. On March 1, 2007, the City agreed to proceed with the water demand projections based on the mid and high population growth projections ranging from 63,000 to 73,000 by 2055, which equates to water demand in 2055 being 12.8 to 14.8 million gallons per day (mgd). Due to the uncertainty of both demand and supply 50 years in the future, a range of net needs was determined to estimate future water supply needs.

b. **Existing Water Supply**

(1) Introduction. Hulah and Copan Lakes provide the majority of the water supply to Washington County. Bartlesville obtains its water from Hulah Lake, which is then pumped to city-owned Hudson Lake prior to treatment. During periods of insufficient supply from Hulah Lake and Hudson Lake, water can be pumped from the Caney River under emergency conditions. An additional 2.0 mgd from Copan Lake is utilized by the Copan Public Works Authority in Washington County.

(2) Hulah Lake. Hulah Lake construction started in May 1946, and was completed in February 1951 for flood control, water supply, low flow regulation, and conservation purposes. Embankment closure began in February 1950 and was completed in June 1950.

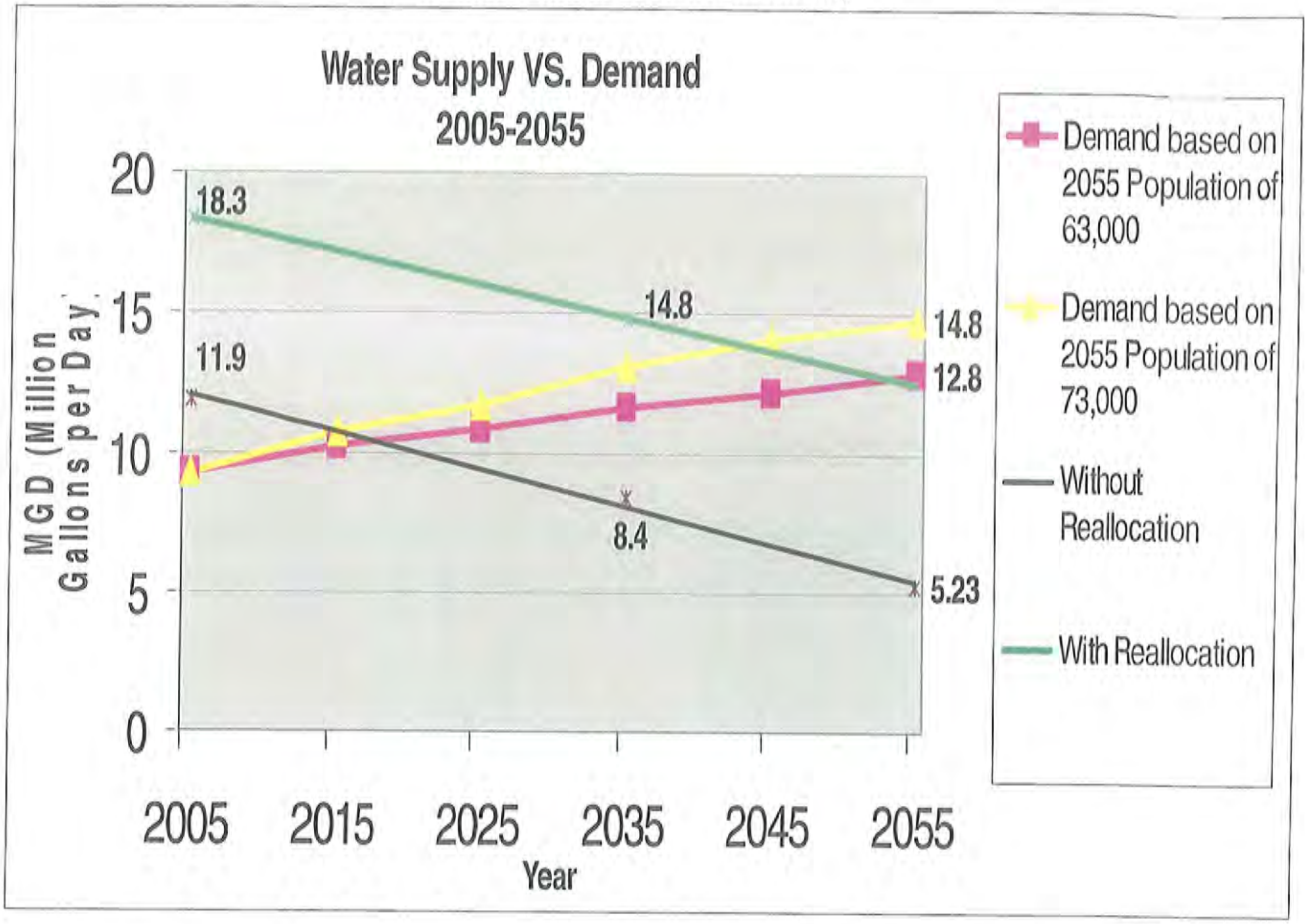
Impoundment of the conservation pool began on September 23, 1951, and was completed on September 24, 1951. The project was placed in full flood control operation in September 1951. Hulah Lake currently has 19,800 acre-feet of original water supply storage , all of which is under contract, which yields 6.4 million gallons per day (mgd) through year 2035.

(3) Copan Lake. Copan Lake construction began in November 1972, and the project was placed in useful operation in April 1983. Copan Lake provides flood control benefits to Bartlesville and is a second close water supply alternative that Bartlesville is considering. The water supply yield of Copan Lake is 7,500 acre-feet with a 3.0 mgd yield. Copan currently has one (1) million gallons per day (mgd) of available water supply not currently under contract.

(4) Dependable Yield. The Corps 2006 Hulah and Copan Reallocation study evaluated both the current (2005) and long term (2035) dependable yield of the two Lakes, including the impacts of sedimentation on available reservoir storage. The evaluation indicated that the City of Bartlesville has 6.4 mgd of dependable yield from Hulah Lake through year 2035 using historical data for the 50 year drought of record and the latest 2002 sediment survey for Hulah Lake. That data was utilized to project the dependable yield of Hulah Lake through 2055 assuming no measurable reduction in the rate of sediment deposition, and the analysis indicated that the dependable yield at Hulah Lake is projected to decline from 6.4 mgd in year 2035 to 4.35 mgd by year 2055. The same evaluation was done for Copan Lake and the findings indicated 0.97 mgd which is available through 2035 will decline to 0.88 mgd by year 2055. Based on that evaluation, a total of 7.35 mgd is currently available from Copan and Hulah lakes through 2035. The analysis indicated that a total available water supply yield from Hulah and Copan Lakes is 5.23 mgd in year 2055.

The 2006 Reallocation Report identified 1,230 acre-feet of additional storage in Hulah Lake and 12,490 acre-feet of additional storage in Copan Lake that is available for water supply purposes. The Hulah Lake yield is currently estimated to be 6.4 mgd and the Copan Lake yield is estimated to be 0.97 mgd. If the city elects to utilize both of their existing water supply contracts, the currently available water supply storage from Copan Lake, and the reallocated storage available from Hulah and Copan Lakes, the current yield would be 12.74 mgd in 2035. That yield is further projected to decline to 6.85 mgd by 2055. Figure 2 shows a graphical representation of the supply versus demand data discussed above for the demand based on population projections of 63,000 (mid) and 73,000 (high) and supply based on with and without the 2006 recommended reallocation.

Figure 2
WASHINGTON COUNTY
Supply versus Demand
(With and Without 2006 Reallocation)



c. **Net Water Needs**

Net water needs for Washington County are based on existing water supply for the City of Bartlesville from Hulah Lake and the water supply from. Copan Lake used by the Copan Public Works Authority. Based on the projections developed for this report, it appears that water demand will exceed available supply in about 2015 for both the mid and high population projections if the city does not utilize the reallocated storage at Hulah and Copan Lakes. If the city dues utilize that storage, projections indicate that the city will have sufficient water supply through 2050 for the mid projection and through 2045 for the high projection. Based on a population projection of 73,000 (high) and the existing water supply storage without the 2006 reallocation, the water demand will be 14.8 mgd and the supply from Hulah Lake will be 4.35 mgd. The net need utilized for the identification and formulation of alternatives in the second phase of this study is, therefore, 10.45 mgd.

9. PHASE II WATER SUPPLY ALTERNATIVE ANALYSIS

a. **Phase II Water Supply Lakes Evaluated**. Phase I identified a maximum net need of 10.45 mgd in 2055 for the study area. Although that net need represents the total Washington County need, including the City of Copan's supply and demand, it is reasonable and conservative to select 10.45 mgd as the net need for the city of Bartlesville and its service area. After evaluating the available alternatives and their potential to meet that need, the study team and the Bartlesville water resource committee members identified Hulah, Copan, Kaw and Sand Lakes as water supply alternatives to be studied in Phase II. A summary of each alternative reservoir's features including a comparison of each hydrologic basin characteristics is described below.

(1) Hulah Reservoir. Hulah Reservoir construction started in May 1946, and was completed in February 1951 for flood control, water supply, low flow regulation, and conservation purposes. Embankment closure began in February 1950 and was completed in June 1950. Impoundment of the conservation pool began on September 23, 1951, and was completed on September 24, 1951. The project was placed in full flood control operation in September 1951.

Table 1 outlines pertinent data for Hulah Lake. Lake data is based on the latest 2002 sedimentation survey.

Table 1. Hulah Lake Data

Feature	Elevation (feet)	Area (acres)	Capacity (acre-feet)	Equivalent Runoff[1] (inches)
Top of Dam	779.5	-	-	-
Top of Flood Control Pool	765	13,000	289,000	7.40
Flood Control Storage	733.0-765.0	-	257,900	6.61
Spillway Crest	740.0	5,160	61,400	1.57
Top of Conservation Pool	733.0	3,120	22,565[2]	0.80
Conservation Storage	710.0-733.0	-	22,553	0.80
Top of Inactive Pool	710.0	0	12	-

[1] From a 732-square-mile drainage area above the dam site.

[2] Includes 16,600 acre-feet for water supply, 5,953 acre-feet for water quality control, and 12 acre-feet for sediment reserve.

(2) Copan Reservoir. Copan reservoir construction began in November 1972, and the project was placed in useful operation in April 1983. Copan Reservoir provides flood control benefits to Bartlesville and is a second close water supply alternative that Bartlesville is considering. Copan currently has one million gallons per day (mgd) of available water supply and a reallocation of water quality storage to water supply was recommended by the Tulsa District and approved by the United States Army Corps of Engineers Headquarters (USACEHQ) in September 2007.

Table 2 displays pertinent data for Copan Reservoir.

Table 2. Copan Lake Data				
Feature	Elevation (feet)	Area (acres)	Capacity (acre-feet)	Equivalent Runoff[1] (inches)
Top of Dam	745.0			
Maximum Pool	739.1	17,850	338,200	12.57
Top of Flood control Pool	732.0	13,380	227,700	8.45
Flood Control Storage	710.0-732.0		184,300	6.84
Top of Conservation Pool	710.0	4,449	34,634	1.61
Conservation Storage	687.5-710.0		33,887[2]	1.59
Spillway Crest	696.5	1,080	4,700	0.17
Top of Inactive Pool	687.5	110	747	0.02
[1] Drainage area is 505 square miles.				
[2] Includes 7,500 acre-feet for water supply (3.0 mgd yield), 26,100 acre-feet for water quality control (16 mgd yield), and 9,200 acre-feet for sediment based on 1983 survey..(In year 2002, useable storage=34,634acre-feet less 747 acre-feet)				

(3) Kaw Reservoir. Kaw Reservoir is located on the Arkansas River at river mile 653.7, about 8 miles east of Ponca City in Kay County, Oklahoma. Kaw Reservoir is about 45 miles from Lake Hudson. Its purpose is flood control, water supply, water quality, hydropower, recreation, and fish and wildlife. Construction began in June 1966 and the project was place into operation in May 1977. Based on a 1986 sedimentation survey, the conservation storage is estimated at 330,180 acre-feet. Flood control storage is 867,310 acre-feet. The power and conservation storage has a capacity of 383,480 acre-feet, and includes 171,200 acre-feet for water supply (167 mgd yield), 31,800 acre-feet for water quality control (39 mgd yield), and 140,500 acre-feet for sediment reserve. Table 8 below outlines Kaw Lake Data.

Table 3. Kaw Lake Data				
Feature	Elevation (feet)	Area (acres)	Capacity (acre-feet)	Equivalent Runoff[1] (inches)
Top of Dam	1065.5	-	-	-
Top of Flood Control Pool	1044.5	39,650	1,327,160	3.74
Flood Control Storage	1010.0-1044.5	-	920,610	2.45
Spillway Crest	997.5	11,070	234,167	0.66
Top of Conservation Pool	1010.0	16,750	406,540	1.15
Conservation Storage	978.0-1010.0	-	330,180	1.08
Top of Inactive Pool	978.0	5,240	76,360	0.22
[1] Contributing drainage area above the dam site is 6,652 square miles. The spillway design drainage area is 8,975 square miles. The total drainage area is 46,530 square miles.				

(4) Sand Lake Reservoir. The proposed Sand Lake reservoir was authorized as a Federal Multiple purpose reservoir in 1962 but was never constructed and was deauthorized in 1999. In addition, the project, as authorized, no longer meets Federal criteria for development due to insufficient flood control benefits. This study evaluated Sand Lake as a non federal water supply only lake, including development of cost estimates. Table 4 below outlines Sand Lake Data as it was originally proposed for Federal Authorization.

Table 4. Sand Lake (as Proposed for Federal Authorization)				
Feature	Elevation (feet)	Area (acres)	Capacity (acre-feet)	Equivalent Runoff (inches)
Top of Dam	808.0	-	-	-
Top of Flood Control Pool	786.0	3,520	51,700	N/A
Spillway Crest	786.0	3,520	51,700	N/A
Top of Conservation Pool[1]	766.5	1,940	35,000	N/A
Conservation Storage	734.0-766.5	1,940	35,000	N/A
Top of Inactive Pool	734.0	-	4,300	N/A

[1] ***Contributing drainage area above the dam site is 137 square miles. Water Supply Yield is 12.0 mgd***

(5) Hydrologic Basin Comparison of Hulah, Copan, Kaw and Sand Lake. Although there are many factors which influence the dependable yield of a reservoir, the two most significant to the four lakes under evaluation are drainage area above the reservoir and the amount of water supply storage in the reservoir. Drainage basin comparisons between the four lakes shows that Kaw Reservoir has by far the most drainage area of the four hydrologic basins with 6,652 square miles. The Hulah Lake drainage area is 732 square miles, the Copan Lake drainage basin is 505 square miles, and the deauthorized Sand Lake would have had a drainage area of 137 square miles. Kaw Lake also has significantly more water supply storage than the other 3 lakes, with 330,180 acre-feet of storage. The originally authorized water supply storage of Hulah Lake is 16,000, Copan Lake has 7,500 acre-feet, and Sand Lake would have had 35,000 acre-feet of water supply storage. Comparison of the four hydrologic basins indicates that Kaw reservoir has the most available water supply of the four basins, and, because of the size of the drainage area above it, it would be the least likely to be impacted by long term localized droughts. That is particularly significant to the city of Bartlesville because of the drought the Hulah Lake drainage area experienced in 2001-2002.

b. Phase II Alternative Study Criteria. Dependable water supply yield, drought frequency effect on dependable water supply yield, impacts to flood control, and cost were the primary study criteria that were analyzed for each alternative in Phase II. Phase II also included a sensitivity analysis to evaluate the effects on dependable water supply yields if the input model parameters of inflows and sediment are modified. Upstream conservation measures were also looked at at Hulah and Copan Lakes.

c. Water Supply Alternatives. After a review of a variety of options, the study team and the Bartlesville Water Committee developed a set of five alternatives. These five alternatives represent approaches that have the highest potential to most efficiently meet the city's future water demand. Alternative #1, the No Action alternative looked at yields assuming no change in existing water supply sources. Alternative #2 studied 2055 water supply storage assuming Bartlesville initiates new water supply storage agreements recommended in the April 2006 Hulah-Copan reallocation study. Alternative #3 evaluated potential reallocation of flood control storage at Hulah and Copan Lakes. Alternative #4 studied Sand Lake as a privately developed water supply alternative. Alternative #5 studied Kaw Lake as a potential long term water supply alternative.

(1) Alternative 1: No Action Alternative - Maintain Hulah Reservoir as its sole water supply Source. This alternative evaluated the dependable yield in Hulah Lake in 2055.

(2) Alternative 2: Implement new water supply agreements proposed in April 2006 Water Supply Storage Reallocation Study at Hulah and Copan Lakes Oklahoma. This alternative evaluated the water supply yield availability through year 2055 assuming that new water supply agreements outlined in the 2006 Hulah and Copan Reallocation study are implemented. The new water supply agreements approved by HQUSACE would provide through year 2035, 1,230 acre-feet (0.82 mgd) of new storage at Hulah Reservoir, 2,185 acre-feet (0.97mgd) of originally authorized water supply at Copan Reservoir, and 10,305 acre feet (4.57 mgd) of new storage at Copan. The three proposed water supply agreements provide 13,720 acre feet of storage to Bartlesville and would provide an additional yield of 6.4 mgd through year 2035 for immediate use.

(3) Alternative 3: Reallocate Flood Pool at Hulah and Copan Reservoirs. This alternative evaluated multiple flood control to water supply reallocation scenarios. Alternative #3 also evaluated potential upstream environmental impact costs and downstream flood control benefits foregone.

(4) Alternative 4: Private Sand Lake Reservoir with pipeline to Hudson Lake. This alternative assumes no Federal Authorization. This alternative evaluated Sand Lake as a non-Federally constructed lake and water supply source. It also included evaluation of costs for construction of a pipeline to the city owned Lake Hudson.

(5) Alternative 5: Purchase water supply storage from Kaw Reservoir with Pipeline to Hudson Lake. This alternative evaluated Kaw Reservoir as a water supply source, including development of costs for constructing a pipeline to Lake Hudson.

d. Evaluation of Alternatives

(1) Alternative #1 - No Action Alternative. The 2006 Hulah and Copan Reallocation study indicated that the City of Bartlesville has 6.4 mgd of dependable yield through year 2035 using historical data for the 50 year drought of record and the latest 2002 sediment survey. Based on the latest 2002 sediment survey, assuming no measurable protection measures are enacted, the dependable yield is projected to decline from 6.4 mgd in year 2035 to 4.35 mgd by year 2055. *Given the water needs assessment of 14.8 mgd projected by IWR Main, an additional 10.45 mgd of new water sources will be required in 2055.*

(2) Alternative #2 - Implement Hulah-Copan Reallocation Report Water Supply Agreements. The April 2006 Reallocation Study and new water supply agreements would provide through year 2035, 1,230 acre-feet (0.82 mgd) of new storage at Hulah Reservoir, 2,185 acre-feet (0.97mgd) of originally authorized water supply at Copan Reservoir, and 10,305 acre feet (4.57 mgd) of new storage that was reallocated from water quality at Copan. The three proposed water supply agreements provided an additional 13,720 acre-feet of storage to Bartlesville and would provide an additional yield of 6.4 mgd through year 2035. Added to their existing contracts for water supply at Hulah Lake, that provides the city with 12.74 mgd through

2035 which is sufficient to meet their needs through 2035. Because of continued sedimentation of the lakes, the water supply storage available will continue to decline as will the water supply yield. The evaluation of yield for 2055 indicates that that yield available at Hulah and Copan Lakes will total 6.85 mgd and will not be sufficient to meet demand projections of 14.8 mgd as projected by IWR Main through year 2055. *Assuming a water demand of 14.8 mgd and a dependable yield of 6.85, Bartlesville will still have a deficiency 7.95 mgd by year 2055.*

(3) Alternative #3 - Reallocate Flood Pool At Hulah And Copan. This option investigated the potential water available from a future reallocation of the flood pool to water supply at Hulah and Copan Reservoirs. Table 10 provides a multiple list of water supply yields that would be available from potential future reallocations from flood control storage to water supply. The yields shown for each alternative include both the yield from originally authorized water supply storage and the yield which would result from the storage identified in the April 2006 Reallocation Report. Any reallocation of storage from flood control, and any associated water supply contracts, would require that the storage reallocated as a result of the April 2006 report be contracted for first, before any additional reallocation could be approved. The alternatives evaluated reallocating some percentage of flood control storage (1%, 2.5%, 5%, and 10%) at either Hulah Lake alone, at Copan Lakes alone, or at both lakes.

The criteria used in selecting alternatives to carry forward was that the total yield had to meet or exceed 14.8 mgd. Alternatives 3A, 3B, 3F, and 3G were rejected because they did not meet the minimum yield required. Alternative E was rejected because it had significantly more yield than was needed. Alternative 3H was rejected because it provided a greater yield than was needed and utilized more flood control storage than did similar Alternative3 I. Alternative 3C provided a total yield of 16.76 mgd while requiring reallocation of 5% of the flood control storage at both Hulah and Copan Lakes. Alternative 3D provided a total yield of 15.07 while requiring reallocation of 10% of the flood control storage at Hulah Lake and no changes at Copan Lake. Alternative 3I provided 16.36 mgd while requiring reallocation of 1% of the flood control storage at Hulah Lake and 10% of the flood control storage at Copan Lake. Each of those 3 alternatives, Alternatives 3C, 3D, and 3I, met the selection criteria and were carried forward for more detailed study.

Table 5
HULAH AND COPAN
REALLOCATION ALTERNATIVES

Alternative Option	Relocations Options Hulah and Copan	Total Available Yield Hulah (mgd)*	Total Available Yield Copan (mgd)*	TOTAL YIELD (mgd)*
3A	2.5% FC Reallocation at Hulah and Copan	6.21	6.88	13.09
3B	5.0% FC Reallocation at Hulah Existing WS at Copan	8.51	0.88	9.39
3C	5.0% FC Reallocation at Hulah and Copan	8.33	8.43	16.76
3D	10.0% FC Reallocation at Hulah Existing WS at Copan	14.19	0.88	15.07
3E	10.0% FC Reallocation at Hulah and Copan	13.75	11.47	25.22
3F	Existing WS at Hulah 10% FC Reallocation at Copan	4.35	5.79	**10.14
3G	2.5% FC Reallocation at Hulah 5% Reallocation at Copan	6.18	8.43	14.61
3H	2.5% FC Reallocation at Hulah 10% FC Reallocation at Copan	6.06	11.47	17.53
3I	1% FC Reallocation at Hulah 10.0% FC Reallocation at Copan	4.89	11.47	16.36

(4) Selected Flood Pool Reallocation Alternatives. Table 6 identifies that Bartlesville could obtain sufficient water supply to meet the projected demand of 14.8 mgd through a reallocation of the flood control pool at Hulah and Copan Reservoirs. Alternatives 3C, 3D, and 3I are summarized in Table 6.

Table 6
REALLOCATION ALTERNATIVES

Alternative	**RelocationsOptions Hulah and Copan**	**TotalYield Hulah (mgd)***	**Total Yield Copan (mgd)***	**Total Yield (mgd)**
3C	5.0% FC Reallocation at Hulah and Copan	8.33	8.43	16.76
3D	10.0% FC Reallocation at Hulah Existing WS at Copan	14.19	0.88	15.07
3I	1% FC Reallocation at Hulah, 10.0% FC Reallocation at Copan	4.89	11.47	16.36

(5) Reallocation Alternative Cost Summary. Reallocating from flood control would require additional water supply contracts to reimburse the government for the investment in flood control storage given up in the reallocation and for other impacts. Those costs include both environmental and physical costs, both of which were estimated for this study. Costs that were considered include pipeline costs from Copan to Lake Hudson, pipeline energy costs, additional storage costs, costs associated with upstream and downstream environmental and cultural resource impacts, upstream replacement costs of capital improvements within the new conservation pool, and downstream flood benefits that would be foregone over the study period.

(6) Alternative 3C - evaluated reallocating 5.0% of the flood control storage at both Hulah and Copan Lakes. This alternative would increase the conservation pool at Hulah from elevation 733.0 to 736.67 feet. For Copan the conservation pool would increase from elevation 710.0 to 711.99. The total estimated cost for this alternative was about $54.7 million (M). The majority of this cost was from pipeline costs from Copan ($25 M) and additional water supply storage costs ($27.2 M). Upstream replacement costs to replace reservoir facilities, mineral leasehold interests, and also to mitigate cultural and environmental assets was estimated at $2.3 million. Somewhat surprising was that the flood control benefits foregone where rather small in relation to the overall projected cost for this alternative. The flood control benefits foregone can be expressed as an increase in average annual damages from the current, or baseline, condition. The increase in average annual damages for Alternative 3C is $ 10,090. The present value of the total damages that could be anticipated over the 50 year study period is estimated to be $188,000.

(7) Alternative 3D - evaluated reallocation of 10% of the flood pool at Hulah Lake, and no changes at Copan Lake. The conservation pool would increase from elevation 733.0 to 739.46 feet at Hulah with no change in the current conservation pool elevation of 710.0 at Copan. The total cost for this alternative was $56.5 M, which was slightly higher than alternative 3C. Pipeline construction and energy costs for the pipeline was slightly greater at $26.8 M, due to necessary pipeline improvements. Water supply storage costs were estimated at $20.8 M, about $6.4 M less than alternative 3C. Storage costs were less because storage costs are a partial function of the initial costs to build Hulah and Copan reservoirs. However, a large change in the conservation pool from elevation 733.0 to 739.46 would result in significantly more upstream replacement of reservoir facilities and environmental mitigation; the total cost was estimated to be $8.6 M. Downstream flood benefits foregone increased the average annual damages by $11,920. The present value of the total damages that could be anticipated over the 50 year study period is estimated to be $222,000

(8) Alternative 3I - evaluated reallocation of 1% of the flood pool at Hulah Lake and a 10% reallocation at Copan Lake. Pipeline and energy costs were $26.5 M. Storage Costs were higher at $37.6 M because of the higher initial construction costs of Copan compared to Hulah. Upstream reservoir replacement costs and environmental mitigation was much lower however and was estimated to be $605,000. Downstream flood benefits foregone increased the average annual damages by $9,044, the least impact of any of the alternatives. The present value of the total damages that could be anticipated over the 50 year study period is estimated to be $176,000

(9) Alternative 4: Private Sand Lake Reservoir with pipeline to Hudson Lake. This alternative assumes no Federal Authorization. Alternative 4 investigated potential sites for non-Federal development of a water supply project at or near the deauthorized Sand Lake site. The location of the previously studied site is about 8.5 miles west and 1.5 miles south of Bartlesville on Sand Creek in Osage County, just upstream of the Town of Okesa. The sit is heavily wooded and the normal pool would back water upstream along Sand Creek past a Boy Scout Camp and Osage Hills State Park. A brief site visit identified that some portions of the park would be permanently inundated by the conservations pool and additional facilities would be temporarily inundated during flood events. Given the concerns identified, a very preliminary search was conducted to see if any other potential locations upstream of the Federal Authorized lake on Sand Creek could be suitable and provide the necessary water supply source to minimize construction costs.

The yield at the deauthorized site was projected to be about 12 mgd. Added to the yield available through the city's existing contract at Hulah Lake or 4.35, that would provide the city with a total of 16.35 mgd which meets the 2055 needs.

The total estimated first cost for this alternative is about $86 M. A breakdown of these costs reveals pipeline construction costs of $23.9 M, with energy costs over a 50 year period of $10.5 M, reservoir construction costs of $32.8 M, land acquisition and relocation costs of $7.6 M, and with contingencies of $10 M. Environmental and cultural impacts analysis costs were estimated at $900,000, but environmental and cultural resources costs were not evaluated and could be significant, as could the costs associated with acquisition of mineral rights.

(10) Alternative 5: Purchase water supply storage from Kaw Reservoir with Pipeline to Hudson Lake Alternative 5 investigated the purchase of water supply storage from Kaw Reservoir and the cost to build a Pipeline to the city owned Hudson Lake. The city's net need of 10.45 mgd is available from Kaw Lake and the estimated cost of a contract for water supply storage at Kaw Lake is $4.8 M.

The cost of this alternative is estimated to be about $106 M, including pipeline construction costs of $86 M, energy costs of $14 M over a 50 year period, and the $4.8 M cost of storage at Kaw Lake. Cost to assess the environmental and cultural impacts is estimated at $200,000, but costs to mitigate for those impacts were not specifically and could increase the $106 M estimate for this alternative.

e. **Summary of Water Supply Alternatives.** This study looked at multiple alternatives to meet the city of Bartlesville's long term water supply needs. The first two alternatives looked at the existing water supply sources to estimate the available yield through year 2055. Alternative # 1 identified a 2055 daily average yield of 4.35 from existing water supply from Hulah Reservoir. Alternative #2 evaluated the water supply yield available through year 2055 assuming that new water supply agreements outlined in the 2006 Hulah and Copan Reallocation study are implemented. This alternative provided sufficient water supply to meet year 2035 water supply demand, but was insufficient to meet projected demand through year 2055. Alternative #2 identified a projected average daily yield of 6.85 mgd through year 2055.

Alternatives 3C, 3D, 3I, 4, and 5 will all supply sufficient water supply to meet 2055 demand requirements. Alternative 3C, 3D, and 3I analyzed reallocation of the flood control pool to water supply at Hulah and Copan lakes. Alternative 4 evaluated constructing a new non-Federal reservoir and pipeline in Osage County at the deauthorized Sand Lake site and

Alternative 5 evaluated water supply and pipeline costs from Kaw reservoir. The table below summarizes the potential costs for Alternatives 3C, 3D, 3I, 4 and 5.

Option	Water Source	Total New Storage Ac-Ft	Total Yield mgd	Total Pipeline Cost Plus Present Value 50 Year Energy Costs	Total Water Supply Storage Cost	Upstream Reservoir & Environment Mitigation	PV of Downstream Reservoir Flood Damages	Total Alternative Cost
3C	5% FLOOD POOL + WQ REALLOCATION AT HULAH & COPAN	22,131	16.76	$24,965,845	$27,253,140	$2,312,000	$188,000	$54,718,988
3D	10% Flood Pool + WQ REALLOCATION AT HULAH & 0% REALLOCATION AT COPAN	27,354	15.07	$26,827,593	$20,796,700	$8,627,000	$222,000	$56,473,293
3I	1% FLOOD POOL + WQ REALLOCATION AT HULAH & 10% REALLOCATION AT COPAN	20,524	16.36	$26,533,348	$37,593,486	$605,000	$176,000	$64,927,834
4	SAND LAKE	30,479	14.80	$34,561,484	$50,511,250	$900,000*	$0	$85,972,734*
5	KAW RESERVOIR	10,710	14.80	$100,832,244	$4,788,731	$200,000*	$0	$105,820,975**

* Costs do not include mitigation or costs of other environmental or cultural resource impacts and do not include costs associated with the acquisition of mineral rights.
** Costs do not include mitigation or costs of other environmental or cultural resource impacts

10. RECOMMENDATIONS

Increasing future demand for adequate water is being recognized as a high priority for many municipalities. It is a complex issue that has many variables to consider. Assumptions related to supply and demand, economic construction costs, environmental impacts, land and easement costs, and reallocation of project purposes from flood control to water supply all impact the final projected cost for each alternative.

For the near term, through year 2035, the approval of new water supply agreements that reallocate storage from water quality to water supply at Hulah and Copan Lakes would provide 7.2 mgd from Hulah and 5.54 mgd from Copan. A total of about 12.74 mgd would be added to the water supply available for the City of Bartlesville, to meet demand through year 2035.

For the longer term, to year 2055, information has been presented for Bartlesville to make an informed long term water supply decision. Based on the evaluation of the costs of the alternatives presented in this study, a future reallocation of the flood pool from both Hulah and Copan appears to be the least costly alternative. These costs would be analyzed in more detail as part of any future flood pool reallocation study. The least costly alternative is alternative 3C, reallocating 5% from the flood pool at both Hulah and Copan lakes.

Downstream flood damage increases were not significant to the final alternative costs. In addition, the buyout of homes and/or businesses highly susceptible to recurrent flood damages could also be effective in reducing flood damages. The costs associated with a flood plain buyout should be looked at in more detail if reallocation of storage from the flood pool is pursued.

APPENDIX A

Letter Agreement

LETTER AGREEMENT
PLANNING ASSISTANCE TO STATES

CITY OF BARTLESVILLE

BARTLESVILLE WATER SUPPLY AND CONVEYANCE STUDY

THIS AGREEMENT, entered into this ____ day of____, 2006, by and between the United States of America (hereinafter called the "Government"), represented by the District Engineer for the Tulsa District, U.S. Army Corps of Engineers; and the City of Bartlesville (hereinafter called the "Sponsor").

WITNESSETH, THAT

WHEREAS, Section 22 of the Water Resources Development Act of 1974 (Public Law 93-251), as amended, authorizes the Secretary of the Army, acting through the Chief of Engineers, to assist the states in the preparation of comprehensive plans for the development, utilization and conservation of water and related land resources; and

WHEREAS, Section 319 of the Water Resources Development Act of 1990 (Public Law 101-640) authorizes the Secretary of the Army to collect from non-Federal entities fees for the purpose of recovering 50 percent of the cost of the program established by Section 22; and

WHEREAS, the Sponsor has reviewed the State's comprehensive water plans and identified the need for planning assistance as described in the Scope of Studies incorporated into this Agreement; and

WHEREAS, the Sponsor has the authority and capability to furnish the cooperation hereinafter set forth and is willing to participate in the study cost-sharing and financing in accordance with the terms of this Agreement;

NOW THEREFORE, the parties do mutually agree as follows:

1. The Government, using funds contributed by the Sponsor and appropriated by the Congress, shall expeditiously prosecute an investigation of potential surface water sources of water supply conveyance systems for the City of Bartlesville, substantially in compliance with Scope of Work attached as Appendix A and in conformity with applicable Federal laws and regulations and mutually acceptable standards of engineering practice. Three alternatives will be developed with the input of the Sponsor that address the present and future water supply needs of the City of Bartlesville and its customers. Alternatives

include supplemental supply from Kaw Lake and additional supply from Hulah and Copan lakes.

2. The Government shall contribute in cash 50 percent of the total study cost, and the Sponsor shall contribute in cash and work-in-kind 50 percent of the total study cost, which total study cost is $245,000; provided, that the Government shall not obligate any cash contribution toward Study costs, until such cash contribution has actually been made available to it by the Sponsor. The Sponsor agrees to provide $_30,000__ in-kind services and funds in the amount of $_92,500__ which shall be made payable to the Finance and Accounting Officer, Tulsa District, 1645 South 101 East Avenue, Tulsa, Oklahoma 74128-4609.

3. No Federal funds may be used to meet the local Sponsor's share of study costs under this Agreement unless the expenditure of such funds is expressly authorized by statute as verified by the granting agency.

4. Before any Party to this Agreement may bring suit in any court concerning any issues relating to this Agreement, such party must first seek in good faith to resolve the issue through negotiation or other form of nonbinding alternative dispute resolution mutually acceptable to the Parties.

5. This Agreement shall terminate upon the completion of the Study; provided, that prior to such time and upon thirty (30) days written notice, either party may terminate or suspend this Agreement without penalty. It is further understood and agreed that if the Study is not completed by December 30, 2007, or cannot be completed within the total study cost of $245,000, this Agreement may be renewed or amended by the mutual written agreement of the parties.

6. Within ninety days after termination of this Agreement, the Government shall prepare a final accounting of the study costs, which shall display (1) cash contributions by the Federal Government, (2) cash and work-in-kind contributions by the Sponsor, and (3) disbursements by the Government of all funds. Subject to the availability of funds, within thirty days after the final accounting, the Government shall reimburse the Sponsor for non-Federal cash contributions that exceed the Sponsor's required share of the total study costs. Within thirty days after the final accounting, the Sponsor shall provide the Government any cash contributions required to meet the Sponsor's required share of the total study costs.

7. In the event that any (one or more) of the provisions of this Agreement is found to be invalid, illegal, or unenforceable by a court of competent jurisdiction, the validity of the remaining provisions shall not in any way be affected or impaired and shall continue in effect until the Agreement is completed.

8. This Agreement shall become effective upon the signature of both Parties.

FOR THE SPONSOR:

By:______________________________

Julie Daniels,
Mayor
City of Bartlesville, Oklahoma

Date:________________

Attest:

By:______________________________
Secretary

Date:________________

(Seal)

FOR THE GOVERNMENT:

By:___________________________

Miroslav P. Kurka
Colonel, U.S. Army
District Commander

Date:_______________

FOR THE OKLAHOMA WATER RESOURCES BOARD (OWRB):

The OWRB hereby attests that this Planning Assistance to States study, to investigate water supply needs and alternatives for the City of Bartlesville and its service area, and promotes the goals and objectives of the State of Oklahoma Water Plan.

By:______________________________
Rudolf J. Herrmann,
Chairman
Oklahoma Water Resources Board

Date:_______________

Attest:

By: _____________________________
Bill Secrest, Secretary

Date:_______________
(Seal)

APPENDIX A

SCOPE OF STUDY
PLANNING ASSISTANCE TO STATES
BARTLESVILLE WATER SUPPLY AND CONVEYANCE STUDY

BARTLESVILLE, OKLAHOMA

1. GENERAL. The Corps shall investigate potential water supply sources for the City of Bartlesville. The evaluation shall define the City of Bartlesville water supply needs through year 2055, provide an evaluation of water conservation measures that could be implemented upstream of Hulah and Copan watersheds, and further analyze additional water supply options from Hulah, Copan, and Kaw Lakes. A quantitative analysis of future municipal and industrial water needs for the City of Bartlesville and its customers shall be conducted. Differing growth scenarios shall be evaluated to determine the most likely future water needs of the City of Bartlesville and its customers. This study is being conducted under authority given in Section 22 of the 1974 Water Resources Development Act, Planning Assistance to States Program. The information developed as a result of this study will enable the determination of the amount and cost of water available for water supply from the respective sources.

2. WORK TO BE PERFORMED. The Corps PAS study Shall develop, with sponsor input, the present and future water supply demand and net water supply needs of the City of Bartlesville and its customers.

Once the City of Bartlesville water supply needs through year 2055 are determined (as outlined in Task a. below, a determination shall be made by City of Bartlesville officials as to the level of effort required for remaining tasks (Tasks b, c, & d outlined below) to be completed.. The sponsor shall identify how future funding will be allocated for the remaining study tasks, upon acceptance of the defined water needs identified in Task a, below. If required, a revision of the scope of work for the remaining tasks shall also be completed.

Primary water supply options to be studied include supplemental water supply from Kaw

Lake and reallocation of water from flood control storage to water supply at Hulah and/or Copan Lake. If upon review by City of Bartlesville and Corps officials, it is determined that Copan Lake and/or Kaw Lake water is considered a viable alternative, engineering pipeline costs for transport of Copan and or Kaw Lake water to Hulah and/or Hudson Lake shall be studied. A fourth study measure shall be to define conservation measures that reduce the sediment load and prolong the available water supply yield through year 2055 for Hulah and Copan Lakes. Tasks necessary to complete the scope of work include:

a. Define Future Needs of Bartlesville And Its Service Area. Based on existing 2006 conditions and in coordination with the Sponsor and other interests in the service area, projections of future water demands shall be made. Categories for users are residential, commercial, industrial, public, and other uses, including losses; however, other categories or subcategories may be developed as required during the conduct of the study. Demographic and economic variables, such as population, employment by North American Industry Classification System (NAICS), housing density, and median household income, shall be used as a basis for projecting future water needs. Types of commercial and industrial use shall be categorized by NAICS Classification. The Institute for Water Resources Municipal and Industrial Needs (IWR-MAIN) Water Demand Management Suite software shall be used to forecast future water needs. Water Conservation shall also be considered in the analysis.

b. Evaluate the engineering and environmental pipeline costs that would be required for purchasing water supply storage from Kaw and Copan Lakes.

(1) Prepare Preliminary Engineering Estimates. The Corps shall review and update existing engineering planning details for water supply delivery alternatives from Kaw Lake and Copan Lake. The routes to be studied will be reviewed with City of Bartlesville representative's prior initiation of this task. It is anticipated that the selected pipeline routes from Kaw and Copan reservoirs shall be to Lake Hudson (or intersect with the existing Hulah pipeline from Hulah Lake to Lake Hudson.) Costs of alternative water supply sources from previous Corps reallocation studies will be used and referred to in preparing preliminary engineering planning revisions. The engineering planning tasks shall update preliminary engineering costs Kaw and Copan water supply alternatives.

(2) Environmental Studies

(a) Endangered Species Coordination. The Corps shall coordinate the study of Kaw and Copan pipeline routes with the United States Fish and Wildlife Service and the Oklahoma Department of Wildlife Conservation Service to learn the impacts, if any, on any listed endangered species. If endangered species are found in the project area, the Corps shall recommend that the Sponsor conduct a biological assessment and possibly formal consultation.

(b) NEPA and Other Environmental Requirements. The Corps shall discuss, in narrative format, National Environmental Policy Act (NEPA) and other environmental requirements that the Sponsor will need to address prior to development of detailed engineering designs. The Corps shall also prepare discussion concerning the requirements for future coordination with Federal, State, and local agencies having legislative and administrative responsibilities for environmental protection.

(3) Real Estate Studies

(a) Real estate activities necessary for the project consist of all tasks related to determining general real estate requirements and identifying and providing general real estate cost estimates.

(b) The Corps shall conduct a limited gross appraisal of the selected alternatives to decide the estimated real estate costs and estates purchase requirements, i.e., fee or type of easement. The Corps shall use available maps of the study area that contain sufficient detail to identify the types of land and improvements that the proposed project would affect. The Corps shall briefly search the local real estate market to gather data concerning a sample of recent sales of improved and unimproved properties comparable to the right-of-way required. The research may involve searching deed records and contacting local appraisers, brokers, attorneys, central appraisal district, and others knowledgeable of the local real estate market. The Corps shall use the market information as a basis for the values of the various types of properties within the proposed project. Cost information shall be incorporated into the MCACES cost estimate.

(4) Prepare Cost Estimates. Cost estimates shall be provided that include preliminary engineering costs, real estate costs, environmental costs, operations and maintenance costs, and cost per 1,000 gallons of water for Kaw and Copan reservoir alternative.

c. Estimate the environmental and flood benefit losses that would be incurred for reallocating part of the flood control pool in Hulah and Copan Lake to water supply storage, dependent on net needs identified in task 1a.

(1) Upstream Flood Pool Losses Depending on the water needs identified in task (a); NEPA, environmental, cultural and real estate (structures, roads, buildings etc.) impacts will be estimated upstream in the conservation and flood pool. These costs will be analyzed upstream of Hulah and or Copan Lake depending on water needs identified. This review shall look for primary impact areas affected by the normal and seasonal conservation pool raise and will provide an evaluation of environmental, cultural, and economic losses incurred upstream/in-lake in the conservation pool area.

(2) Downstream Flood Damages Depending on the water needs identified to be reallocated from flood control in task (a), flood damages will be estimated downstream from Hulah and Copan lakes based on flood pool changes. Economic flood control losses will be determined for Hulah and Copan. Depending on the demand needed the collective combined losses from a flood control reallocation from both lakes will also be estimated. This review will provide a cost estimate of flood benefit losses.

d. Water Supply Initiatives and Conservation Measures for Hulah & Copan and the City of Bartlesville

(1) Evaluate the feasibility and costs associated with studying potential actions to lengthen the longevity and viability of Hulah and Copan Lakes. Possible study actions include dredging to remove silt accumulations; in-stream silt traps, erosion control for adjacent uplands, natural stream restoration on tributaries to restore stability and thus reduce sedimentation.

(2) Evaluate the potential of stream restoration and watershed Natural Resource Conservation Service (NRCS) impoundments on Caney River tributaries upstream from Bartlesville, and tributaries that flow into Hulah and Copan Lakes, to compensate for reduced flood control on Hulah and Copan Lakes resulting from potential reallocation of some flood control storage to water supply.

(3) Institutional Analysis. Focusing on addressing Hulah and Copan Reservoirs, the study will review existing authorities, agreements and other basin-wide institutional arrangements that could be used to address water supply related issues. The review will include local, state, and federal memorandums of agreement, compacts, regulations, and laws. Water supply issues include such as stream flow, inflow of nutrients and control of sediments into the lake which impact the quality and quantify of water in the lake for the purposes of water supply. The study will outline how those existing institutional arrangements might be used to develop best management practices in the basins above these two reservoirs, including areas in the State of Kansas.

e. Project Management.

(1) This work item shall include all scheduling and organizing of the study; regular periodic meetings with technical elements to review progress; preparing budget documentation; monitoring and managing all funds being obligated and expended; preparing project-related correspondence; coordinating with Federal, Tribal, State, and local agencies; and providing guidance and support as required to ensure that they have answered all questions and they have solved all study-related problems. The Corps shall do this task for the duration of the study.

(2) The Corps shall manage the tasks associated with overall

coordination of the various study work items including funds management and work item scheduling. The overall purpose of this work item is to ensure that the study shall accomplish the goals established, maintain schedule and cost estimates, and address all items in the Scope of Study.

f. Report Preparation.

(1) Report preparation shall consist of preparing a draft report, duplicating and distributing the draft report, reviewing and editing the draft report to final form, and then duplicating and distributing the final report. The report will be direct, concise, and written in a style that is easy to understand and may include graphics, illustrations, and photographs. The report shall also include the study findings and recommendations.

(2) The Corps shall document the study results in report form. The Corps shall base the report on all studies and investigations conducted and on published reports applicable to the study area.

3. DATA TO BE PROVIDED BY NON FEDERAL SPONSOR

The City of Bartlesville shall provide all data available and related to water availability and water use in the study area. The demand for water study area includes the City of Bartlesville and its customers in Washington, Osage, and Nowata counties, Oklahoma. The City shall provide data and information about the current monthly water usage by major use category, as explained below, and the capability of the existing and planned future supply/treatment facilities. Specific information to be gathered shall include:

Name of customer, or user, and service area
Description of distribution system
Location, capacity, and description of treatment facilities
Cost of water, price to consumers
Quantity of water used by month and major use category, if available

4. DELIVERY AND SCHEDULE.

(a) Draft Document. The Corps shall provide a draft copy of the report to the Sponsor and Oklahoma Water Resources Board. The report shall include discussion concerning methodology, data sources, findings, and other appropriate data for review and approval. The report shall identify all data sources and references.

(b) Final Document. Upon the Sponsor's approval and return of the edited draft to the Corps, the Corps shall furnish the final original document to the Sponsor.

(c) Meetings and Conferences. The Corps and the Sponsor shall hold meetings, either face-to-face or through telephone conference calls as needed upon request to discuss problems as identified.

(d) Schedule. The Corps shall submit the above items according to the following schedule.

Item	Schedule
Task (a) Water Supply & Conveyance Study	120 Calendar days after the date of the receipt of funds.
Task (a) Water Supply & Conveyance Draft Document	150 Days after receipt of Funds
Task (a) Water Supply & Conveyance Draft Sponsor Review	180 Days after Receipt of Funds
DECISION POINT*	
Task (b) Engineering Kaw and or Copan Pipeline Evaluation	330 Days after Receipt of Funds
Task (c) Estimate Environmental and Flood benefit losses for reallocation of flood pool at Hulah and/or Copan.	330 Days after Receipt of Funds
Task (d) Water Supply Initiatives and Conservation Measures	330 Days after Receipt of Funds
Final Document	360 Days after Receipt of Funds

* THE CITY OF BARTLESVILLE WILL REVIEW TASK (a) WITH THE CORPS OF ENGEENERS AND DISCUSS POTENTIAL SCOPE MODIFICATIONS AS TO THE LEVEL OF EFFORT REQUIRED FOR THE REMAINING SCOPE OF WORK TASKS. REMAINING TASKS (b,c,&d)SHALL NOT START UNTIL THIS DECISION POINT IS FINALIZED AND APPROVED BY THE CITY OF BARTLESVILLE AND THE CORPS OF ENGINEERS. THERE WILL BE NO NET CHANGE IN THE TOTAL STUDY COST.

(e) Coordination. The Corps of Engineers shall maintain a close working relationship with the City of Bartlesville and its representative throughout the execution of the study.

(f) Report and Documentation. The computation and procedures used in this study shall be documented in a final report. The report shall include pertinent table, graphs, plots, maps, and other related documents.

(g) Review. All computations shall be reviewed by qualified personnel for soundness and legitimacy. All comments and discussion shall be documented and included as part of the study file.

(h) Final Delivery. Final delivery shall include a bound report and documentation along with a CD or DVD with all computations and backup data.

5. PROJECT MANAGER

The Government manager for this contract shall be Ms. Cynthia Kitchens, Project Manager for the Planning Assistance to States Program, Programs and Project Management Division, Tulsa District, U.S. Army Corps of Engineers. Questions or problems that may arise during the performance of the work specified in this Agreement shall be discussed with Ms Kitchens. The Sponsor shall coordinate entry clearance with Ms. Kitchens before planning site or office visits. The Sponsor shall appoint a project coordinator to serve as a single point of contact or liaison with the Corps of Engineers. The name of the individual so designated shall be furnished in writing to the Corps. The project coordinator shall be responsible for complete coordination of the work.

APPENDIX B

TIME AND COST ESTIMATE
PLANNING ASSISTANCE TO STATES

WATER SUPPLY AND CONVEYANCE STUDY
BARTLESVILLE AREA, OKLAHOMA

Study Item	Duration (Workdays)	Cost ($)
Water Supply Needs And Conservation Measures	180	50,000
2. Preliminary Planning-Copan to Lake Hudson	330	70,000
3. Preliminary Planning-Kaw Lake to Lake Hudson	330	50,000
4. Evaluate Hulah/Copan upstream restoration measures	330	25,000
5. Report Preparation and Study Management	360	50,000
Total PAS Project Cost		245,000

APPENDIX B

Water Supply Needs Analysis

APPENDIX B

Water Supply Needs Analysis

Phase I

Introduction

Background. Phase I of a two-phase study effort was completed in March 2007. This first phase determined future net water needs for the City of Bartlesville and the surrounding communities, rural water systems, and other areas to which the City provides water. The first phase contains an estimate of future demand for water based on three different population growth scenarios Washington County could experience from 2005 to 2055, with year 2005 representing the base year. The City of Bartlesville expects population growth in the city and in Washington County to occur at a much faster rate than historic growth. Population forecast scenarios were made for the City of Bartlesville, two rural water districts the City supplies, and Washington County. The City supplies water to approximately 99% of the residents in Washington County. Since nearly all of the water demands in Washington County are supplied by the City of Bartlesville, forecasts are based on Washington County data.

Water Demand

Introduction. Estimates of the quantities of water needed in the future require the use of appropriate econometric models. These models are used to project future water use that is statistically consistent with long-term water supply planning. In order to forecast Municipal & Industrial (M&I) water demand the Institute for Water Resources-Municipal and Industrial Needs (IWR-MAIN) Water Demand Management Suite, a Windows based PC software package, was used to translate existing population, housing, and employment into estimates of existing water demands for the 2005 base year. These base year estimates are then used to fine-tune the water use equations for translating the long-term projections of population, housing, and employment into disaggregated forecasts of water use. Washington County is the basic study area unit for forecasting water demand due to the availability of demographic data for the county and for the sub-sectors as well. Actual water use data for year 2005 was supplied by the City of Bartlesville and included Washington County and the City of Dewey. Some of the included rural water districts may overlap into neighboring counties. Residential and non-residential are the major water use sectors specified within IWR-MAIN. The residential sector includes both single-family and multi-family sub-sectors. The non-residential sector includes the sub-sectors of commercial, manufacturing, and government. The commercial sub-sector includes construction, transportation, wholesale, retail, finance, and services. The public use sector and unaccounted for water use are also included in the evaluation.

Projection Scenarios . Three water demand scenarios were presented to the Water Resource Committee of Bartlesville. The Baseline Projection Scenario is based on historical growth and weather pattern trends experienced in the study area. Due to the fact that the population of Washington County has not increased significantly over the past ten years, the baseline water demand forecasts have not deviated from the base year by a substantial amount. The baseline projection is based on a 2055 population of 53,000 in Washington County.

The City of Bartlesville provided information on actual water use for the base year 2005. This information was disaggregated into different sectors of water use such as residential, municipal, industrial, commercial, water districts, and public schools. In addition to this, the City also provided information on population and housing projections for years 2000-2050. This data was then used to develop a high growth scenario for the water system that Bartlesville supplies. The City developed these growth projections based on the current level and pace of development. The water demand forecast for the high growth projection was based on a 2055 population of 73,000 and developed by the Tulsa District using the Institute of Water Resources Municipal and Industrial Needs (IWR-MAIN) forecast system. The Oklahoma Department of Commerce (ODC) provided the demographic data of population estimates as the basis for employment and housing projections. Other sources of information include, the U.S. Department of Labor, the U.S. Census, and the Oklahoma State Climate Center, and the National Weather Service.

A third projection, called the mid projection, was interpolated from the baseline and high projections. Water demand forecasts for the mid projection population were not conducted with IWR-MAIN but are derived as an average of the baseline and high-growth projections.

Projection Scenario Variables

The projection scenarios were developed using the following variables:

Population

Population is a key parameter used in IWR-MAIN to project residential water demand. In 2002, the Oklahoma Department of Commerce, under contract with the OWRB, expanded their 2000-2030 projections of the resident population of Oklahoma by county. The projections were made using a cohort component projection model. With this method, each component of the population, births, deaths, and migration, is projected separately, based on algorithms developed by the U.S. Bureau of the Census. The base population used is April 1, 2000, the date of the U.S. Census of Population and Housing count of the United States resident population. Fertility, death, and migration rates are applied to that base population to arrive at the near year projection period. For this analysis, the medium set of 5,000 person in-migration per year was used. Year 2005 was interpolated between 2000 and 2010. Table 1 shows the baseline and the high population projections for Washington County.

Table 1: Population Projections for Washington County						
Year	**2005**	**2015**	**2025**	**2035**	**2045**	**2055**
Baseline Projection	48,996	50,300	51,100	51,600	52,300	53,000
High Projection	48,996	53,436	58,065	63,877	69,685	73,169

Employment

Commercial and industrial water use is also considered in determining current and future water demand in Washington County. IWR-MAIN projects water demand for commercial, industrial, and public use categories using the number of persons employed in a city or county by each Standard Industrial Classification (SIC) category, and, since 1997, the North American Industry Classification System (NAICS) code. Data used in IWR-MAIN utilizes the NAICS system. National water use survey data was utilized to provide water use coefficients for each industrial sector, by two or three digit code, based on the number of employees.

To project future industrial water demand, the model utilizes a linear relationship using employment and water use per employee by NAICS code. Employment in Washington County by place of work is the basic unit of analysis for projecting future water demand. Because future employment in Washington County for the 50-year projection period has not been completed by the State, a method was developed to estimate future employment using State employment data for the base year and the U.S. Census Bureau County Business Patterns and U.S. Department of Labor projections of future labor force conditions. Table 2 displays employment projections for Washington County by year.

Table 2: Employment Projections for Washington County						
Year	**2005**	**2015**	**2025**	**2035**	**2045**	**2055**
Baseline Projection	16,100	18,330	20,746	22,625	23,103	23,530
High Projection	16,100	19,473	23,574	28,008	30,783	32,484

Housing

Another parameter used by IWR-MAIN to project future residential water use is housing units. Data from the U.S. Bureau of Census and Housing were used to develop housing units for the 2005 base year. Because the Census is released decennially, population and housing information from 2000 was used in lieu of developing new baseline data for the year 2005. It is assumed that the person- per -household ratio will remain constant over the entire projection range. Table 3 shows the baseline and high projections for housing for Washington County.

Table 3: Housing Projections for Washington County

Year	2005	2015	2025	2035	2045	2055
Baseline Projection	22,511	23,110	23,478	23,707	24,029	24,351
High Projection	22,511	25,040	27,441	30,446	33,462	35,135

Future Water Demand

Introduction. The forecasting algorithm of IWR-MAIN is built to operate on data corresponding to study areas, water use sectors/sub-sectors, months, and forecast years. The needs and data available dictate the degree of detail required to use the model. The methodology utilized is known as the "Driver Times Rate of Use." In other words, for a given study area, sector, month, and forecast year, water use can be calculated as a product of the number of users, the rate of use, and the number of days in the given month. This allows the disaggregation of a water demand forecast and permits unit water use rate, such as gallons per household, gallons per employee, etc, to be assumed or predicted via the water use model. The algorithm used in the projection of residential water demand uses persons per household, population divided by number of housing units, as well as housing density. The housing density variable is a parameter used to characterize the outdoor component of water use for the summer season

Methodology. The method that was selected for forecasting residential water demand uses median household income, persons per household, housing density, marginal price of water, maximum temperature, and precipitation, to adjust per unit usage rates for residential information, but not for non-residential variables. For the non-residential sector, a model for water demand was customized using values for intercept terms, model variables, and associated coefficients and elasticities. The base year per unit water use rate is calculated from the base year water use and the number of counting units for the sub-sector. This calculated rate of use is then adjusted by the relationship between sub-sector water use and those explanatory variable selected for the sub-sector, which are selected by the user and may change over time. Year-to-year changes in water use are explained by the change in the selected explanatory variables and the counting units. Counting units derived from population projections, are the driver variables, such as employee counts, housing units, acres, etc., associated with each sub-sector.

Unaccounted Water Usage/System Losses. The amount of unaccounted water use and system losses was calculated by taking the difference in the amount of water that Bartlesville draws to supply to the system (raw water) and the amount of water reported as being used by the City of Bartlesville for the year 2005. This calculation showed that approximately 13% of the water is not accounted for in Washington County.

Peak Demand. Another output IWR-Main can forecast is peak water demand. Peak use for a community can vary month to month depending mainly on temperature and rainfall. Typically record peak use will occur in the hottest summer months, because this is a period where water demand significantly increases as homeowners are watering their lawns and gardens more frequently and precipitation rates are low. The system peak use may be specified in gallons per day, thousand gallons per day, or million gallons per day. The user must select the month in which the base system peak occurs and enter the peak use value. For this study, the City of Bartlesville supplied the peak use in million gallons per day which occurred in the month of July.

Results. Table 4 displays the results of the water demand evaluation for the baseline projection by sector and projection year for Washington County. The baseline demand projection reflects the minimum water demand by year 2055 in order to determine net needs from water supply sources.

Table 4: Water Demand by Sector and Year
Washington County
Baseline Projection
(million gallons a day)

Year	2005	2015	2025	2035	2045	2055
Residential	5.89	6.18	6.20	6.31	6.30	6.60
Commercial	1.3	1.38	1.39	1.4	1.46	1.44
Industrial	0.65	0.71	0.76	0.81	0.82	0.827
Municipal	0.26	0.27	0.275	0.28	0.29	0.29
Unmetered/Unaccounted	1.2	1.26	1.28	1.3	1.33	1.34
Total	9.3	9.8	9.9	10.1	10.2	10.5

Table 5 displays the results of the water demand evaluation for the high projection by sector and projection year for Washington County. The high demand projection reflects the maximum water demand by year 2055 in order to determine net needs from water supply sources.

Table 5: Water Demand by Sector and Year Washington County High Projection (million gallons a day)						
Year	**2005**	**2015**	**2025**	**2035**	**2045**	**2055**
All Residential	5.9	6.8	7.4	8.2	9.0	9.5
Commercial	1.3	1.4	1.6	1.8	1.9	2.0
Industrial	.65	.76	.83	.93	.98	1.0
Municipal	.26	.30	.31	.36	.39	.40
Unmetered/ Unaccounted	1.2	1.4	1.6	1.8	1.8	1.9
Total	9.3	10.7	11.7	13.1	14.1	14.8

On March 1, 2007, the City agreed to proceed with the water demand projections based on the mid and high population growth projections ranging from 63,000 to 73,000 by 2055, which equates to water demand in 2055 being 12.8 to 14.8 million gallons per day (mgd). Due to the uncertainty of both demand and supply 50 years in the future, a range of net needs was determined to estimate future water supply needs.

Existing Water Supply

Introduction. Currently, Bartlesville obtains most of its water from Hulah Lake, which is then pumped to Hudson Lake prior to treatment. During periods of insufficient supply from Hulah Lake and Hudson Lake, water can be pumped from the Caney River under emergency conditions.

Hulah Lake. Hulah Lake construction started in May 1946, and was completed in February 1951 for flood control, water supply, low flow regulation, and conservation purposes. Embankment closure began in February 1950 and was completed in June 1950. Impoundment of the conservation pool began on September 23, 1951, and was completed on September 24, 1951. The project was placed in full flood control operation in September 1951.

Table 6 outlines pertinent data for Hulah Lake. Lake data is based on the 2002 sedimentation survey.

Table 6: Hulah Lake Pertinent Data				
Feature	**Elevation (feet)**	**Area (acres)**	**Capacity (acre-feet)**	**Equivalent Runoff* (inches)**
Top of Dam	779.5	-	-	-
Top of Flood Control Pool	765	13,000	289,000	7.40
Flood Control Storage	733.0-765.0	-	257,900	6.61
Spillway Crest	740.0	5,160	61,400	1.57
Top of Conservation Pool	733.0	3,120	22,565**	0.80
Conservation Storage	710.0-733.0	-	22,553	0.80
Top of Inactive Pool	710.0	0	12	-

*From a 732-square-mile drainage area above the dam site.

** Includes 16,600 acre-feet for water supply, 5,953 acre-feet for water quality control, and 12 acre-feet for sediment reserve.

Copan Lake. Copan reservoir construction began in November 1972, and the project was placed in useful operation in April 1983. Copan Reservoir provides flood control benefits to Bartlesville and is a second close water supply alternative that Bartlesville is considering. Copan currently has one million gallons per day (mgd) of available water supply and a reallocation of water quality storage to water supply was recommended by the Tulsa District and approved by the United States Army Corps of Engineers Headquarters (USACEHQ) in September 2007. Table 7 displays pertinent data for Copan Reservoir.

Table 7: Copan Lake Pertinent Data				
Feature	**Elevation (feet)**	**Area (acres)**	**Capacity (acre-feet)**	**Equivalent Runoff* (inches)**
Top of Dam	745.0			
Maximum Pool	739.1	17,850	338,200	12.57
Top of Flood control Pool	732.0	13,380	227,700	8.45
Flood Control Storage	710.0-732.0		184,300	6.84
Top of Conservation Pool	710.0	4,449	34,634	1.61
Conservation Storage	687.5-710.0		33,887**	1.59
Spillway Crest	696.5	1,080	4,700	0.17
Top of Inactive Pool	687.5	110	747	0.02

* Drainage area is 505 square miles.

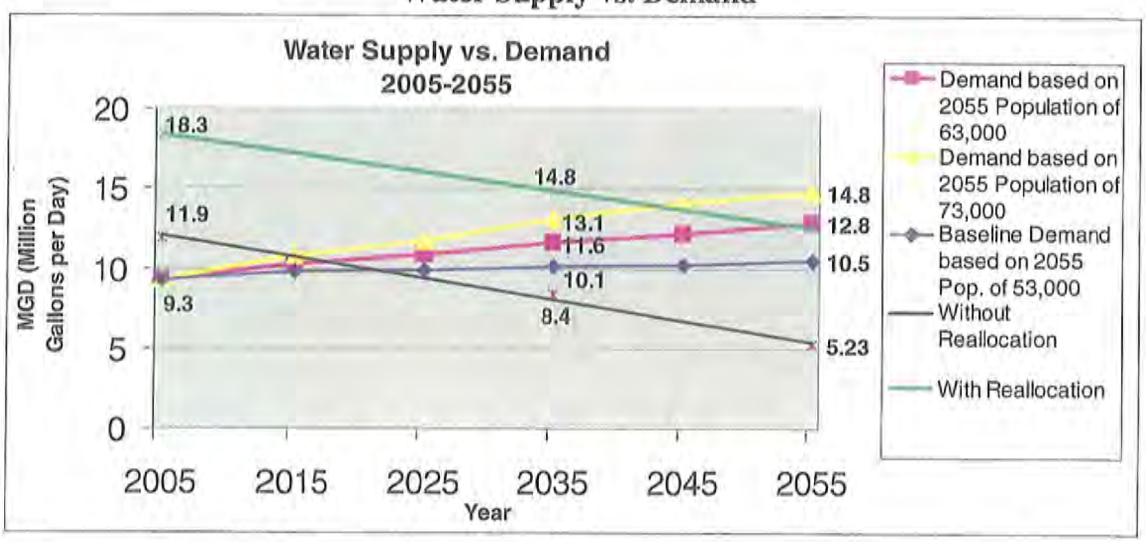
** Includes 7,500 acre-feet for water supply (3.0 mgd yield), 26,100 acre-feet for water quality control (16 mgd yield), and 9,200 acre-feet for sediment based on 1983 survey..(In year 2002, useable storage=34,634acre-feet less 747 acre-feet)

Dependable Yield. The Corps of Engineers' 2006 Hulah and Copan Reallocation study indicated that the City of Bartlesville has 6.4 million gallons a day (mgd) of dependable yield from Hulah Lake through year 2035 using historical data for the 50 year drought of record and the latest 2002 sediment survey for Hulah Lake. Based on the latest 2002 sediment survey, assuming no measurable reduction in the rate of sediment deposition, the dependable yield at Hulah Lake is projected to decline from 6.4 mgd in year 2035 to 4.35 mgd by year 2055. The total available water supply only yield from both reservoirs is 5.23 mgd in year 2055.

Net Water Needs

Results. . Net water needs for Washington County are based on existing water supply sources for the City of Bartlesville from Hulah Lake and Hudson Lake. In addition, Copan Lake provides 2.0 mgd of water supply to the Copan Public Works Authority. Water is also obtained from the Caney River from releases at Copan Lake in emergency situations. Figure 1 shows the existing supply of water and the available supply after the recommended reallocation against the demand for water based on the three different projection scenarios by year 2055. Based on these projections, water demand will exceed available supply(depicted by the without reallocation line) beginning around year 2015 and continuing over the next 40 years if no reallocation of the existing sources of water supply or the addition of new water supply sources. The with reallocation line depicts the water supply after the recommended reallocation is complete. There, demand exceeds supply before year 2045 for the high projection demand or before 2050 for the mid projection demand. However, the baseline projection demand does not exceed the reallocation supply through the study period. Based on the population projection of 73,000 and water demand of 14.8 mgd, an additional 10.45 mgd of water will be required by year 2055.

Figure 1:
Washington County
Water Supply vs. Demand



APPENDIX C

H&H Analysis - Bartlesville PAS Water Supply Analysis for Year 2055

APPENDIX C

H&H Analysis
Bartlesville PAS Water Supply Analysis for Year 2055 for the City of Bartlesville Using SUPER

The city of Bartlesville experienced a critical shortage in available water supply at Hulah Lake, beginning in the summer of 2001. The lake experienced a drawdown to 20 percent of the conservation pool by early April 2002. Fortunately, the pool filled with a large, single event in early May 2002. The drought conditions prompted the city to investigate and possibly develop other sources of water supply to meet future water supply demands.

The City of Bartlesville has estimated their regional water supply need to be 14.8 mgd through 2055. Current yield projections show that without any reallocations the city will have 4.35 mgd of yield available at Hulah and 0.88 mgd of yield available at Copan, for a combined yield of 5.23 mgd through 2055. This combined yield will not meet Bartlesville's future requirements. This portion of the study provides yield analysis for reallocating various portions of flood control storage at both Hulah and Copan Lakes, and looks at the sensitivity of the yields developed, since there is a degree of uncertainty with developing projections and yields this far into the future. Therefore, this study looks at specific, possible alternatives to meet the city's future water supply needs including reallocations from the flood control pools at both Hulah and Copan.

Yield projections through the year 2055, required that pool sediment projections be made through the year 2055 at both Hulah and Copan based on past sediment surveys, historic rates of sedimentation, soil types, and inflows. There is a great deal of uncertainty when projecting this far into the future. The sediment projections were used to establish the elevation-area-capacity relationship of both Hulah and Copan Lakes through 2055.

All modeling for this study was accomplished with the Corps of Engineers Southwest Division Modeling System for the Simulation of the Regulation of a multi-purpose Reservoir System, otherwise known as SUPER. The SUPER Model is a suite of computer programs used to model multi-purpose reservoir system regulation.

Overview of SUPER Model

The suite of programs used to model multi-purpose reservoir system regulation known as SUPER, was developed over a thirty-year period by Ronald L. Hula, primarily as a planning tool to perform period-of-record analysis, to evaluate changes in operational scenarios. The model has the ability to simulate flood control operations, and conservation pool operations including hydropower, water supply, water quality, diversions, and returns. In addition to period-of-record analysis, it has the capability to perform conservation pool yield analysis, and firm energy analysis. It has the capability to develop unregulated conditions models, simulating systems with some or all reservoirs "dummied" out or non-existent. Besides system modeling, SUPER can perform economic analyses of impacts between plans, and it can provide a wide variety of output from which to evaluate scenarios including tabular or graphical formats of hydrographs, duration plots, and frequency curves at all reservoirs and control points within the system model.

SUPER is a daily simulation model that assumes all reservoirs are in place for the entire period of record specified for each model, based on data availability. For each SUPER model, a complex set of intervening area flows is developed for the entire period of record. This is the culmination of the pre-processing of data, before any simulation is done. When simulation is begun, headwater reservoir inflows and subsequent derived releases based on current and future forecast conditions, are then routed through the system on a daily basis. These routed flows are combined with intervening area flows at all control point locations. Reservoir releases are made for flood control, hydroelectric power generation, water supply requirements, and stream flow requirements such as water quality and irrigation. Other regulating considerations include channel capacities and bank stability. All releases are analyzed to determine their impact on current and future forecasted conditions, and are adjusted as needed to meet predefined system constraints. In addition to the above requirements, SUPER works to achieve a target uniform balance between all competing reservoirs during the draw down of system flood storage, and a target uniform balance in system conservation storage remaining, as defined by the model, during a conservation pool draw down. SUPER continues to evolve to meet the complex challenge of modeling system operations while meeting system and local constraints, and balancing requirements. The SUPER algorithms and data will soon be incorporated into the RiverWare modeling program which is a more object oriented and flexible platform for reservoir system modeling.

The Arkansas River SUPER model has a hydrologic period of record from January 1940 to December 2000, based on observed gage data. Therefore, all analyses using SUPER reflect actual hydrologic conditions which occurred during this 61 year period.

Yield Analysis

Water supply yield analysis using SUPER was performed to determine how much yield would be available for the City of Bartlesville, for a number of possible alternatives including:

- Existing Conditions through year 2055 at both Hulah and Copan.
- Reallocate available water quality storage to water supply storage at both projects for 2055 conditions.
- Reallocate 2.5% and 5.0% of flood control storage along with available water quality storage at both projects for 2055 conditions.
- Find combinations of the above alternatives and other possible percentages of flood control reallocation, to achieve enough yield to meet the estimated 2055 demand.
- Perform a yield sensitivity analysis by varying the monthly demand to reflect demands similar to the 2002 drought, vary sedimentation rates by 10 and 20%, and reducing inflows by 10 and 20%.

The yields determined in this study were the critical period dependable yields, meaning there were no deficiencies in water supply experienced during the worst drought in the historic period of record from 1940-2000. Water supply demands are input into the model as a monthly value for each month, and are modeled as continuous flows out of the reservoir for the entire period of record. A typical, conservative monthly demand distribution exists in the SUPER model for all water supply reservoirs, based on historical usage, however in reality, this is a dynamic parameter and can change over time. Super however, uses the same monthly distribution for the entire 61 year period of record. The yield computation is an iterative solution to determine the maximum water that can be continually removed from the lake based on the storage, inflow, evaporation, and any required releases such as water quality demands. The reservoir is drawn down just to the bottom of conservation pool, at the end of one modeling time period, only once during the period of record. For existing conditions at Hulah, however, there was one day of water supply deficiency during the critical period, based on existing contracts. The yields determined in the modeling of Hulah and Copan reflect the necessity to meet water quality requirements at Hulah outflow, Copan outflow, and Bartlesville. Minimum water quality requirements at these locations are shown in Table 1. To ensure water supply and water quality requirements are met at these three locations, a systems approach to yield analysis was required. Reservoir yields determined this way may be less than if analyzing each reservoir, Hulah and Copan, individually. However, yields shown in the analyses, meet water supply and water quality requirements at all times during the critical period without deficiencies, with the exception of Hulah existing conditions for 2055. The historic drawdown period for the yield analysis began in Oct 1955, reaching the maximum drawdown in Mar 1957, and fully recovering by April to May 1957.

Table 1 Current Water Quality Demands

Month	**Water Quality Demands below Hulah in cfs**	**Water Quality Demands below Copan in cfs**	**Water Quality Demands at Bartlesville in cfs**
Jan	2	5	10
Feb	2	5	10
Mar	2	5	10
Apr	2	5	10
May	2	5	10
Jun	4	8	11
Jul	4	8	13
Aug	4	8	13
Sep	2	5	10
Oct	2	5	10
Nov	2	5	10
Dec	2	5	10

Existing conditions storages and yields for Hulah and Copan are shown in Tables 2 and 3 for 2055 conditions.

Table 2

Hulah – Existing Conditions based on 2055 conditions – if no changes are made					
	Elevation (ft)	Usable Storage for 2055 Conditions (ac-ft)	Yield (mgd)	Percent of	
				Usable Total Storage (%)	Usable Conservation Storage (%)
Flood Control	733-765	251,824		96.77	
Conservation	710-733	8397	5.94	3.23	100.00
Water Supply		6180	4.37	2.37	73.60
City of Bartlesville		4807	3.40	1.85	57.249
City of Bartlesville, MOD		687	0.49	0.26	8.178
Hulah Water District, Inc		31	0.02	0.01	0.37
City of Bartlesville		656	0.46	0.25	7.807
Water Quality		2217	1.57	0.85	26.40
Total Usable Storage		260,221		100.00	

- There is no storage below El 725
- Note: 1 day of WS deficiency at Hulah during the drought of record, with both Hulah and Copan modeled as existing conditions.
- City of Bartlesville requires 14.8 mgd in year 2055.
- WS available to Bartlesville at Hulah = 4.35 mgd and WS available to Bartlesville at Copan = 0.88 mgd, for a total of 5.23 mgd.

Table 3

Copan – Existing Conditions based on 2055 conditions – if no changes are made					
	Elevation (ft)	Usable Storage for 2055 Conditions (ac-ft)	Yield (mgd)	Percent of	
				Usable Total Storage (%)	Usable Conservation Storage (%)
Flood Control	710-732	180,126		86.97	
Conservation	687.5-710	26,980	11.81	13.03	100.00
Water Supply		6,022	2.64	2.91	22.32
Copan Public Works		4,015	1.76	1.94	14.881
Uncontracted		2,007	0.88	0.97	7.44
Water Quality		20,958	9.17	10.12	77.68
Total Usable Storage		207,106		100.00	

Tables 4 and 5 show the modified condition storages and yields for Hulah and Copan when all unused water quality storage is reallocated to water supply for 2055 conditions.

Table 4

Hulah – Existing Conditions based on 2055 conditions – There is no WQ available to reallocate to WS

	Elevation (ft)	Usable Storage for 2055 Conditions (ac-ft)	Yield (mgd)	Percent of	
				Usable Total Storage (%)	Usable Conservation Storage (%)
Flood Control	733-765	251,824		96.77	
Conservation	710-733	8397	5.94	3.23	100.00
Water Supply		6180	4.37	2.37	73.60
City of Bartlesville		4807	3.40	1.85	57.249
City of Bartlesville, MOD		687	0.49	0.26	8.178
Hulah Water District, Inc		31	0.02	0.01	0.37
City of Bartlesville		656	0.46	0.25	7.807
Water Quality		2217	1.57	0.85	26.40
Total Usable Storage		260,221		100.00	

There is no storage below El 725

Table 5

Copan – Modified Conditions based on 2055 conditions – Reallocate all available WQ storage to WS

	Elevation (ft)	Usable Storage for 2055 Conditions (ac-ft)	Yield (mgd)	Percent of	
				Usable Total Storage (%)	Usable Conservation Storage (%)
Flood Control	710-732	180,126		86.97	
Conservation	687.5-710	26,980	11.81	13.03	100.00
Water Supply		9,732	4.26	4.70	36.07
Copan Public Works		4,015	1.76	1.94	14.881
Uncontracted		5,717	2.50	2.76	21.19
Water Quality		17,248	7.55	8.33	63.93
Total Usable Storage		207,106		100.00	

- Maintains only 1 day of WS deficiency at Hulah during the drought of record (Mar 1957)
- City of Bartlesville requires 14.8 mgd in year 2055.
- WS available to Bartlesville at Hulah = 4.35 mgd and WS available to Bartlesville at Copan = 2.50 mgd, for a total of 6.85 mgd.

After modeling existing conditions and the reallocation of all available water quality water at both Hulah and Copan, additional analysis was done to model reallocation of 2.5% and 5% of the flood pool at both lakes in addition to water quality storage. From this analysis, it was determined that an approximate10% reallocation of flood pool would be required at either lake to obtain enough yield to meet 2055 demands. Table 6 shows a listing of the possible system combinations that were analyzed and the determined yields available to Bartlesville.

Table 6 System Yield Analysis

System Combinations					
Hulah	**Available Yield for Bartlesville (mgd)***	**Copan**	**Available Yield for Bartlesville (mgd)***	**Total Available Yield for Bartlesville (mgd)***	**Deficiencies during Drought of Record (Mar 1957)**
Existing	4.35	Existing	0.88	5.23	1 day WS @ Hulah
Existing	4.35	Available WQ Reallocation	2.50	6.85	1 day WS @ Hulah
2.5% FC Realloc + avail WQ	6.21	2.5% FC Realloc + avail WQ	6.88	13.09	None
5.0% FC Realloc + avail WQ	8.51	Existing	0.88	9.39	None
5.0% FC Realloc + avail WQ	8.33	5.0% FC Realloc + avail WQ	8.43	16.76	None
10.0% FC Realloc + avail WQ	14.19	Existing	0.88	15.07	None
10.0% FC Realloc + avail WQ	13.75	10.0% FC Realloc + avail WQ	11.47	25.22	None
Existing	4.35	10.0% FC Realloc + avail WQ	5.79	10.14	1 day WS @ Hulah
2.5% FC Realloc + avail WQ	6.18	5.0% FC Realloc + avail WQ	8.43	14.61	None
2.5% FC Realloc + avail WQ	6.06	10.0% FC Realloc + avail WQ	11.47	17.53	None
1% FC Realloc + avail WQ	4.89	10.0% FC Realloc + avail WQ	11.47	16.36	None

*All yields above are based on 2055 sediment conditions at Hulah and Copan
Estimated water supply requirement for the City of Bartlesville in 2055 is 14.8 mgd.

Selected Alternatives for Period of Record Analysis

The system combinations highlighted in Table 4, which met the city's required 2055 water supply demand of 14.8 mgd, were further analyzed to determine the associated costs versus damages to provide an economic basis from which to analyze the alternatives. Period of record runs were made of these selected alternatives, to develop discharge-frequency data that was input into a backwater model of the Caney River and the Little Caney to develop stage-damages. Also, a volume-duration relationship was established to aid in determining damages due to event durations.

Sensitivity Analysis

1. Update Yield using updated monthly peak distribution data from drought of record that occurred in the 2002 drought.

Based on the 2001, 2002, and 2003 water demand records from the City of Bartlesville, the average monthly water demand from Hulah Lake was calculated, and the distribution was input into the existing conditions Hulah yield SUPER model. The current and drought water supply distributions for Hulah Lake are shown in Table 7, along with the reservoir yields for these two conditions. The 2001-2003 drought distribution was modeled throughout the entire 61 year period of record, and since the distribution is based on a drought period, it is a more severe or worst case distribution compared to the existing or typical distribution. As can be seen in Table 7, the summer demands are greater in the drought distribution compared to the existing distribution, and this continues through the fall and early winter months, a time of year when conditions are drier. The drawdown period for this yield run began in October 1955 with the maximum drawdown occurring in February 1957, and recovering in April 1957. The maximum drawdown occurred earlier than yield runs made with the existing demand distribution, Because of the higher demands during a drier period, a lower overall yield at Hulah was experienced for this condition. It is highly unlikely that this more severe distribution would occur for a prolonged period of time. It is more realistic to use a more conservative, typical distribution, based on longer term historic demands. However, this analysis shows the sensitivity of the overall yield to drought, or possible long term climate change, towards a drier period. As shown in the table, the existing conditions 2055 yield available to the City of Bartlesville with the revised monthly water demand distribution = 3.92 mgd versus 4.35 mgd with the original distribution.

Table 7

Average Monthly Water Demand from Hulah Lake		
Month	**Current Distribution-Existing Conditions (mgd)**	**2001-2003 Distribution-Existing Conditions (mgd)**
January	3.66	4.94
February	3.66	2.94
March	3.92	2.42
April	3.92	1.11
May	4.18	2.28
June	4.98	2.96
July	5.75	6.16
August	5.75	6.18
September	4.98	4.17
October	4.21	4.36
November	3.66	4.41
December	3.66	5.15
Yield available to Bartlesville	**4.35**	**3.92**

2. Using updated monthly peak distribution data, determine the % of time that demands are met.

With the updated monthly peak distribution data from 2001-2003 for existing conditions, there are 27 days during the 61 year period of record when there are water supply deficiencies (Jan-Mar 1957 drought), which is 0.12% of the time. This means that 99.88% of the time demands are met. With the current distribution for existing conditions, there is 1 day in the 61 year period of record (Mar 1957) when there are deficiencies. This means that 99.9955119% or rounded to 100% of the time water supply demands are met.

3. Vary sedimentation rates in both Hulah and Copan by 10%and 20%, and determine respective change in yields.

Modeling results are shown in Table 8.

Table 8

Impacts to Yield due to Variable Rates of Sedimentation			
Lake	**Condition**	**Available Yield for Bartlesville (mgd)**	**Change from Existing Conditions (mgd)**
Hulah	Existing	4.35	
	10% decrease in sedimentation rate (more storage available)	5.43	1.08
	10% increase in sedimentation rate (less storage available)	4.26	-0.09
	20% decrease in sedimentation rate (more storage available)	5.62	1.27
	20% increase in sedimentation rate (less storage available)	3.87	-0.48
Copan	Existing	0.88	
	10% decrease in sedimentation rate (more storage available)	0.89	0.01
	10% increase in sedimentation rate (less storage available)	0.87	-0.01
	20% decrease in sedimentation rate (more storage available)	0.90	0.02
	20% increase in sedimentation rate (less storage available)	0.86	-0.02

4. Reduce the inflows into Hulah and Copan by 10% and 20% and determine respective changes in yields.

Modeling results are shown in Table 9.

Table 9

Bartlesville PAS
Sensitivity Analysis
Impacts to Yield due to Decreased Inflow Rates

Lake	Condition	Available Yield for Bartlesville (mgd)	Change from Existing Conditions (mgd)
Hulah	Existing	4.35	
	10% decrease in inflow	3.92	-0.43
	20% decrease in inflow	3.54	-0.81
Copan	Existing	0.88	
	10% decrease in inflow	0.87	-0.01
	20% decrease in inflow	0.85	-0.03

As seen in tasks 3 and 4 in the sensitivity analysis, Hulah yield is much more sensitive to changes in inflow and sedimentation, as compared to Copan yield. This is likely due to the much smaller conservation pool at Hulah (8397 ac-ft in 2055) versus Copan (26,980 ac-ft in 2055). Yield is dependent on storage, inflow, evaporation, and required releases such as water quality releases. So, slight changes to inflow and storage are not dampened as much at Hulah, as they are at Copan Lake.

APPENDIX D

Cost of Alternative Water Supply Sources

APPENDIX D
COST OF ALTERNATIVE WATER SUPPLY SOURCES

The U.S. Army Corps of Engineers-Tulsa District (USACE-Tulsa), the Oklahoma Water Resources Board (OWRB) and the City of Bartlesville, Oklahoma (Bartlesville or City) are cooperating under Section 22 of the Water Resources Development Act of 1974 (Public Law 93-251), Planning Assistance to the States Program to develop a comprehensive water plan. C.H. Guernsey & Company (GUERNSEY) was contracted by the USACE-Tulsa to assist in evaluating least cost alternatives for providing additional water supply to the City of Bartlesville and their customers in the region.

1.1 CUSTOMER NEEDS

Bartlesville is working proactively to plan for their community and regional customer long-term water needs. In this regard, the City wants to explore alternative sources of water to supplement their primary supply from Hulah Lake. Recent short-term drought episodes resulted in significantly lowered lake levels in Oklahoma lakes. Even though 2007 has been an above average (and in some areas a record) year for rain and runoff, the City leaders vividly remember the impacts of drought and want to prepare a comprehensive plan to meet their community's future water needs and those of its customers. This is especially timely in the midst of growing evidence of global warming and its potential negative impact on the region's rainfall.

1.2 CLIMATE CHANGE AND ITS IMPLICATIONS FOR WATER RESOURCES PLANNING

The Intergovernmental Panel on Climate Change recently issued a landmark report [*Climate Change 2007: The Physical Basis - Summary for Policymakers* (IPCC, February 2007 Geneva, Switzerland)] concluding that it is very likely that hot extremes, heat waves, and heavy (short intensity) precipitation events will continue to become more frequent on a global scale. The report states there is strong observational evidence and results from modeling indicate that, at least over the last 50 years, human activities are a major contributor to climate change and global warming. Annual precipitation is projected to decrease across the southwestern United States, especially during the summer. Warmer temperatures will cause more evaporation in summer resulting in less available soil moisture. These drier conditions will lead to episodes of extreme heat, particularly across the southwest. It is projected that our typical drought episodes may transform into a more prolonged 1930s and 1950s style drought. The warmer/drier weather could increase the risk for and intensity of wildfires. It is important to keep in mind that climate model projections are uncertain because the impact depends on our socio-economic responses to climate change.

It is important to put this climate change information in context with what the City has experienced recently. Figure 1 provides a climatological perspective of rainfall in the North Central climate division of Oklahoma for the period 1895-2006, and is representative of the study region watershed as a whole. It is clear that the study region has enjoyed a recent, prolonged wet period whose duration has lasted some 15-20 years versus the more normal 8-12 year wet cycle. Additionally, the magnitude of the recent wet cycle has been greater (more annual precipitation) than any other wet cycle during the period of record, and is similar in magnitude to the droughts of the 1910s, 1930s, and 1950s. The bottom line is the City should approach this study from the perspective that their recent "memory" of rainfall conditions is far and away from what would be considered "normal." In fact, recent rainfall history has been quite abnormally high. Furthermore, the City may indeed be conducting this long-range planning effort on the doorstep of a much more prolonged period of dry weather should the climate change forecasts prove accurate.

Whatever one's view is of climate change – be it man-made or part of a normal cycle – the globe is currently warming. That fact means the City should exercise due diligence to plan, develop and protect their water resources for the future. In this regard, a "no regrets" strategy offers the best of both worlds. Should a major climate shift not occur, the benefits of a no regrets strategy would be significant. There would be no complaints about a more robust and better protected water supply.

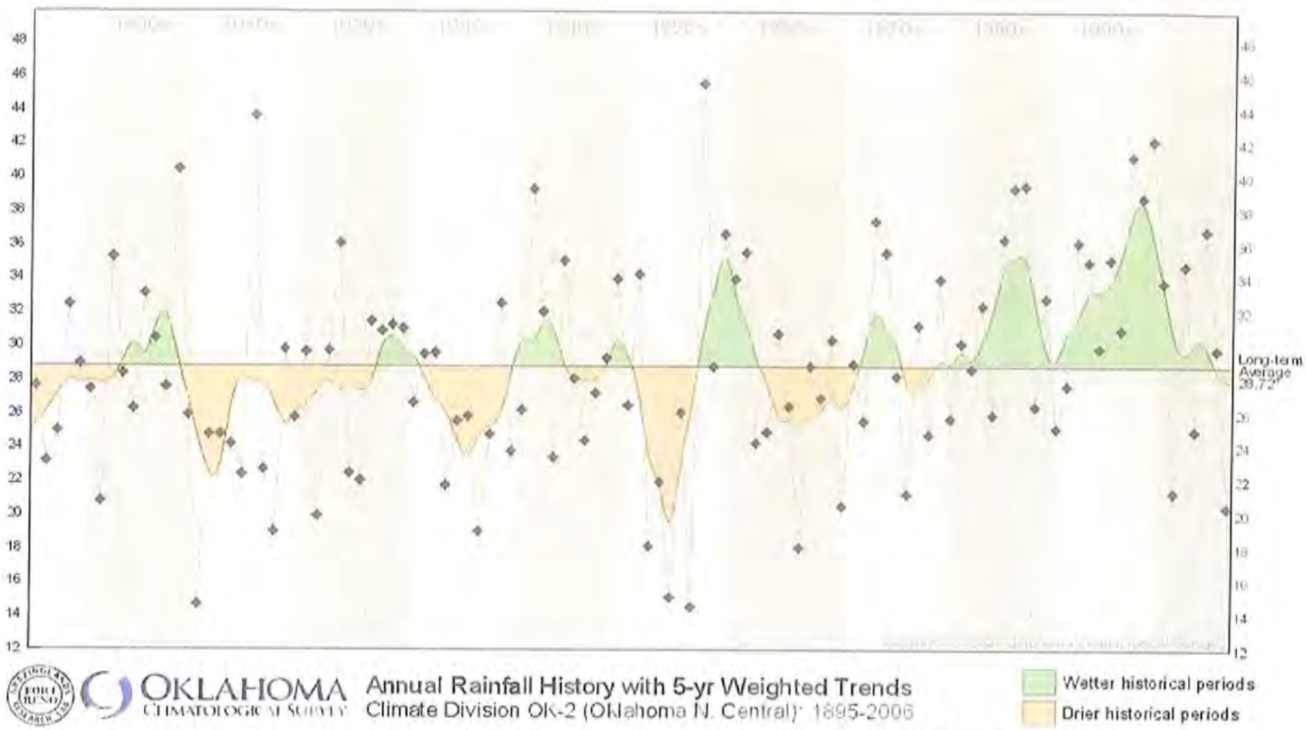


Figure 1: Rainfall History for North Central Oklahoma

NOTE: This climate change discussion was excerpted, in part, from a recent presentation by Dr. Kenneth Crawford (State Climatologist, Oklahoma Climatological Survey and Regents' Professor of Meteorology at the University of Oklahoma) to the Oklahoma Water Resources Board.

1.3 SITE DESCRIPTION

Bartlesville is located in Washington County in northeast Oklahoma. The general study area includes Washington and Osage Counties. Figure 2 (Oklahoma Comprehensive Water Plan, OWRB, 1995) provides a vicinity map.

Originally a part of the Cherokee Nation, Indian Territory, Washington County was created at statehood and named for President George Washington. The County has an area of approximately 417 square miles and a population of approximately 49,000 in 2000. Bartlesville, the County seat, was the first oil boom town in Indian Territory and has historically been home to the Phillips Petroleum Company (now ConocoPhillips).

In 1872, the United States Government purchased land from the Cherokee Nation for the Osage Tribe and it was then that the Tribe moved to Indian Territory. As statehood (1907), this Osage Reservation became Osage County, the largest county in Oklahoma. The County has an area of approximately 2,251 square miles and a population of approximately 44,000 in 2000. Pawhuska serves as the County seat. Oil and gas, as well as horse and cattle ranching on the famous bluestem grass, contribute to the economy of Osage County. Attractions to the County include Native American cultural activities, the Tall Grass Prairie Reserve north of Pawhuska, the Osage Tribal Museum and Headquarters in Pawhuska, and the Osage Hills State Park.

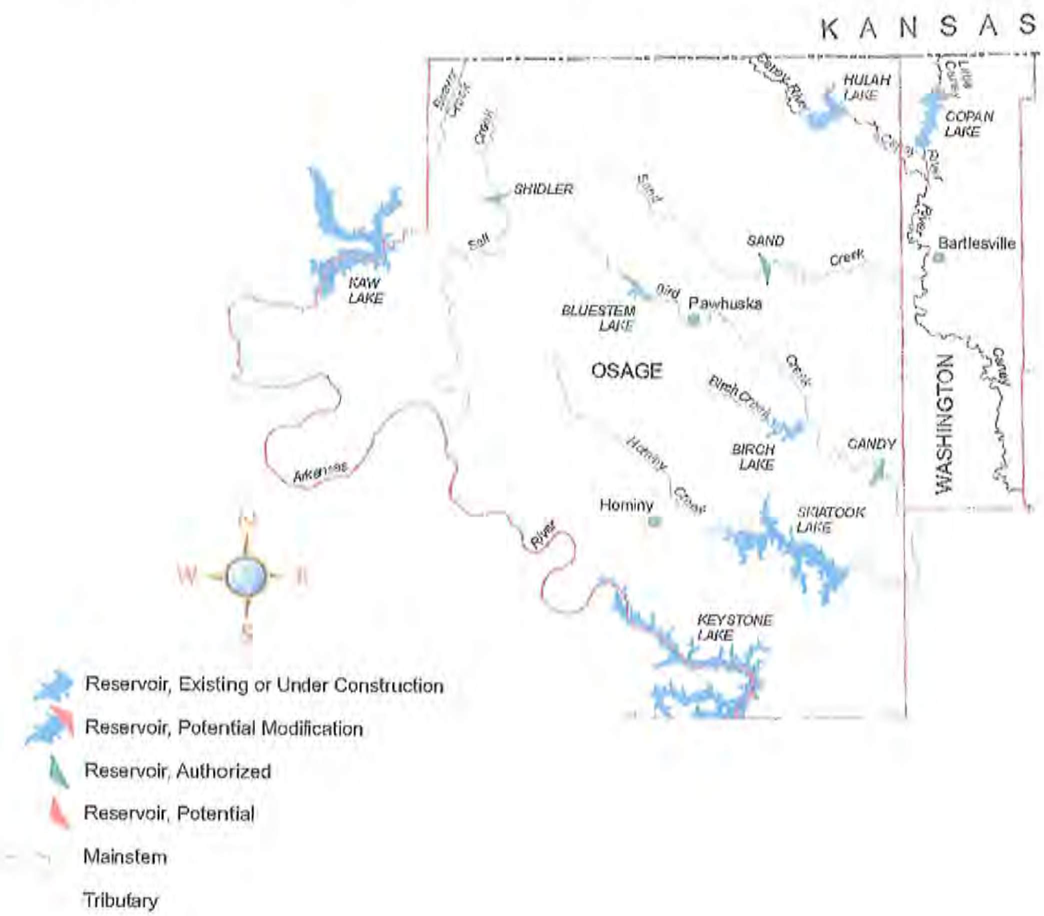


Figure 2: Vicinity Map (OWRB, 1995)

2.0 STUDY METHODOLOGY

The study will evaluate several water supply alternatives for the City of Bartlesville. The results will aid in decision-making for a 50-year planning horizon.

2.1 PROJECT KICK-OFF MEETING

A project kick-off meeting was held on October 2, 2007 at which USACE-Tulsa personnel provided some historical context of the USACE's water supply study work at Bartlesville. This included background of the Hulah-Copan Reallocation Study and more recent Planning Assistance to the States study work on evaluating long-term water supply alternatives for the City and its customer systems.

2.2 ALTERNATIVES FOR INVESTIGATION

The USACE-Tulsa has been assisting Bartlesville in evaluating reallocation of storage at both Hulah and Copan Lakes. Hulah storage is the lower cost alternative as it is an older project. USACE-Tulsa is developing updated yield projections through the year 2055 for both sites. Additionally, they are evaluating opportunities to reallocate flood control storage to water supply as an option to meet long-term needs.

The Tulsa District has tasked GUERSNEY to examine/update costs from the 2004 Tetra-Tech report on water supply and transmission options from Hulah, Copan, Kaw, and the proposed Sand Lake sites. Additionally, GUERNSEY has been requested to develop a non-federal design/constructed Sand Lake cost estimate based on the original USACE Sand Lake design information.

2.3 STUDY AREA RECONNAISSANCE

GUERNSEY personnel conducted a visual observation of the overall project region to observe identified project features in the 2004 Tetra-Tech report. This included:

- Potential intake location at Kaw Lake
- Kaw water transmission line potential alignment
- Hudson Lake outside of Bartlesville
- Copan Lake outlet, and potential intake location
- Copan Lake transmission line potential alignment to Hulah Lake
- Hulah Lake outlet and intake location
- Bartlesville transmission line from Hulah to Hudson Lake
- Pertinent feature locations around the original Sand Lake dam site

The following provides observations relevant to the current study.

2.3.1 KAW WATER TRANSMISSION LINE

The Kaw water transmission line would begin from a potential water intake structure in Kaw Lake just off the southeast bank of the State Highway 11 (SH-11) crossing (Photograph 1). The transmission line would then traverse an east/west section line alignment parallel and just south of SH-11 to Shidler. As this pipeline alignment crosses SH-11/18 south of Shidler, a high voltage power transmission line begins an east/west alignment that could prove ideal for sharing easement with the Kaw line (Photograph 2). The power/Kaw pipeline alignment then runs cross country to the east until crossing Highway 99 northeast of Pawhuska (Photograph 3) where it then parallels US Highway 60 (US-60) to within close proximity to Lake Hudson. The general countryside varies from open range lands to forested and rocky terrain. Access to the alignment was limited in many areas due to the cross country nature of the power line and the open range private land it runs through with extremely limited county road access.

Photograph 1: Kaw Lake at SH-11 Bridge Crossing

Photograph 2: Power Transmission Line Easement South of Shidler

Photograph 3: Highway 99 Power Transmission Line Crossing

2.3.2 SAND LAKE DAM SITE

The Sand Lake dam site is located about 8.5 miles west and 1.5 miles south of Bartlesville on Sand Creek in Osage County, just upstream of the Town of Okesa. The site is heavily wooded (Photograph 4) and the normal pool would back upstream along Sand Creek past a Boy Scout Camp and Osage Hills State Park. The Kaw water transmission line runs parallel and just a short distance north of the Sand Lake site and would be a logical connect/alignment for a Sand Lake water line.

Photograph 4: Sand Lake Vicinity

While performing the site visit, the Park Manager at Osage Hills State Park (OHSP) expressed deep concern for construction of the Sand Lake project as he fears that some portions of the park (Photograph 5) will be permanently inundated by the conservations pool and additional facilities temporarily inundated during flood events. The park manager also expressed concerns that the free flowing nature of Sand Creek would be lost to park visitors including the scenic "Falls" (Photograph 6). He encouraged exploring an alternative upstream dam site that would not inundate OHSP features and therefore would not be as controversial.

Photograph 5: Osage Hills State Park Swimming Pool

Photograph 6: The "Falls" at Osage Hills State Park

Additionally, the Boy Scout Camp manager expressed similar concerns that the proposed Sand Lake dam site would likely inundate significant portions of the camp facilities (Photograph 7) and remove the free flowing nature experience of Sand Creek for the visiting campers. The camp manager encouraged exploring opportunities to locate the dam site to an upstream location that would not impact the camp or the state park and therefore be less controversial.

Photograph 7: Boy Scout Camp

2.4 DATA REVIEW

USACE-Tulsa personnel provided several reference reports and maps including; two Caney River Basin Reports; Draft Environmental Assessment Report for Hulah/Copan Reallocation; Cost of Alternative Water Supply Sources, Hulah-Copan Reallocation Study; Hulah-Copan Draft Water Supply Reallocation Report and Water Supply Agreements; Kaw Lake Wholesale Water Treatment and Conveyance Study; and miscellaneous Sand Lake historical planning documents.

3.0 WATER QUALITY ANALYSIS

The water quality of Hulah, Copan, Kaw, Hudson and Sand (proposed) Lakes are all suitable for public water supply purposes. The following generalized water quality information is taken from the Oklahoma Water Resources Board's 2006 Beneficial Use Monitoring Program Report.

3.1 HULAH LAKE

Hulah Lake is considered to be eutrophic, indicative of high primary productivity and nutrient levels. The lake is currently listed as a Nutrient Limited Watershed in the Oklahoma Water Quality Standards. Water clarity was rated "poor" based on true color, turbidity, and secchi disk depth. Specific conductivity measurements indicated moderate concentrations (242 – 358 µS/cm) of electrical current conducting compounds (salts) in the lake system.

3.2 COPAN LAKE

Copan Lake is considered to be eutrophic, indicative of high primary productivity and nutrient levels. Water clarity was rated "poor" based on true color, turbidity and secchi disk depth. Specific conductivity measurements indicated low to occasionally moderate levels (176-344 µS/cm) of current conducting compounds (salts) in the lake system.

3.3 KAW LAKE

Kaw Lake is considered to be eutrophic, indicative of high primary productivity and nutrient levels. Water clarity was rated "average" based on true color, turbidity, and secchi disk depth, better than observed in 2003. Specific conductivity measurements indicated high levels (563-1172 µS/cm) of current conducting compounds (salts) in the lake system. The highest salinity and specific conductivity values were found in the Arkansas River arm during the spring and summer.

3.4 HUDSON LAKE (OSAGE COUNTY)

Hudson Lake is considered to be eutrophic, indicative of high primary productivity and nutrient levels. Water clarity was rated "good" based on true color, turbidity, and secchi disk depth. Specific conductivity measurements indicated low to occasionally moderate levels (178-297 µS/cm) of current conducting compounds (salts) in the lake system.

3.5 SAND LAKE (PROPOSED)

Unfortunately, the OWRB does not have an ambient trend monitoring station on Sand Creek; however, there is a permanent monitoring station on the Caney River near Ramona. Water enters the Caney River at Ramona from Sand Creek, Keeler Creek, and Rabb Creek, among other smaller tributaries. Therefore, this station is considered representative of the Caney River from the confluence of Sand Creek downstream to the confluence of the Caney River with Rabb Creek. While this station can give some indication as to what might be expected of the water quality in Sand Creek, actual sampling of Sand Creek should be conducted to more clearly identify its water quality characteristics. This segment of the Caney River is considered to be nutrient-threatened. Turbidity exceeded standards 50% of the time. Total dissolved solids ranged between 100 – 400 mg/L. Minerals and nutrients were consistently below standards. The Public and Private Water Supply beneficial use is supported.

4.0 RAW WATER SUPPLY INFRASTRUCTURE AND ENERGY COSTS

Five alternative water supply source combinations were studied in-depth. Alternative Source Cost Estimates are presented in Appendix A. These alternatives, labeled as Case 1 through Case 5, are described as follows:

- Case 1 – 5% Flood Pool + Water Quality Reallocation at Hulah and Copan
- Case 2 – 10% Flood Pool + Water Quality Reallocation at Hulah
- Case 3 – 1% Flood Pool + Water Quality Reallocation at Hulah and 10% Reallocation at Copan
- Case 4 – No Reallocation at Hulah and Construct Sand Lake
- Case 5 – No Reallocation at Hulah and Kaw Pipeline

4.1 COST CRITERIA

For each source combination, an average raw water supply of 14.80 million gallons per day (MGD) will be required. Intake structures, pumps, and pipelines will be required to handle twice this flow rate, or 29.60 MGD, during times of peak demand. To evaluate the cost of each source combination, a present value has been calculated. Construction costs have been estimated in 2007 dollars, with the present value set equal to the estimated cost. For energy costs, the electricity required for pumping the average of 14.80 MGD was assumed to be level during a 50-year period. Electricity costs were escalated using a 2.5% per year (compounded) inflation rate. The present value of the resulting cost series was determined using a 4-7/8% discount rate.

Three potential electricity suppliers have been identified for the various pump locations. These are PSO, Indian Electric Cooperative, and Verdigris Valley Electric Cooperative (VVEC). VVEC has a transmission line near the site of the proposed Sand Lake. Therefore, to standardize the cost projections, their rates have been used for all electricity costs:

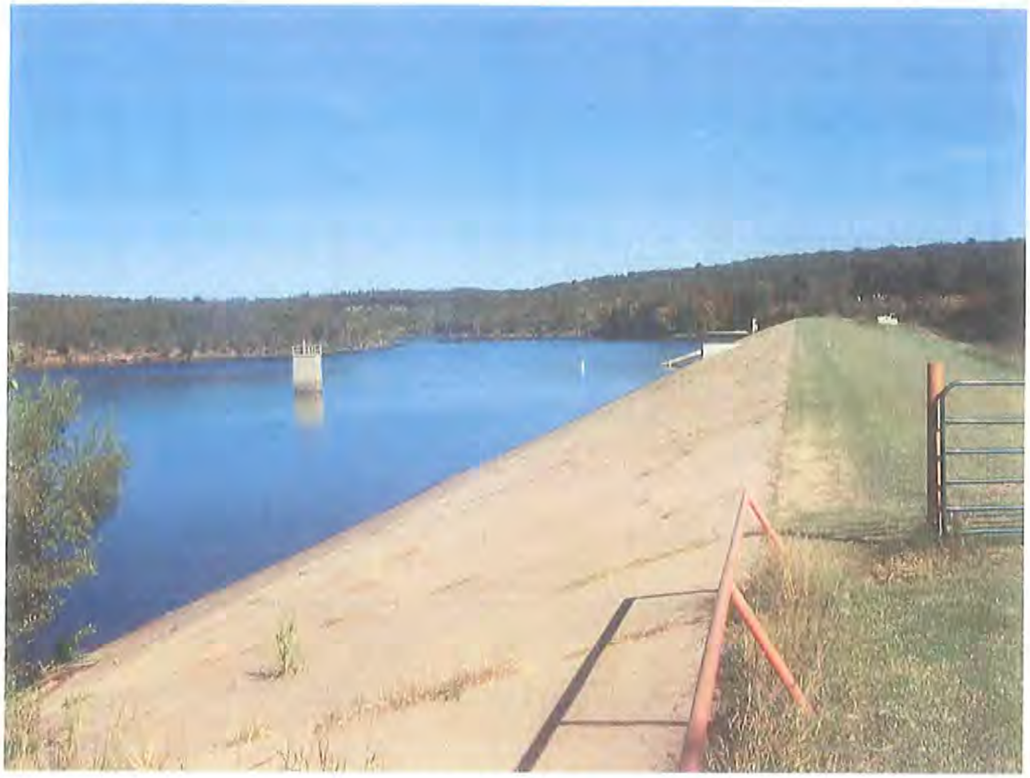
- Base Electric Charge = $50 per month
- Energy Charge = $0.02743 per kilowatts per hour (KWH)
- Demand Charge = $6.50 per KW

Pump replacements, for existing pumps at Hulah Lake as well as all proposed pumps, have been assumed to be made during the 25th year. The same 4-7/8%% discount rate was used to calculate the present value of these replacements.

Pump size has been shown as the required calculated horsepower for the given flow and head. A 70% efficiency was assumed for all pumps. Horsepower for the existing pumping station at Hulah Lake was also calculated in the same manner. Pump head was determined by assuming a hydraulic grade line running through a point 35 feet above the high point along the pipeline route. This would mean that the minimum pressure in the pipeline at the high point is 15 pounds per square inch (psi). (For all designs, the pipeline will operate by gravity flow once past the high point.) The hydraulic grade line was then projected back to the pump location based on the peak flow rate (at double the average flow) and the pipe size. Concrete cylinder pipe with a "C" value of 130 was used for all pipes.

Bartlesville maintains a run-of-the-river pump station on the Caney River (Photograph 8), located near the old water treatment plant. When the new treatment plant was constructed, an existing 30-inch pipe was "reversed" in flow direction so that the river pump station could supply raw water to the new treatment plant. Both Hulah Lake and Copan Lake discharge into the Caney River upstream of this pump station. Water from either lake could be discharged and then pumped by the river pump station to the treatment plant. However, there would be significant water losses if this method was used as a primary mode of transferring water from either of these lakes to the treatment plant. Therefore, only the small flow required from Copan Lake for Case 2 uses the river pump station. In all other supply scenarios, a pump and pipeline were used to transfer water to Lake Hudson (Photograph 9).

Photograph 8: City of Bartlesville, Caney River Intake Structure



Photograph 9: Lake Hudson (City of Bartlesville)

4.2 CASE 1 – 5% FLOOD POOL + WATER QUALITY REALLOCATION AT HULAH AND COPAN

Under Case 1, the required water supply can be met by pumping an average of 7.36 MGD from Hulah Lake and an average of 7.44 MGD from Copan Lake to Lake Hudson. Two existing 24-inch pipelines, one cast iron and one ductile iron, already convey water from the Hulah Lake pump station to Lake Hudson. These pipes will handle the required average flow of 7.36 MGD. For energy costs at the existing pump station, the inactive pool elevation of 710.0 for Hulah Lake was used. The high point along the pipeline occurs at elevation 923 located 32,100 feet away from the pump. Projecting the hydraulic grade line back to the pump gives an elevation of 1,036 for a pump head of 326 feet or 141 psi.

At Copan Lake, a new intake structure and pump station will be required. Flow to Lake Hudson will be transferred by a 30-inch pipeline. The inactive pool elevation of 687.5 for Copan Lake was used for energy costs. The 30-inch pipeline will run from the northeast to the southwest, entering an upstream arm of Lake Hudson. The high point along the pipeline occurs at elevation 919 located 21,900 feet away from the pump. Projecting the hydraulic grade line back to the pump gives an elevation of 1,003 for a pump head of 316 feet or 137 psi.

Estimated present value of the infrastructure and energy costs for Case 1 is $24,965,000 for the 50-year period.

4.3 CASE 2 – 10% FLOOD POOL + WATER QUALITY REALLOCATION AT HULAH

Almost all of the required water supply is available from Hulah Lake in Case 2. The average flow of 14.19 MGD from Hulah Lake is too much for the two existing 24-inch pipelines. A parallel 30-inch pipeline has been added, with a modified intake structure and a supplemental pump station. With this arrangement, the two existing pipes will convey an average of 6.88 MGD and the parallel 30-inch pipeline will convey an average of 7.31 MGD. The 30-inch pipeline will parallel the two existing pipes from Hulah Lake to Lake Hudson. The hydraulic grade line at the pump will be at elevation 1,027 for a pump head of 317 feet or 137 psi.

The remaining 0.61 MGD of average flow is assumed to be pumped from the Caney River back to the new treatment plant. For this level of analysis, the energy costs for this pumping were prorated using the horsepower calculated for the Hulah Lake water supply.

Estimated present value of the infrastructure and energy costs for Case 2 is $26,828,000 for the 50-year period.

4.4 CASE 3 – 1% FLOOD POOL + WATER QUALITY REALLOCATION AT HULAH AND 10% REALLOCATION AT COPAN

Case 3 is similar to Case 1, with some water supplied from Hulah Lake and the balance supplied from Copan Lake. For Case 3, the required water supply can be met by pumping an average of 4.89 MGD from Hulah Lake and an average of 9.91 MGD from Copan Lake to Lake Hudson. The two existing 24-inch pipelines from Hulah Lake to Lake Hudson will handle the required average flow of 4.89 MGD. Given this flow and the design points identified in Case 1, the projected hydraulic grade line at the pump is at elevation 994 for a pump head of 284 feet or 123 psi.

At Copan Lake, a new intake structure and pump station will be required. As the required flow is higher for Case 3 than Case 1, water to Lake Hudson will be transferred by a 36-inch pipeline instead of the 30-inch pipeline used in Case 1. The alignment of the 36-inch pipeline will be the same as described for Case 1, entering an upstream arm of Lake Hudson. With the design points identified in Case 1, the projected hydraulic grade line at the pump is at elevation 988 for a pump head of 301 feet or 130 psi.

Estimated present value of the infrastructure and energy costs for Case 3 is $26,553,000 for the 50-year period.

4.5 CASE 4 – NO REALLOCATION AT HULAH AND CONSTRUCT SAND LAKE

The September 1984 Reconnaissance Report of the Caney River Basin identified Sand Lake as a potential reservoir location at Mile 19.1 (upstream from the confluence with the Caney River) of Sand Creek. The Reconnaissance Report presented a conceptual design that included both a flood control component and a water supply component for Sand Lake.

For the purposes of this report, a cost estimate has been prepared assuming the elimination of the flood control component of Sand Lake. This allowed the top of dam elevation to be reduced from 808.5 to 788.5 and the maximum pool elevation to be reduced from 802.7 to 783.5. The top of the conservation pool was left at elevation 766.5 and the top of the inactive pool was left at 734.0. The Reconnaissance Report identified a water supply yield of 12.0 MGD, which is slightly higher than the 10.45 MGD required for this alternative. Construction of Sand Lake at this location will inundate 1,930 acres at the top of the conservation pool elevation and 3,216 acres at the maximum pool elevation. Issues regarding this inundation were previously identified in Section 2.0.

The cost estimate for Sand Lake reflects the purchase of 4,300 acres of property, including the inundated area and immediately adjacent land. Relocation costs have been included for two short sections of US-60, structures in OHSP, the Boy Scout camp, oil-field wells and pipelines, and power lines. Reservoir construction would include clearing, construction of the main dam embankment and spillway, construction of an outlet works and water supply intake, construction of access roads, and erection of a small equipment storage building. An allocation has been made for recreational facilities, although these are not specifically identified.

A pump station and 36-inch pipeline will be required to transfer raw water from Sand Lake to Lake Hudson. The pipeline will run northeast from the dam site, then parallel US-60 for several miles. The pipeline will leave the highway alignment and run northeast, then north to an arm of Lake Hudson located just upstream from the dam. The inactive pool elevation of 734.0 was used for energy costs. The high point along the pipeline occurs at elevation 970 located 36,200 feet away from the pump. Projecting the hydraulic grade line back to the pump gives an elevation of 1067 for a pump head of 333 feet or 144 psi.

The balance of the water required for Case 4 will be supplied from Hulah Lake. The required water supply can be met by pumping an average of 4.35 MGD from Hulah Lake to Lake Hudson. The two existing 24-inch pipelines from Hulah Lake to Lake Hudson will handle the required average flow. Given this flow and the design points identified in Case 1, the projected hydraulic grade line at the pump is at elevation 987 for a pump head of 277 feet or 120 psi.

Estimated present value of the infrastructure and energy costs for Case 4 is $85,073,000 for the 50-year period.

A second dam site on Sand Creek, Lake of the Osage, was identified in the Reconnaissance Report at Mile 6.8. This location is closer to Bartlesville and would require less pipeline length to convey water to Lake Hudson. As described in the Reconnaissance Report, a multi-use reservoir at this location would inundate 5,067 acres at the maximum flood pool elevation of 753.3. This elevation would also require extensive realignment of US-60. Eliminating the flood control component would reduce the maximum flood pool elevation by 22.3 feet to 731.0. This would reduce the inundated area, but would still require extensive relocation of US-60. The location of the dam site for Lake of the Osage is shown on Sheet 5 of the maps (see Appendix B).

An alternative dam site for Sand Lake is identified on Sheet 4 of the maps (Appendix B). This location is at Mile 26.8 of Sand Creek, and thus is upstream from the location identified in the Reconnaissance Report. Using this location would eliminate the inundation of the valley at

OHSP and the Boy Scout camp. To provide the required water supply yield of 10.45 MGD, the top of the conservation pool is estimated to be at elevation 793.0 and the top of the inactive pool is estimated to be at elevation 760.5. These are both 26.5 feet higher than at the original location. The maximum flood pool elevation is estimated at 810.0. The dam at this alternative location will be approximately the same height as at the original location, but will be approximately 340 feet longer. In addition, a dike would be required at a low saddle south of the dam site to contain the flood pool. Therefore, the dam construction costs will be more than for the original location. Also, a 36-inch raw water pipeline will still be required from this location to Lake Hudson. This pipeline will be approximately 18,000 feet longer than the pipeline from the original location. A slightly larger pump will be required as the head at the pump will increase from 333 feet to 344 feet.

4.6 CASE 5 – NO REALLOCATION AT HULAH AND KAW PIPELINE

The final alternative is to pump raw water from Kaw Reservoir to Lake Hudson. The intake structure at Kaw would be located at an upstream bend of the reservoir. The top of the inactive pool at Kaw is at elevation 978.0. This will require that the intake structure be located in the middle of the lake to provide enough depth during times of low water. Access to this structure will need to be from the SH-11 causeway, immediately east of the east end of the bridge. A large pump station will be required to transfer raw water through a 36-inch pipeline. The inactive pool elevation of 978.0 was used for energy costs. The 36-inch pipeline will run east from the pump station. Much of the pipeline will follow an electric transmission line until turning north, entering an arm of Lake Hudson located just upstream from the dam. The high point along the pipeline occurs at elevation 1,290 located 81,200 feet away from the pump. Projecting the hydraulic grade line back to the pump gives an elevation of 1464 for a pump head of 486 feet or 211 psi. Additional study of this 45-mile long pipeline is required to determine if an intermediate pump station will be required for pipeline integrity or will be more energy cost effective than having a single pump station.

The balance of the water required for Case 5 will be supplied from Hulah Lake. The required water supply can be met by pumping an average of 4.35 MGD from Hulah Lake to Lake Hudson. The two existing 24-inch pipelines from Hulah Lake to Lake Hudson will handle the required average flow. Given this flow and the design points identified in Case 1, the projected hydraulic grade line at the pump is at elevation 987 for a pump head of 277 feet or 120 psi.

Estimated present value of the infrastructure and energy costs for Case 5 is $100,832,000 for the 50-year period.

As discussed in previous sections, the water quality of Kaw Lake is different from the water quality of other supply sources. One major concern is the introduction of a large quantity of water with a much higher salinity level into Hudson Lake over a long period of time. This has the potential to change the environmental quality of Hudson Lake. Additional study is required to determine if this change will be harmful to the Hudson Lake aquatic environment. A potential terminal storage reservoir site is indicated on Sheet 5 of the maps (Appendix B). The pipeline from Kaw could discharge into a small reservoir at this location. From this terminal storage reservoir, raw water could be moved by gravity to the treatment plant.

4.7 LAKE HUDSON TO WATER TREATMENT PLANT

As a part of the relocation of Bartlesville's water treatment plant, a new 36-inch pipeline was installed essentially parallel to two existing pipelines (20-inch and 30-inch) from Lake Hudson to west of the water treatment plant site. From this location, a 42-inch pipeline was installed to the treatment plant. All of these pipelines operate by gravity flow using the head generated by the elevation of the water in Lake Hudson. The available head is sufficient to meet the demands used in this report. Therefore, no costs have been added for this system.

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 1 - 5% Flood Pool + WQ Reallocation at Hulah & Copan

Case 1	Hulah Existing				Copan Lake				Case 1	Case 1
Pipe Size & Flow	Ex 2 @ 24" - 7.36 MGD				30" - 7.44 MGD				Total	Present Value
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total		
Pipeline										
R.O.W.										
Land Cost		Acre	$0.00	$0	17	Acre	$1,500.00	$25,500		
Acquisition		LS	$0.00	$0	1	LS	$8,000.00	$8,000		
Total R.O.W.				**$0**				**$33,500**		
Pipe & Pump Station										
Pipe		LF	$0.00	$0	30,295	LF	$190.00	$5,756,050		
Highway Boring		EA	$0.00	$0	0	EA	$70,000.00	$0		
Pump Station		HP	$0.00	$0	1,181	HP	$2,855.00	$3,371,755		
Intake Structure		LS	$0.00	$0	1	LS	$620,000.00	$620,000		
Engineering		LS	$0.00	$0	1	LS	$975,000.00	$975,000		
S.I.O.H.		LS	$0.00	$0	1	LS	$585,000.00	$585,000		
Total Pipe & Pump Station				**$0**				**$11,307,805**		
Pipeline Costs				**$0**				**$11,341,305**		
Contingency @ 25%				**$0**				**$2,835,326**		
Total Pipeline Costs				**$0**				**$14,176,631**	**$14,176,631**	
Lake										
R.O.W.										
Land Cost		Acre	$0.00	$0		Acre	$0.00	$0		
Residential Relocation		EA	$0.00	$0		EA	$0.00	$0		
Acquisition		LS	$0.00	$0		LS	$0.00	$0		
Total R.O.W.				**$0**				**$0**		
Reservoir										
Infra. & Facility Reloc.		LS	$0.00	$0		LS	$0.00	$0		
Dam/Equip./Bldg.		LS	$0.00	$0		LS	$0.00	$0		
Recreation		LS	$0.00	$0		LS	$0.00	$0		
Engineering		LS	$0.00	$0		LS	$0.00	$0		
S.I.O.H.		LS	$0.00	$0		LS	$0.00	$0		
Total Reservoir				**$0**				**$0**		

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 1 - 5% Flood Pool + WQ Reallocation at Hulah & Copan

Case 1	Hulah Existing				Copan Lake				Case 1	Case 1
Pipe Size & Flow	Ex 2 @ 24" - 7.36 MGD				30" - 7.44 MGD				Total	Present Value
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total		
Lake Costs				$0				$0		
Contingency @ 25%				$0				$0		
Total Lake Costs				$0				$0	$0	
Total Construction Cost				$0				$14,176,631	$14,176,631	$14,176,631
Energy Costs										
Base Charge	12	Mo	$50.00	$600	12	Mo	$50.00	$600		
Energy Charge (Yearly)	3,929,000	KWH	$0.02743	$107,772	3,859,000	KWH	$0.02743	$105,852		
Demand Charge (Yearly)	10,764	KW	$6.50	$69,966	10,572	KW	$6.50	$68,718		
Energy Cost, Year 1				$178,338				$175,170	$353,508	
Energy Cost, Year 50				$598,017				$587,394	$1,185,411	
Total Energy Cost				$17,385,164				$17,076,333	$34,461,497	$10,644,221
Pump Replacement										
Year 25	1,202	HP	$200.00	$240,400	1181	HP	$200.00	$236,200	$476,600	$144,996
Total Present Value										$24,965,848

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 2 - 10% Flood Pool + WQ Reallocation at Hulah

Case 2	**Hulah Existing**				**Hulah Parallel**			
Pipe Size & Flow	**Ex 2 @ 24" - 6.88 MGD**				**30" - 7.31 MGD**			
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total
Pipeline								
R.O.W.								
Land Cost		Acre	$0.00	$0	14	Acre	$1,500.00	$21,000
Acquisition		LS	$0.00	$0	1	LS	$5,000.00	$5,000
Total R.O.W.				**$0**				**$26,000**
Pipe & Pump Station								
Pipe		LF	$0.00	$0	39,561	LF	$190.00	$7,516,590
Highway Boring		EA	$0.00	$0	0	EA	$70,000.00	$0
Pump Station		HP	$0.00	$0	1,162	HP	$2,855.00	$3,317,510
Intake Structure (Mod)		LS	$0.00	$0	1	LS	$300,000.00	$300,000
Engineering		LS	$0.00	$0	1	LS	$1,114,000.00	$1,114,000
S.I.O.H.		LS	$0.00	$0	1	LS	$668,000.00	$668,000
Total Pipe & Pump Station				**$0**				**$12,916,100**
Pipeline Costs				**$0**				**$12,942,100**
Contingency @ 25%				**$0**				**$3,235,525**
Total Pipeline Costs				**$0**				**$16,177,625**
Lake								
R.O.W.								
Land Cost		Acre	$0.00	$0		Acre	$0.00	$0
Residential Relocation		EA	$0.00	$0		EA	$0.00	$0
Acquisition		LS	$0.00	$0		LS	$0.00	$0
Total R.O.W.				**$0**				**$0**
Reservoir								
Infra. & Facility Reloc.		LS	$0.00	$0		LS	$0.00	$0
Dam/Equip./Bldg.		LS	$0.00	$0		LS	$0.00	$0
Recreation		LS	$0.00	$0		LS	$0.00	$0
Engineering		LS	$0.00	$0		LS	$0.00	$0
S.I.O.H.		LS	$0.00	$0		LS	$0.00	$0
Total Reservoir				**$0**				**$0**

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 2 - 10% Flood Pool + WQ Reallocation at Hulah

Case 2	**Hulah Existing**				**Hulah Parallel**			
Pipe Size & Flow	**Ex 2 @ 24" - 6.88 MGD**				**30" - 7.31 MGD**			
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total
Lake Costs				**$0**				**$0**
Contingency @ 25%				**$0**				**$0**
Total Lake Costs				**$0**				**$0**
Total Construction Cost				**$0**				**$16,177,625**
Energy Costs								
Base Charge	12	Mo	$25.00	$300	12	Mo	$25.00	$300
Energy Charge (Yearly)	3,574,000	KWH	$0.02743	$98,035	3,798,000	KWH	$0.02743	$104,179
Demand Charge (Yearly)	9,792	KW	$6.50	$63,648	10,404	KW	$6.50	$67,626
Energy Cost, Year 1				**$161,983**				**$172,105**
Energy Cost, Year 50				**$543,174**				**$577,116**
Total Energy Cost				**$15,790,807**				**$16,777,544**
Pump Replacement								
Year 25	1,094	HP	$200.00	**$218,800**	1162	HP	$200.00	**$232,400**
Total Present Value								

Caney River				Case 2	Case 2
0.61 MGD				Total	Present Value
Quantity	Unit	Cost	Total		
	Acre	$0.00	$0		
	LS	$0.00	$0		
			$0		
	LF	$0.00	$0		
	EA	$0.00	$0		
	HP	$0.00	$0		
	LS	$0.00	$0		
	LS	$0.00	$0		
	LS	$0.00	$0		
			$0		
			$0		
			$0		
			$0	**$16,177,625**	
	Acre	$0.00	$0		
	EA	$0.00	$0		
	LS	$0.00	$0		
			$0		
	LS	$0.00	$0		
	LS	$0.00	$0		
	LS	$0.00	$0		
	LS	$0.00	$0		
	LS	$0.00	$0		
			$0		

Caney River				Case 2	Case 2
0.61 MGD				Total	Present Value
Quantity	Unit	Cost	Total		
			$0		
			$0		
			$0	$0	
			$0	$16,177,625	$16,177,625
12	Mo	$50.00	$600		
315,000	KWH	$0.02743	$8,640		
864	KW	$6.50	$5,616		
			$14,856	$348,944	
			$49,816	$1,170,106	
			$1,448,227	$34,016,578	$10,506,798
97	HP	$200.00	$19,400	$470,600	$143,170
					$26,827,593

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 3 - 1% Flood Pool + WQ Reallocation at Hulah & 10% Reallocation at Copar

Case 3	Hulah Existing				Copan Lake				Case 3	Case 3
Pipe Size & Flow	Ex 2 @ 24" - 4.89 MGD				36" - 9.91 MGD				Total	Present Value
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total		
Pipeline										
R.O.W.										
Land Cost		Acre	$0.00	$0	17	Acre	$1,500.00	$25,500		
Acquisition		LS	$0.00	$0	1	LS	$8,000.00	$8,000		
Total R.O.W.				**$0**				**$33,500**		
Pipe & Pump Station										
Pipe		LF	$0.00	$0	30,295	LF	$216.00	$6,543,720		
Highway Boring		EA	$0.00	$0	0	EA	$70,000.00	$0		
Pump Station		HP	$0.00	$0	1,496	HP	$2,855.00	$4,271,080		
Intake Structure		LS	$0.00	$0	1	LS	$620,000.00	$620,000		
Engineering		LS	$0.00	$0	1	LS	$1,144,000.00	$1,144,000		
S.I.O.H.		LS	$0.00	$0	1	LS	$686,000.00	$686,000		
Total Pipe & Pump Station				**$0**				**$13,264,800**		
Pipeline Costs				**$0**				**$13,298,300**		
Contingency @ 25%				**$0**				**$3,324,575**		
Total Pipeline Costs				**$0**				**$16,622,875**	**$16,622,875**	
Lake										
R.O.W.										
Land Cost		Acre	$0.00	$0		Acre	$0.00	$0		
Residential Relocation		EA	$0.00	$0		EA	$0.00	$0		
Acquisition		LS	$0.00	$0		LS	$0.00	$0		
Total R.O.W.				$0				**$0**		
Reservoir										
Infra. & Facility Reloc.		LS	$0.00	$0		LS	$0.00	$0		
Dam/Equip./Bldg.		LS	$0.00	$0		LS	$0.00	$0		
Recreation		LS	$0.00	$0		LS	$0.00	$0		
Engineering		LS	$0.00	$0		LS	$0.00	$0		
S.I.O.H.		LS	$0.00	$0		LS	$0.00	$0		
Total Reservoir				$0				**$0**		

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 3 - 1% Flood Pool + WQ Reallocation at Hulah & 10% Reallocation at Copar

Case 3	Hulah Existing				Copan Lake				Case 3 Total	Case 3 Present Value
Pipe Size & Flow	Ex 2 @ 24" - 4.89 MGD				36" - 9.91 MGD					
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total		
Lake Costs				$0				$0		
Contingency @ 25%				$0				$0		
Total Lake Costs				$0				$0	$0	
Total Construction Cost				$0				$16,622,875	$16,622,875	$16,622,875
Energy Costs										
Base Charge	12	Mo	$50.00	$600	12	Mo	$50.00	$600		
Energy Charge (Yearly)	2,278,000	KWH	$0.02743	$62,486	4,888,000	KWH	$0.02743	$134,078		
Demand Charge (Yearly)	6,240	KW	$6.50	$40,560	13,392	KW	$6.50	$87,048		
Energy Cost, Year 1				$103,646				$221,726	$325,372	
Energy Cost, Year 50				$347,554				$743,509	$1,091,063	
Total Energy Cost				$10,103,863				$21,614,815	$31,718,678	$9,797,038
Pump Replacement										
Year 25	697	HP	$200.00	$139,400	1496	HP	$200.00	$299,200	$438,600	$133,435
Total Present Value										$26,553,348

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 4 - No Reallocation at Hulah & Construct Sand Lake

Case 4	Hulah Existing				Sand Lake				Case 4	Case 4
Pipe Size & Flow	Ex 2 @ 24" - 4.35 MGD				36" - 10.45 MGD				Total	Present Value
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total		
Pipeline										
R.O.W.										
Land Cost		Acre	$0.00	$0	29	Acre	$1,500.00	$43,500		
Acquisition		LS	$0.00	$0	1	LS	$10,000.00	$10,000		
Total R.O.W.				$0				**$53,500**		
Pipe & Pump Station										
Pipe		LF	$0.00	$0	49,915	LF	$216.00	$10,781,640		
Highway Boring		EA	$0.00	$0	1	EA	$70,000.00	$70,000		
Pump Station		HP	$0.00	$0	1,745	HP	$2,855.00	$4,981,975		
Intake Structure		LS	$0.00	$0	1	LS	$620,000.00	$620,000		
Engineering		LS	$0.00	$0	1	LS	$1,645,000.00	$1,645,000		
S.I.O.H.		LS	$0.00	$0	1	LS	$987,000.00	$987,000		
Total Pipe & Pump Station				$0				**$19,085,615**		
Pipeline Costs				$0				**$19,139,115**		
Contingency @ 25%				$0				$4,784,779		
Total Pipeline Costs				$0				**$23,923,894**	**$23,923,894**	
Lake										
R.O.W.										
Land Cost		Acre	$0.00	$0	4,300	Acre	$1,500.00	$6,450,000		
Residential Relocation		EA	$0.00	$0	4	EA	$200,000.00	$800,000		
Acquisition		LS	$0.00	$0	1	LS	$400,000.00	$400,000		
Total R.O.W.				$0				**$7,650,000**		
Reservoir										
Infra. & Facility Reloc.		LS	$0.00	$0	1	LS	$3,875,000.00	$3,875,000		
Dam/Equip./Bldg.		LS	$0.00	$0	1	LS	$22,866,000.00	$22,866,000		
Recreation		LS	$0.00	$0	1	LS	$1,500,000.00	$1,500,000		
Engineering		LS	$0.00	$0	1	LS	$2,824,000.00	$2,824,000		
S.I.O.H.		LS	$0.00	$0	1	LS	$1,694,000.00	$1,694,000		
Total Reservoir				$0				**$32,759,000**		

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 4 - No Reallocation at Hulah & Construct Sand Lake

Case 4	Hulah Existing				Sand Lake				Case 4 Total	Case 4 Present Value
Pipe Size & Flow	Ex 2 @ 24" - 4.35 MGD				36" - 10.45 MGD					
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total		
Lake Costs				$0				$40,409,000		
Contingency @ 25%				$0				$10,102,250		
Total Lake Costs				$0				$50,511,250	$50,511,250	
Total Construction Cost				$0				$74,435,144	$74,435,144	$74,435,144
Energy Costs										
Base Charge	12	Mo	$50.00	$600	12	Mo	$50.00	$600		
Energy Charge (Yearly)	1,975,000	KWH	$0.02743	$54,174	5,703,000	KWH	$0.02743	$156,433		
Demand Charge (Yearly)	5,412	KW	$6.50	$35,178	15,624	KW	$6.50	$101,556		
Energy Cost, Year 1				$89,952				$258,589	$348,541	
Energy Cost, Year 50				$301,634				$867,120	$1,168,754	
Total Energy Cost				$8,768,912				$25,208,380	$33,977,292	$10,494,663
Pump Replacement										
Year 25	604	HP	$200.00	$120,800	1745	HP	$200.00	$349,000	$469,800	$142,927
Total Present Value										$85,072,734

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 5 - No Reallocation at Hulah & Kaw Pipeline

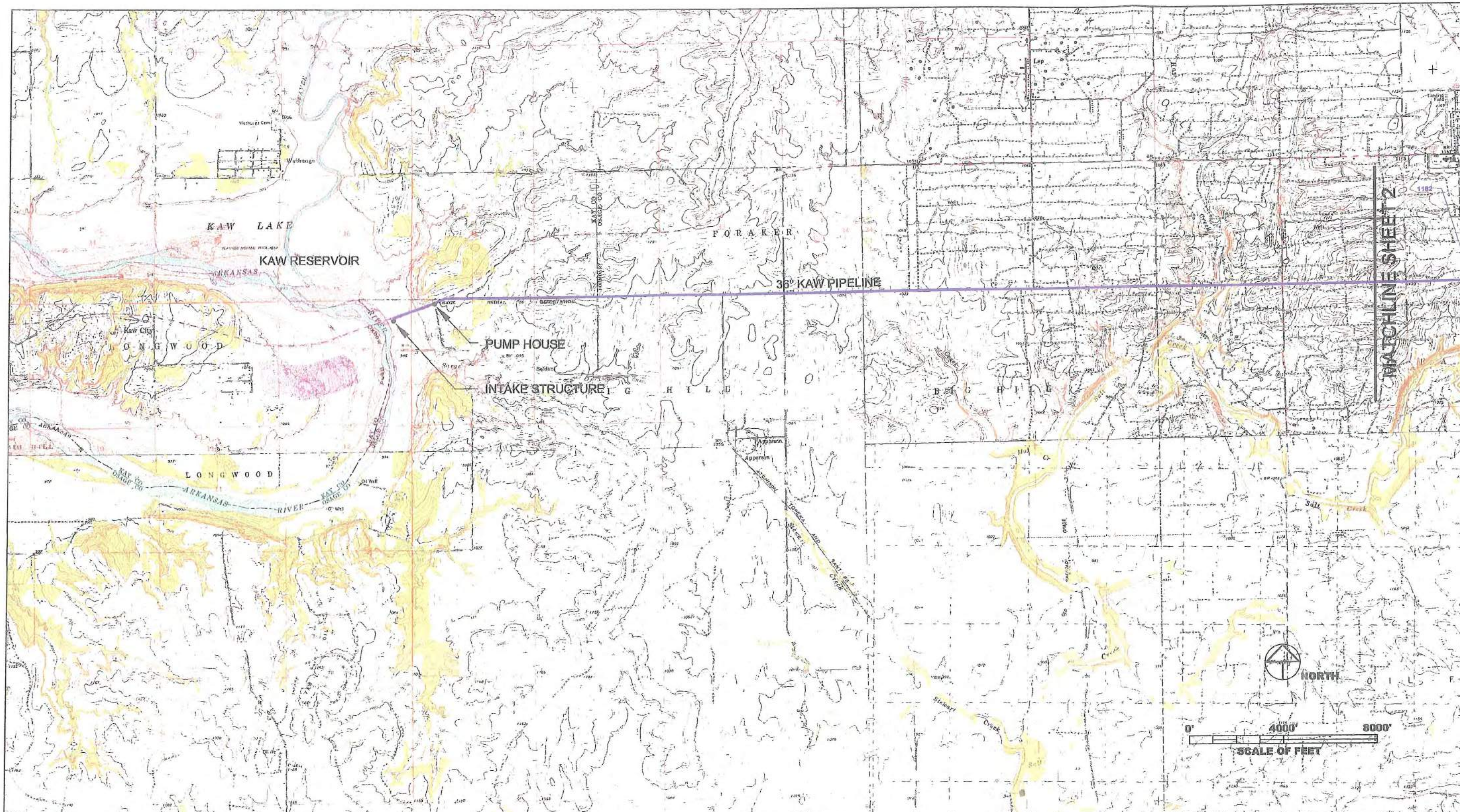
Case 5	Hulah Existing				Kaw Reservoir				Case 5	Case 5
Pipe Size & Flow	Ex 2 @ 24" - 4.35 MGD				36" - 10.45 MGD				Total	Present Value
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total		
Pipeline										
R.O.W.										
Land Cost		Acre	$0.00	$0	137	Acre	$1,500.00	$205,500		
Acquisition		LS	$0.00	$0	1	LS	$40,000.00	$40,000		
Total R.O.W.				$0				$245,500		
Pipe & Pump Station										
Pipe		LF	$0.00	$0	238,266	LF	$216.00	$51,465,456		
Highway Boring		EA	$0.00	$0	2	EA	$70,000.00	$140,000		
Pump Station		HP	$0.00	$0	2,547	HP	$2,855.00	$7,271,685		
Intake Structure		LS	$0.00	$0	1	LS	$620,000.00	$620,000		
Engineering		LS	$0.00	$0	1	LS	$5,950,000.00	$5,950,000		
S.I.O.H.		LS	$0.00	$0	1	LS	$3,570,000.00	$3,570,000		
Total Pipe & Pump Station				$0				$69,017,141		
Pipeline Costs				$0				$69,262,641		
Contingency @ 25%				$0				$17,315,660		
Total Pipeline Costs				$0				$86,578,301	$86,578,301	
Lake										
R.O.W.										
Land Cost		Acre	$0.00	$0		Acre	$0.00	$0		
Residential Relocation		EA	$0.00	$0		EA	$0.00	$0		
Acquisition		LS	$0.00	$0		LS	$0.00	$0		
Total R.O.W.				$0				$0		
Reservoir										
Infra. & Facility Reloc.		LS	$0.00	$0		LS	$0.00	$0		
Dam/Equip./Bldg.		LS	$0.00	$0		LS	$0.00	$0		
Recreation		LS	$0.00	$0		LS	$0.00	$0		
Engineering		LS	$0.00	$0		LS	$0.00	$0		
S.I.O.H.		LS	$0.00	$0		LS	$0.00	$0		
Total Reservoir				$0				$0		

Bartlesville Water Supply Study
Alternative Source Estimated Costs
Case 5 - No Reallocation at Hulah & Kaw Pipeline

Case 5	Hulah Existing				Kaw Reservoir				Case 5	Case 5
Pipe Size & Flow	Ex 2 @ 24" - 4.35 MGD				36" - 10.45 MGD				Total	Present Value
	Quantity	Unit	Cost	Total	Quantity	Unit	Cost	Total		
Lake Costs				$0				$0		
Contingency @ 25%				$0				$0		
Total Lake Costs				$0				$0	$0	
Total Construction Cost				$0				$86,578,301	$86,578,301	$86,578,301
Energy Costs										
Base Charge	12	Mo	$50.00	$600	12	Mo	$50.00	$600		
Energy Charge (Yearly)	1,975,000	KWH	$0.02743	$54,174	8,322,000	KWH	$0.02743	$228,272		
Demand Charge (Yearly)	5,412	KW	$6.50	$35,178	22,800	KW	$6.50	$148,200		
Energy Cost, Year 1				$89,952				$377,072	$467,024	
Energy Cost, Year 50				$301,634				$1,264,427	$1,566,061	
Total Energy Cost				$8,768,912				$36,758,618	$45,527,530	$14,062,218
Pump Replacement										
Year 25	604	HP	$200.00	$120,800	2547	HP	$200.00	$509,400	$630,200	$191,725
Total Present Value										$100,832,244

Bartlesville Water Supply Study - Unit Costs

Rates	Unit	Cost
Land Value	Acre	$1,500.00
Pipeline Costs		
30"	LF	$190.00
36"	LF	$216.00
Highway Boring	EA	$70,000.00
Pump Stations	HP	$2,855.00
Power Costs		
Base Charge	MO	$50.00
Energy Charge	KWH	$0.0274
Demand Charge	KW	$6.50
Pump Replacement	HP	$200.00
Inflation Rate		2.500%
Discount Rate		4.875%



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BARTLESVILLE WATER SUPPLY STUDY

BARTLESVILLE, OK
SCALE 1"=4000'

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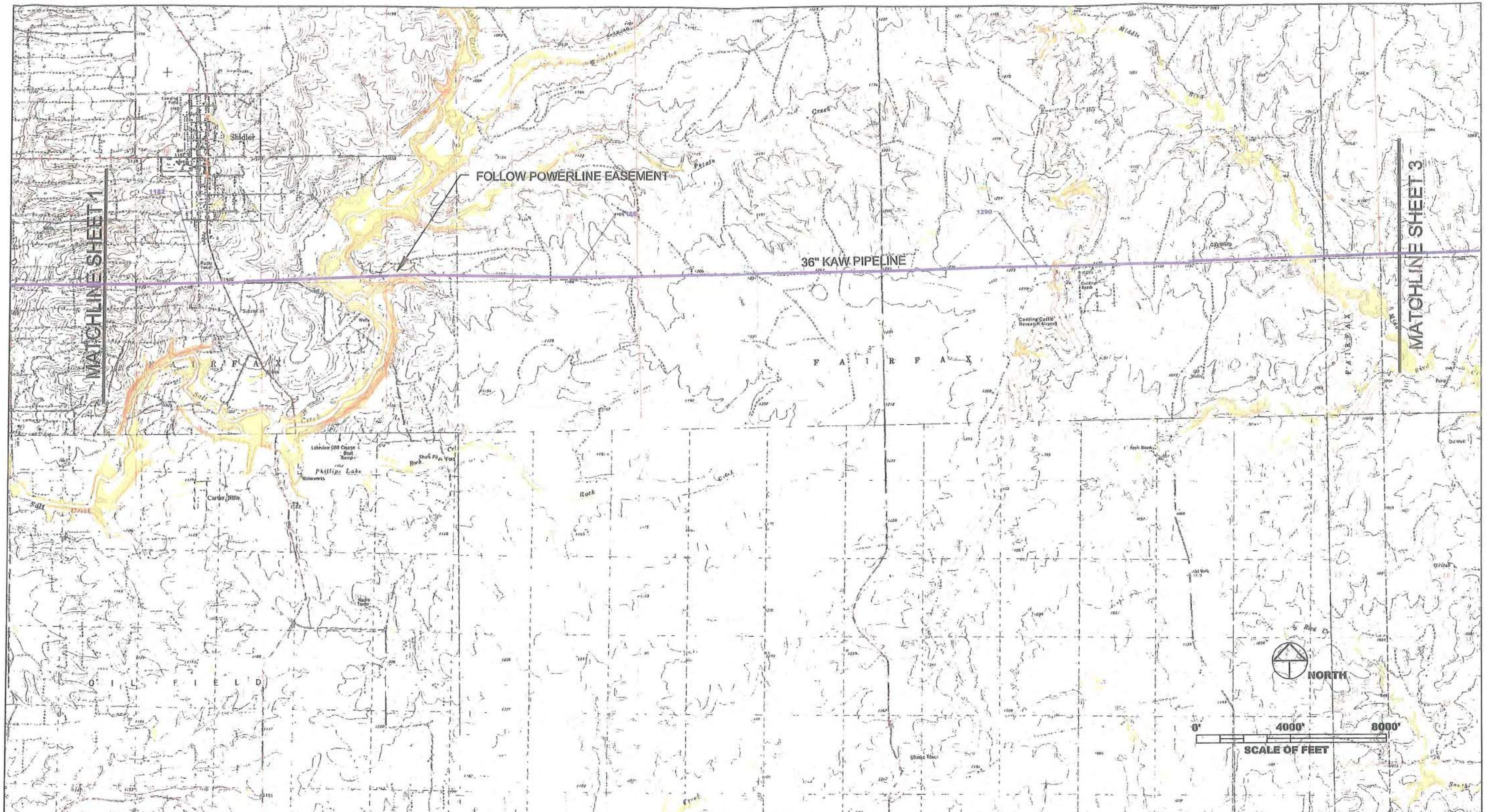
SHEET 1 OF 7



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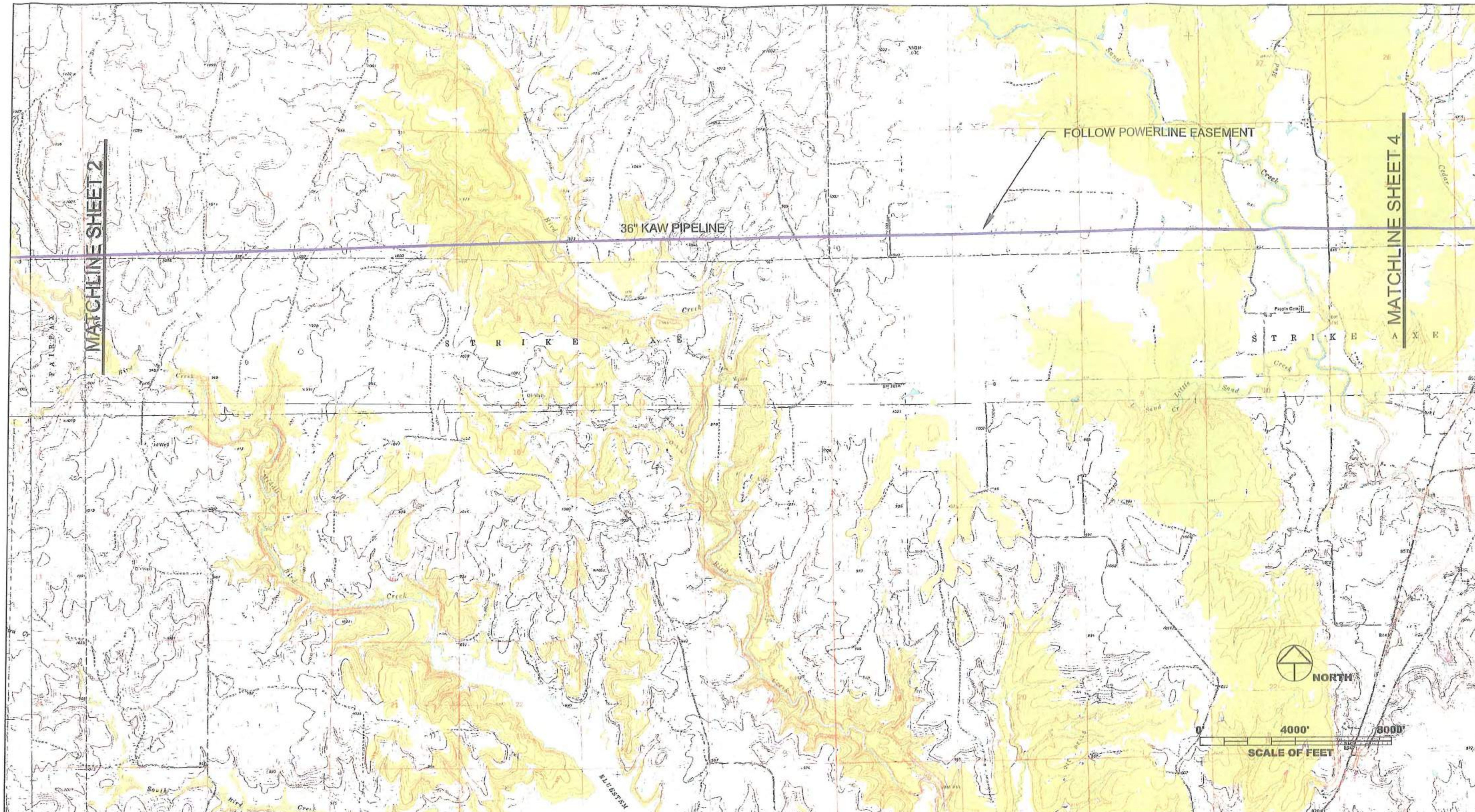

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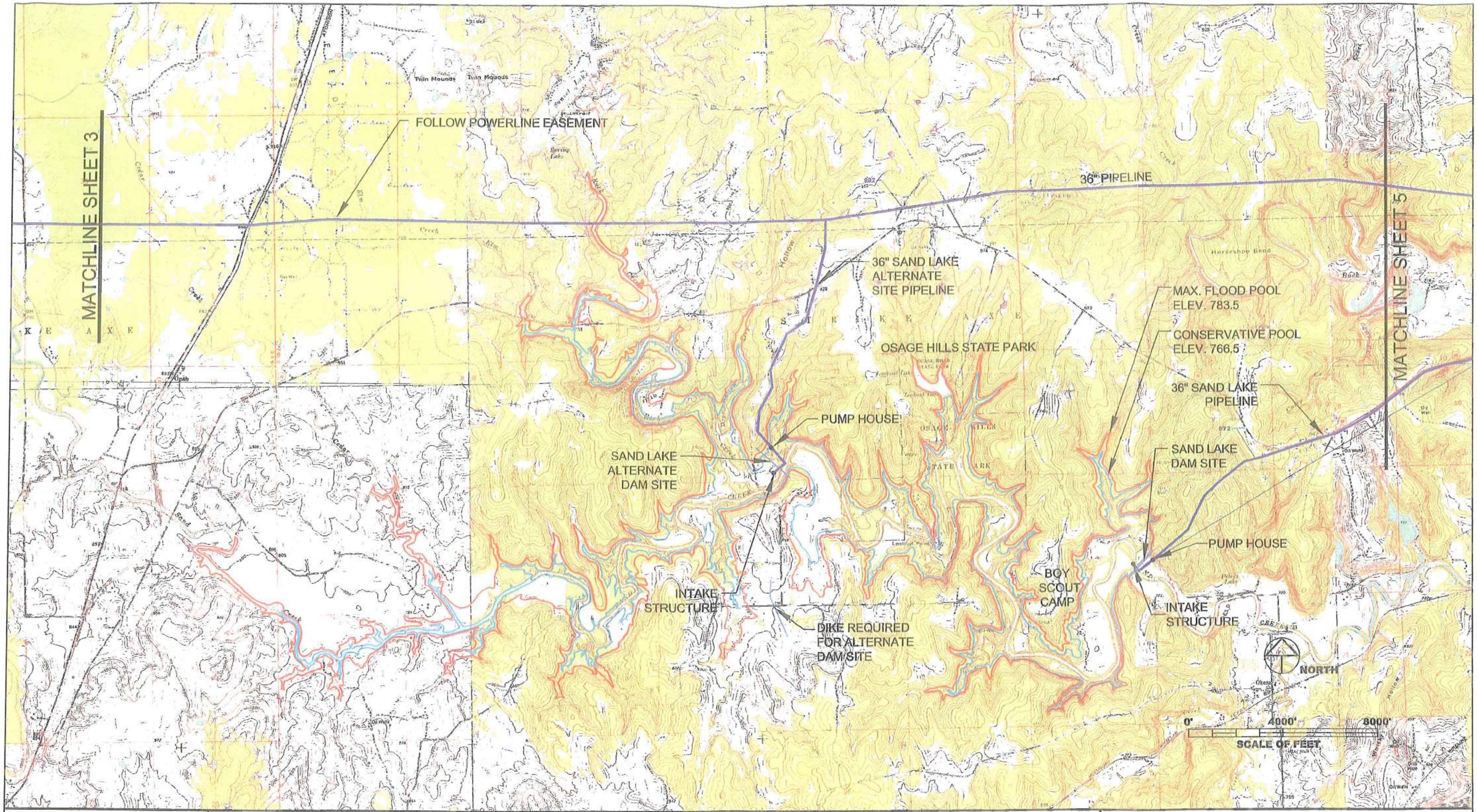

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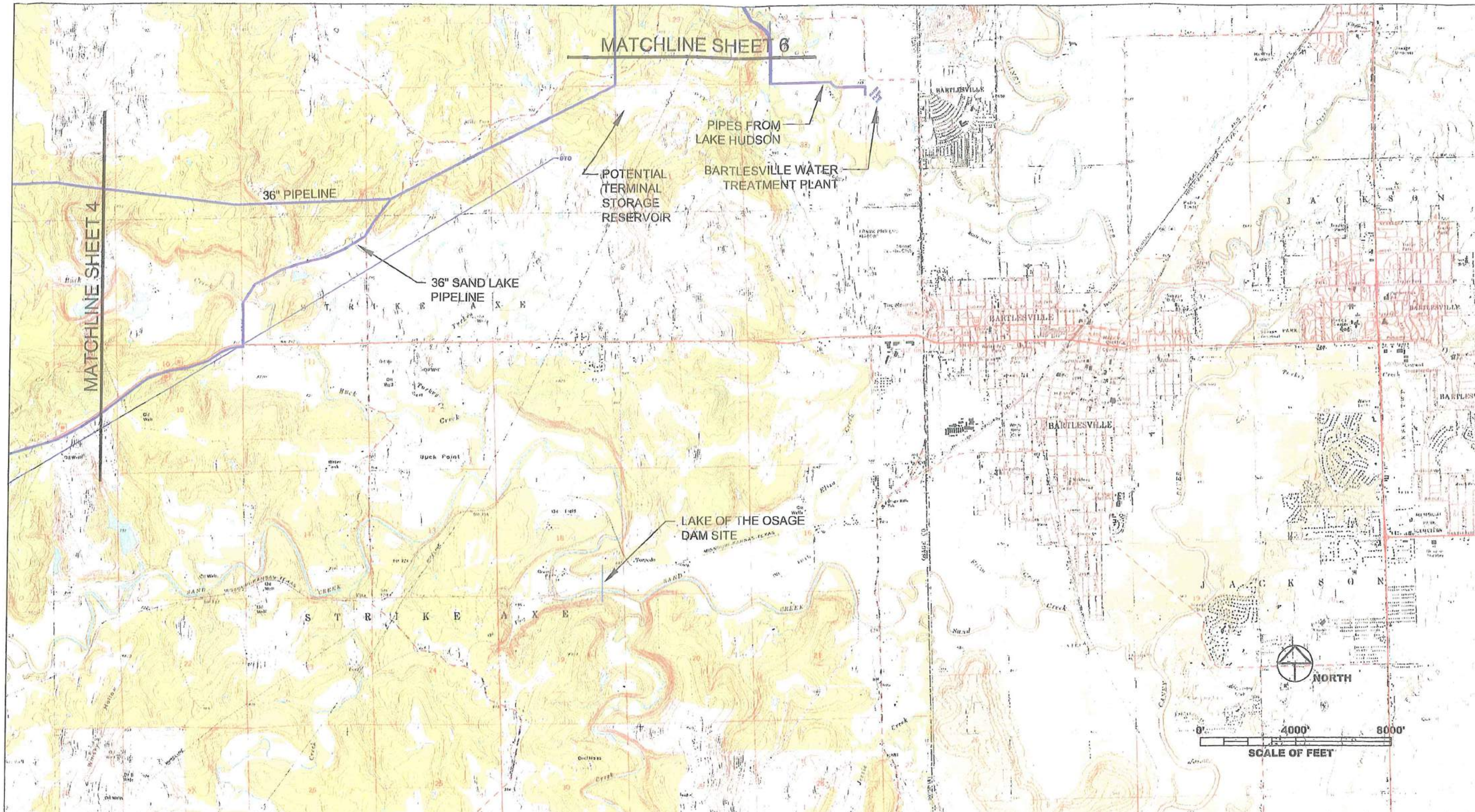

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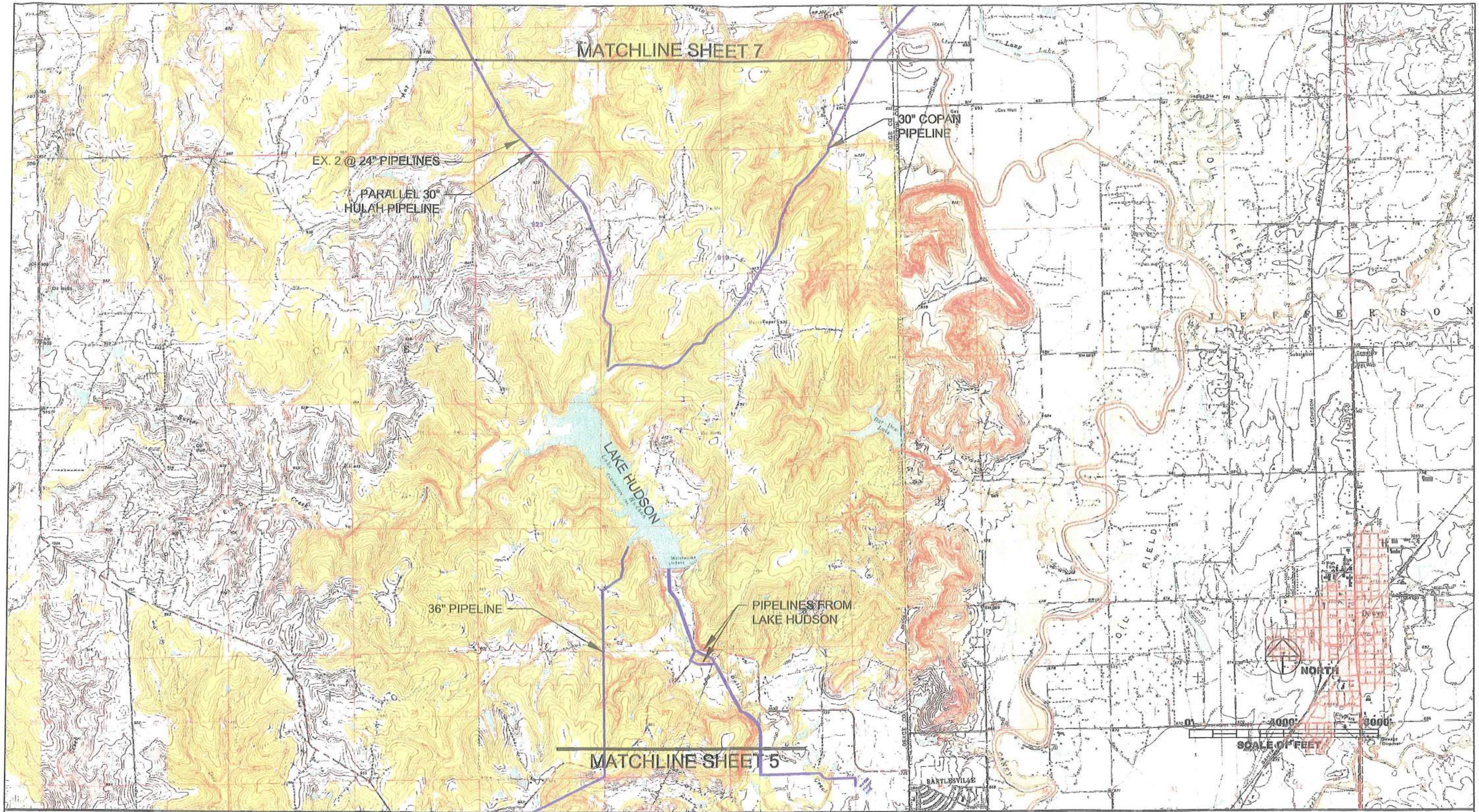

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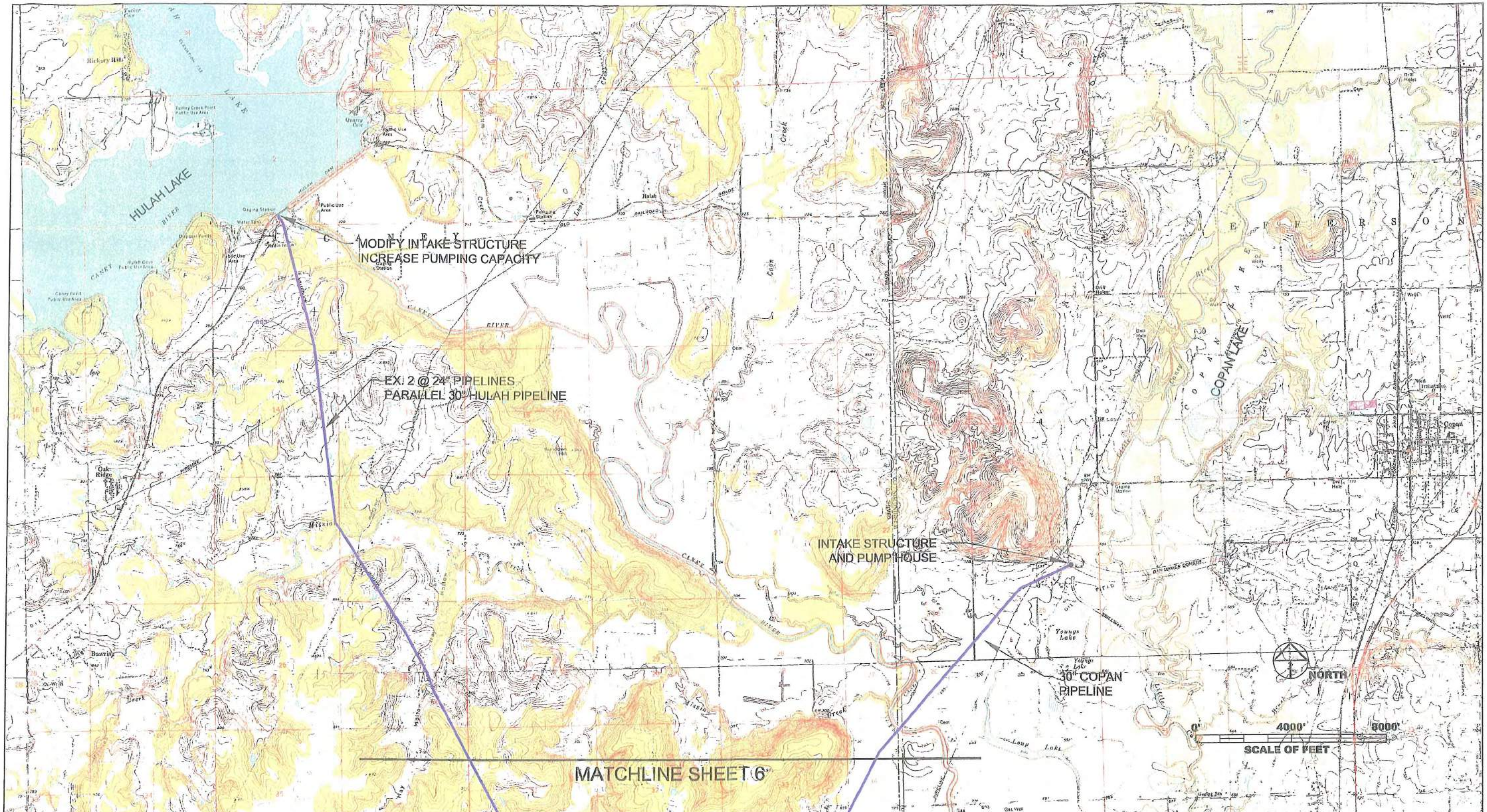

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APPENDIX E

Cultural Resource Analysis

APPENDIX E

Bartlesville Water Supply Alternatives

Cultural Resources Overview

Northeastern Oklahoma has shown evidence of human occupation dating from the Paleoindian Period through the Archaic, Woodland, Caddoan, Plains Village, and up through the Historic period. The study area that includes Copan and Hulah lakes in Washington County has recorded sites dating back to the Archaic Period in Oklahoma, and extensive evidence for Woodland and Plains Village occupation. These sites are in various settings around the Lakes and include camps, processing areas, habitation sites, trash dumps, rock shelters, and more. In addition, there is the likelihood for finding additional sites buried beneath alluvial soils and in areas that have not been surveyed.

Cultural Resources Impacts

The proposed alternatives for the reallocation at Hulah and Copan Lakes all have the potential to impact historic properties. Sections 106 and 110 of the National Historic Preservation Act (NHPA) of 1966 (as amended) require agencies to evaluate the impacts of federal undertakings on historic properties, which include prehistoric and historic archaeological sites, and historic standing structures. Section 106 requires the identification of all historic properties, which emphasizes an evaluation of eligibility for listing on the National Register of Historic Places (NRHP). Agencies must then determine which historic properties (those eligible for listing on the NRHP) will be adversely impacted. Sections 106 and 110 require that agencies resolve adverse effects to these properties. Plans for resolving adverse effects will be determined through consultation with the Oklahoma State Historic Preservation Office (SHPO), Oklahoma Archeological Survey (OAS), potentially the Advisory Council on Historic Preservation (ACHP), and appropriate and interested Native American tribes and other interested parties.

To fulfill the requirements outlined in Sections 106 and 110 of the NHPA, several tasks will require funding and execution within the feasibility phase of this project. In order to accomplish these tasks, the project area should be expanded to its fullest extent possible, so that design considerations can incorporate multiple variables, including cultural resources. Archaeological reconnaissance investigations, to include archival research, will be necessary to identify archaeological sites and standing structures that exist within the proposed project area. Each site and structure will require National Register evaluation; some will require sub-surface evaluation, detailed archival research or architectural documentation. NRHP-eligible sites and structures that will be adversely impacted by the undertaking will require mitigation, which will be determined through formal consultation with the SHPO and OAS, and potentially the ACHP. Mitigation requirements will be established in a Memorandum of Agreement (MOA).

Based on previous survey reports and cultural resources maps, there are several important sites in the impact area at both lakes, including a few sites related to the Delaware Big House religion at Copan Lake. The initial survey of Copan Lake by Rohn and Smith in December-January of 1971/72 was performed quickly and inhibited by bad weather and lack of access to many areas. No map of surveyed areas is available, and therefore it is unknown exactly what areas have been surveyed near the shoreline. The subsequent archaeological work at the lake in the following years focused on the excavation of river bottom sites that would be lost when the lake filled. The portion of the lake between the top of the current conservation pool (710 ft) and the top of the highest alternative raise (10% = 713.76ft) has not been thoroughly surveyed and should be investigated before a pool raise. This investigation would encompass up to 610 acres and could be a combination of shoreline survey by boat at the southern end of the lake, more intensive pedestrian surveys in the northern end of the lake where larger surface areas are affected, and possible re-examination of known sites in the impact area. Additionally, the impact to the historic properties in the area was not well assessed in the early investigations and this oversight will have to be amended in the proposed investigation.

Hulah Lake was built in 1946-1951 and was not surveyed for cultural resources until 1986. The survey at that time consisted of a random sampling strategy with additional areas included based on intuition and environmental potential for habitation. The survey encompassed 113.14 of 20,676 acres of project lands and 18.67 miles of 62 miles of shoreline (conservation pool). Both historic and prehistoric resources were included in this survey, and some preliminary recommendations on National Register eligibility were made for sites located in the survey area. Though many of the sites were not determined to be eligible, five sites were recommended for further testing to determine eligibility. The random nature of the sampling and the additional work to be done at Hulah would require additional survey of the area of potential impact, including shoreline survey by boat and pedestrian reconnaissance in larger areas of impact at the western and northern edges of the lake. The survey report estimated that based on their sample nearly 350 sites could exist on federal lands at the lake. The raise of the pool from 733 ft up to 739.46ft (the 10% alternative) would include approximately 800 acres of land of which a large portion appears to have not been covered by the previous survey.

Cultural Resources Investigation Costs

This is only an estimate for the purposes of this document and does not include the cost of mitigation if it becomes necessary. If the scope of the project changes the estimates will not be valid. Estimates are based on the current cost of work in Oklahoma in 2007 and will need to be amended if used in future planning work. Costs are based on a very broad, generalized view of the project and may vary based on contractor's research design.

The work that would need to be performed during the cultural resource investigations at Copan and Hulah would be generally the same at each lake. The variations would arise from the length of shoreline, the amount of acreage impacted by the pool raise, and the variations in the alternative pool raises being considered. Three alternatives were selected for further analysis in the discussion of the flood pool reallocation: #1) 5% reallocation at both lakes; #2) 10% reallocation at Hulah, none at Copan; #3) 1% reallocation at Hulah and 10% at Copan. Although option #3 was most favored based on water quality needs, all three options will be presented in this cost estimate to provide as much information as possible.

The initial fieldwork would be focused on identifying historic properties and cultural resources as defined in the NHPA. This would involve such actions as pre-field research, field reconnaissance, and report preparation and delivery. More in-depth work on identifying National Register eligible sites and assessing any adverse effects would be done at a later time after coordination with the appropriate agencies. Again, these estimates DO NOT include any possible mitigation costs. Table 1 is a breakdown of costs by reallocation option and project. It is based on current labor and overhead on cultural resources work in Oklahoma and on the following assumptions:

- Copan Lake has 30 miles of shoreline, 307 acres of impact at 5%, and 610 acres at 10%.
- Hulah Lake has 62 miles of shoreline, 100 acres of impact at 1%, 400 acres of impact at 5%, and 800 acres of impact at 10%.
- New sites will be discovered at each Lake
    - Hulah = 10, 20, and 40 sites per 1%, 5%, and 10% raise;
    - Copan = 20 and 30 sites per 5% and 10% raise
    - 10% of located sites will need testing for National Register Eligibility.
- These costs DO NOT include the additional investigations necessary if roads, facilities, or other lake amenities are relocated due to a pool raise. It is recommended that those relocations be planned in advance of the cultural resource investigations so that the cost may be added to the total and they may be all completed at one time.

Table 1: Cost Estimate Breakdown By Reallocation Option and Project

OPTION 1: 5% POOL RAISE EACH LAKE	COPAN	HULAH	TOTAL
Identify Historic Properties	150,000.00	150,000.00	300,000.00
Determine National Register Eligibility	100,000.00	100,000.00	200,000.00
OPTION 2: 10% POOL RAISE HULAH	**COPAN**	**HULAH**	**TOTAL**
Identify Historic Properties	0.00	200,000.00	200,000.00
Determine National Register Eligibility	0.00	160,000.00	160,000.00
OPTION 3: 1% POOL RAISE HULAH, 10% POOL RAISE COPAN	**COPAN**	**HULAH**	**TOTAL**
Identify Historic Properties	150,000.00	90,000.00	240,000.00
Determine National Register Eligibility	120,000.00	50,000.00	170,000.00

Other Alternatives

The two alternatives not discussed above are the construction of Sand Lake and the construction of a pipeline from Kaw Lake to Bartlesville. These alternatives are generally larger, costlier options that would entail a high level of effort to identify cultural resources in the area of potential effect. Each would have the potential to impact prehistoric and historic properties based on their size and locations. The area surrounding Kaw Lake has well-known archaeological sites related to the French and Wichita trading settlements that were located in the area, including the Deer Creek site which is listed as a National Historic Landmark. Investigations in these areas would include background research, full pedestrian survey with subsurface testing including backhoe trenching and/or coring, and testing for National Register Eligibility as described for the previous alternatives.

Assumptions made with each project are as follows:

- Sand Lake has 4,300 subject acres of land of which 3,216 would be affected by the lake at maximum elevation.
- For Sand Lake, all relocations of US-60, Osage Hills State Park structures, Boy Scout Camps, oil-field wells and pipelines, power lines, pump station, 36" pipeline to Lake Hudson, and other utilities ARE NOT included in this cost estimate and should be included in the initial planning for the cultural resources investigation to save on cost and time. These additions will likely increase the estimate depending on their location and extent.
- Possible location of up to 100 sites with testing of 10% for National Register Eligibility at Sand Lake, and 25 sites with 10% testing at Kaw Lake Pipeline.
- The Kaw Lake Pipeline will be a 36" pipe that extends 45 miles from the east side of Kaw Lake to Lake Hudson, assuming a 100 foot total easement = 545.5 acres.
- Any potential reservoir, pipeline, and water treatment facility that may be necessary depending on water quality issues or any relocation of other structures and utilities for the Kaw Lake Pipeline ARE NOT included in this estimate and should be included in initial planning as mentioned for Sand Lake.
- <u>None of the estimates for cultural resource investigations include the cost of mitigation should it become necessary</u>.

Cost Estimates for Sand Lake and Kaw Lake Pipeline

Sand Lake	=	$500,000 Identify Historic Properties $400,000 National Register Testing $900,000 Approximate Cost
Kaw Lake Pipeline	=	$125,000 Identify Historic Properties

$75,000 National Register Testing
$200,000 Approximate Cost

APPENDIX F

Downstream Flood Impacts From Flood Pool

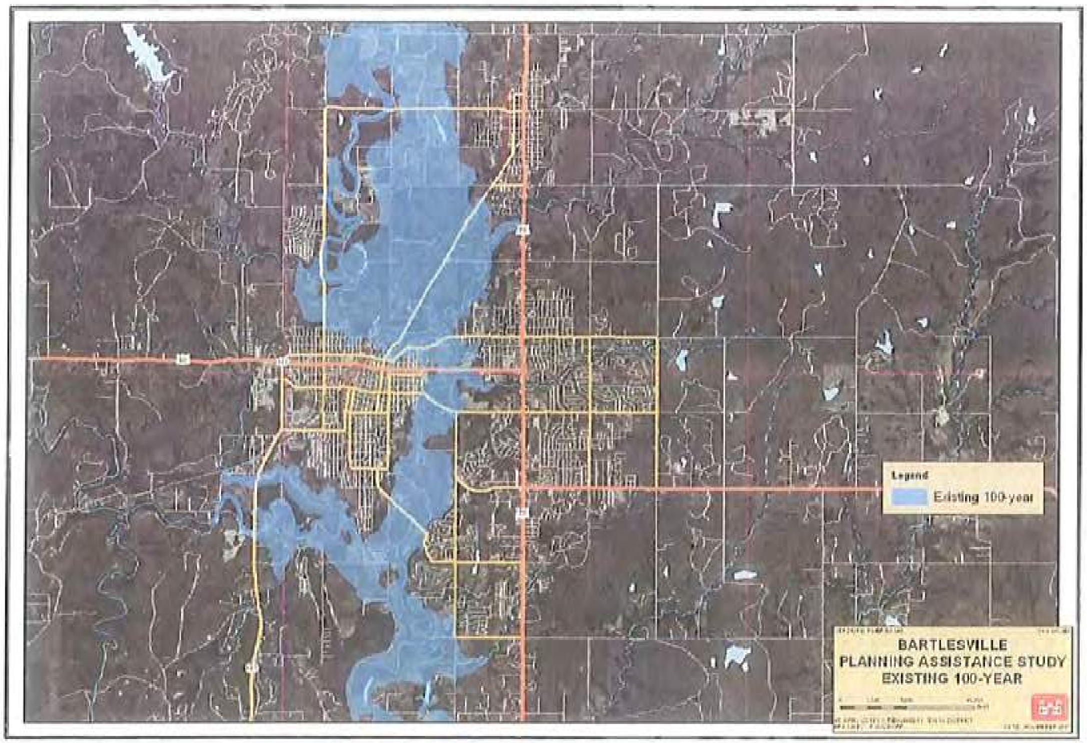
APPENDIX F

DOWNSTREAM FLOOD IMPACTS FROM FLOOD POOL REALLOCATION ALTERNATIVES e, f, k

There will be some minor downstream flood impacts due to any future reallocation of the flood pool to water supply storage. The level of impact to downstream flooding, is predicated on the reallocation alternative selected. The greatest difference of flood levels and flood duration (compared to existing flood conditions) will occur for the smaller frequency flood events. You will see slightly greater flood events for the 2, 5,10,25 and 50 flood events, and minimal if any difference in the greater storm events of 100, 250 years and above. The reallocations being proposed will have no measurable impact for these larger storm events.

For easy reference, Figure 3 below delineates the 100-year flood map through the City of Bartlesville. Economic projections estimate that for the three different reallocation scenario's 3a, 3b, and 3c, downstream flood damages would increase approximately $10,000 to $12,000 annually over a 50-year time period. Over a 50-year period, assuming a discount interest rate of 4 7/8%, approximately $200,000 in additional flood losses above current levels could be expected. Summarized below is the estimated flood damage increases that could be expected from a potential flood reallocation.

Plan Name	Description	Increased Annual Flood Damages Induced	Present Value of Additional Flood Damages - (50 years at 4-7/8%, 2007 Prices)
Existing	Existing Conditions	$ -	-
Plan 3a	5% Hulah, 5% Copan	$ 10,090	$188,000
Plan 3b	10% Hulah, Exist Copan	$ 11,920	$222,000
Plan 3c	1% Hulah, 10% Copan	$ 9,044	$176,000


Legend
Existing 100-year
BARTLESVILLE
PLANNING ASSISTANCE STUDY
EXISTING 100-YEAR

APPENDIX G

Sediment Protection Measures above Hulah and Copan

APPENDIX G

SEDIMENT PROTECTION MEASURES ABOVE HULAH AND COPAN

Reducing the existing sediment deposits and implementing sediment protection measures above Hulah and Copan Reservoirs is highly desirable. Based on existing sediment rates, the water supply yield at Hulah reservoir will decline to 4.35 million gallons per day (mgd) by year 2055 at the existing conservation storage elevation of 733.0. The water supply yield at Copan will also decline to 5.23 mgd by year 2055 at the existing conservation storage elevation of 710.00. Sediment deposits also reduce flood storage benefits that currently exist from both reservoirs. This study looked at potential sediment sources and outlined protection measures that could be encouraged above Hulah and Copan Lakes.

In Kansas just above Hulah and Copan Reservoirs, there are 7 conservation watershed districts which provide flood prevention and watershed protection, under the Federal PL-566 program. These programs help prevent sediment deposits into Hulah and Copan Reservoirs, which protects both water supply and flood storage. **Although not specifically addressed in this study, future joint venture water supply and flood protection initiatives could be explored with these upstream Conservation Watershed Districts.**

Grant-Schanghai, and Upper and Lower Caney watersheds are located above Hulah Reservoir. Bee Creek, Twin Caney, Middle Caney and Aiken Creek watersheds are situated above Copan Reservoir. All seven watershed projects have completed all measures planned to address flooding concerns in their respective drainage areas. Table 13 below lists the number of floodwater retarding structures (FRD) for each watershed district.

	TABLE 13	
Watershed District	No. of FRD's Complete	Multi-Purpose Structures (in addition to flood control)
Grant-Shanghai	7	0
Big Caney (Upper and Lower)	31	0
Bee Creek	7	0
Aiken Creek	1	0
Twin Caney	15	2 (1 recreation, 1 municipal water supply)
Middle Caney	15	1 (1 municipal water supply)

Each watershed has individual project maps that provide the location of each watershed structure. You can contact each individual watershed district listed above, or request these maps from the Natural Resource Conservation Service (NRCS) thru the local District Conservationist, Ronald Rader, Chautauqua County at Howard Service Center, 131 N. Wabash, Howard, KS 67349; Phone 620-374-2410 (or 2511) Email:ron.rader@ok.usda.gov. You can also view other watershed district contact information at:
http://scc.ks.gov/index.php?option=com_contact&catid=54&Itemid=141

Potential Sediment Sources and Protection Measures above Hulah and Copan Lakes

U.S. Geological Survey (USGS) Digital Elevation Model (DEM, USGS, 1999) and National Land Cover Database (NLCD, USGS, 2006) data were used to delineate watershed boundaries, stream channels, and land use/cover in the Hulah Lake and Copan Lake Watersheds.

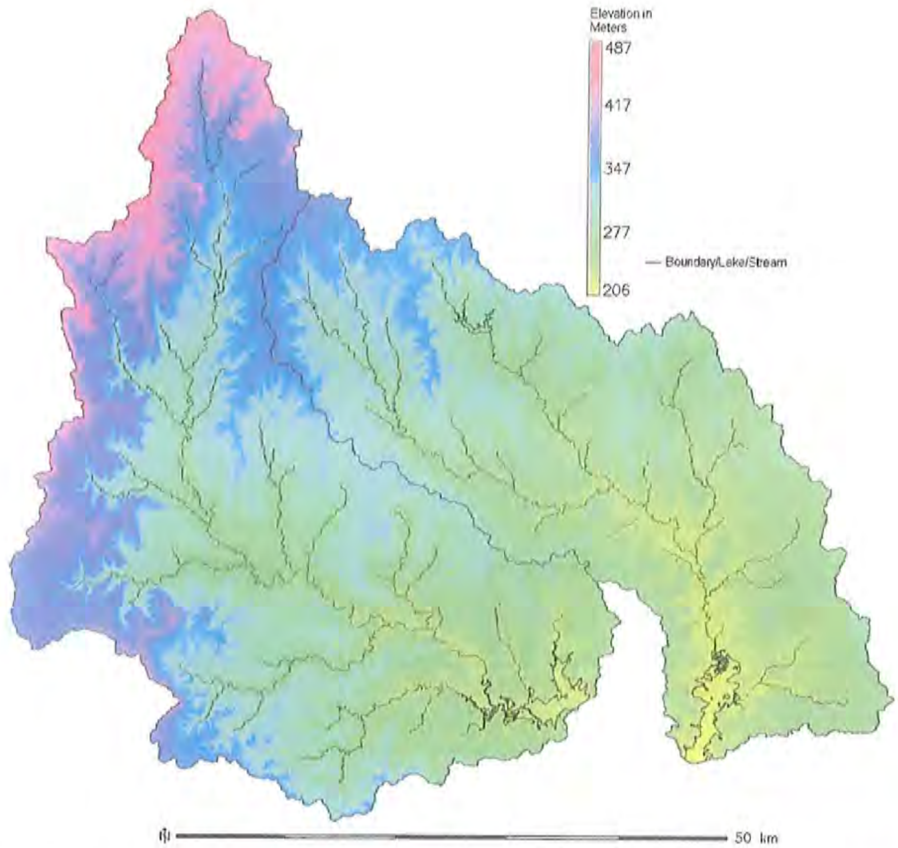


Figure 1. Elevation (in meters, NAVD88) from USGS NED. Locations of Hulah and Copan Lakes are indicated by black polygons in the lower portions of each watershed

Analysis of the digital elevation data included watershed delineation completed using Geographic Resources Analysis Support System software (GRASS 6.2, GRASS Development Team, 2006) and the 'watershed' script (GRASS Development Team, 2005). Drainage areas, slope, aspect, and drainage channel accumulation files were generated from the 30-meter resolution digital data (Figure 1). USGS NLCD data, developed from Landsat Enhanced Thematic Mapper Plus (ETM+) images collected between 1999 and 2002, also 30-meter resolution, was extracted for the defined watersheds to determine recent land use/cover classifications within each of the watersheds. Detailed data are pictured in Figure 2 and listed in Table 14 below.

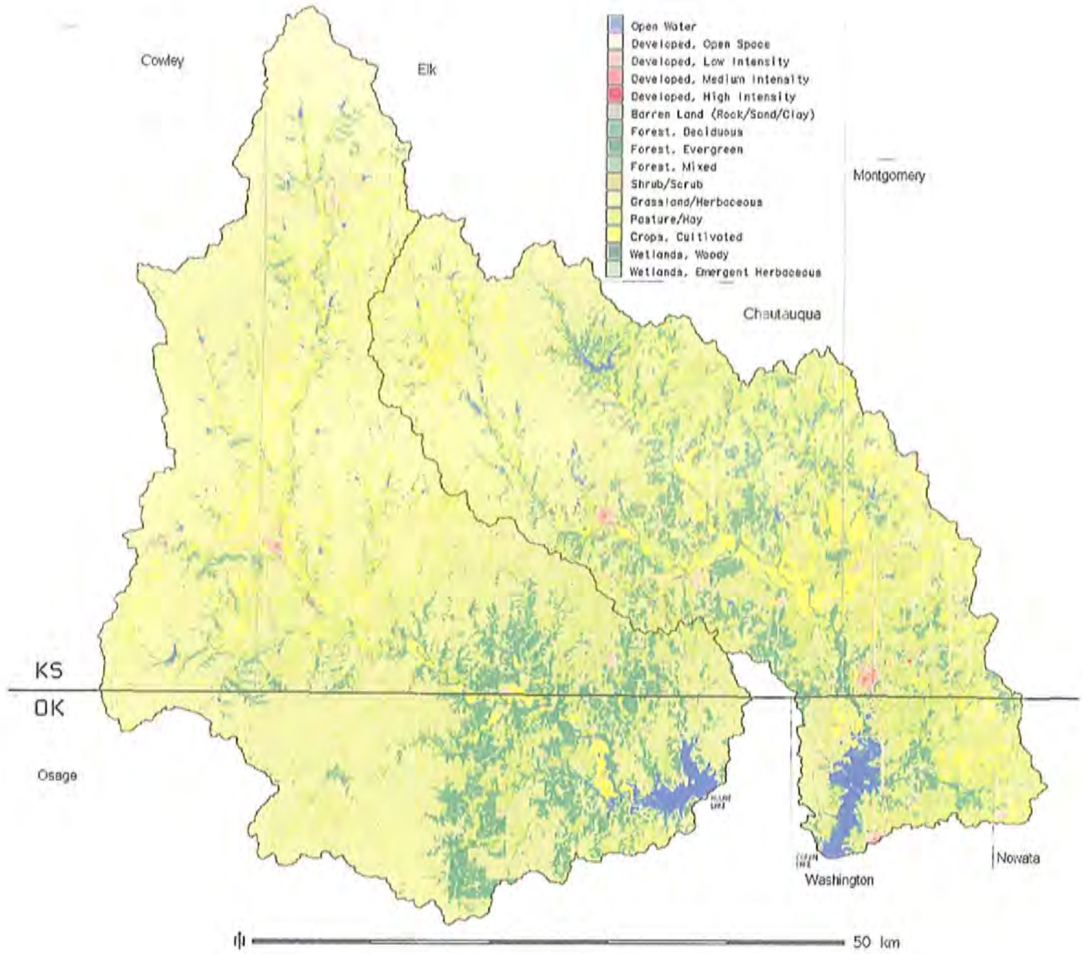


Figure 2. Land use/cover in the Hulah Lake and Copan Lake Watersheds from NLCD (USGS, 2006).

Based on the NLCD data, each watershed is dominated by grassland, pasture, and forest. The 455,570 acre Hulah Lake watershed is comprised of 66.3% grassland, 13.6% pasture, 12.0% forest, 3.5% developed (urban, residential, transportation, commercial, and industrial land uses), 3.0% cropland, 1.4 % open water (lakes, ponds, streams), 0.1% barren land (bedrock, surface mines, and gravel pits), and 0.1% wetlands. The 324,160 acre Copan Lake watershed is comprised of 43.4% grassland, 29.3% pasture, 14.9% forest, 5.4% cropland, 4.7% developed, 2.3% open water, and the remainder a combination of wetlands, shrub land, and barren lands.

Common upland sources of sediment delivered to streams and impoundments are croplands, overgrazed pasture and range lands, and unvegetated developed areas. Stream bank erosion and stream channel down-cutting are also potential sources of sediment.

Table 14. Land use/cover statistics derived from DEM (USGS, 1999) and NLCD (USGS, 2006) for the Hulah Lake and Copan Lake Watersheds.

LULC classes	Hulah Lake Watershed		Copan Lake Watershed	
	acres	%	acres	%
Open Water	6,152.64	1.35	7,398.03	2.28
Developed, Open Space	14,812.06	3.25	13,235.32	4.08
Developed, Low Intensity	1,226.04	0.27	1,828.71	0.56
Developed, Medium Intensity	89.85	0.02	224.61	0.07
Developed, High Intensity	7.78	0.00	68.72	0.02
Barren Land (Rock/Sand/Clay)	378.51	0.08	47.15	0.01
Forest, Deciduous	53,337.13	11.71	46,704.57	14.41
Forest, Evergreen	1,061.02	0.23	649.82	0.20
Forest, Mixed	374.06	0.08	193.03	0.06
Shrub/Scrub	89.18	0.02	147.44	0.05
Grassland/Herbaceous	302,131.72	66.32	140,788.66	43.43
Pasture/Hay	62,135.54	13.64	95,024.36	29.31
Crops, Cultivated	13,458.60	2.95	17,490.75	5.40
Wetlands, Woody	281.32	0.06	312.01	0.10
Wetlands, Emergent Herbaceous	32.91	0.01	43.14	0.01
Total	455,568.36	100.00	324,156.32	100.00

Additional analysis of land use within the two watersheds involved determining land use/cover within buffer zones of major drainage channels. A buffer zone of 500 meters around stream channels, developed from the DEM data, was generated and land use/cover within these zones was identified based on the NLCD data. The purpose of this exercise was to determine if 'managed' land uses within this zone are really significant in terms of potential sediment sources and contributions to the lakes. Assuming that forest, grassland, wetland, and water land use classes are either well-vegetated or insignificant sources of sediment from the land surface, an extraction of the acreage of developed, barren, pasture, and cropland was performed (Figure 3).

In the Hulah Lake Watershed, areas within 500 meters of stream channels and the lake include 3,680 acres classified as developed, 6,090 acres of cropland, and 20,325 acres classified as pasture/hay land use. In the Copan Lake Watershed, areas within 500 meters of stream channels and the lake include 3,195 acres classified as developed, 16 acres of barren land, 17,680 acres classified as pasture/hay, and 7,195 acres of cropland. Best management practices applied to these areas could potentially provide the greatest reductions of upland sources of sediment carried by storm runoff to streams and eventually to downstream reservoirs.

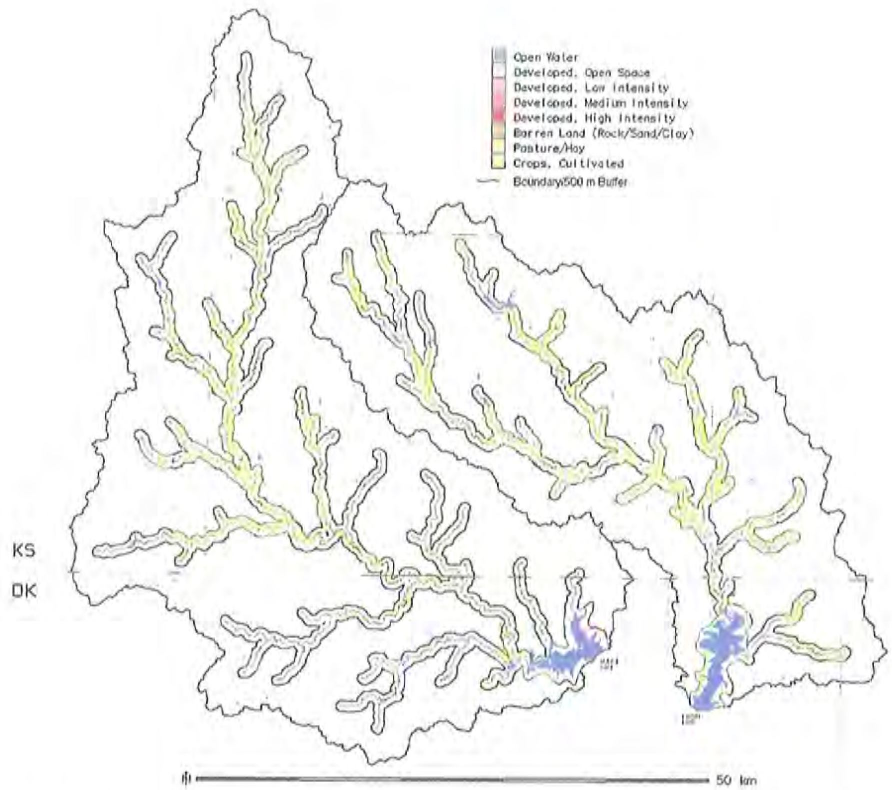


Figure 3. **Buffer zones (500 meters) around stream channels indicating the relative proximity of potential sediment contributing land use/cover classes to stream channels and the lakes, based on NLCD (USGS, 2006) land use data.**

Potentially effective Best Management Practices:

The following compilation of Best Management Practices (BMPs) have been found to be effective, to varying degrees depending on site specific criteria, in reducing runoff water velocity and erosion from the land surface. Detailed information on many applicable BMPs is available from a variety of sources including *Best Management Practices for Soil Erosion* by the Purdue Research Foundation (2001). Incentives for landowner implementation of some of these practices may be available through state and federal agencies. The list is not exhaustive. Cropping practices stress maintenance of vegetative cover during critical time periods and the primary objective is a reduction of soil erosion by decreasing soil particle detachment. Structural practices stress reduction in runoff water velocity enabling settling of heavier suspended particles before the water reaches stream channels and/or downstream impoundments.

Filter strips around croplands - Strips of closely-grown vegetation placed between field edges and water bodies or riparian areas to control sediment loss and erosion. The strip inhibits the transport of sediment by reducing storm runoff water velocity and allows sediment and adsorbed pollutants to drop out before reaching the stream or lake. Effective width is partially dependent on field size and drainage area and can vary from five to 100 meters.

Grassed waterways in drainages on croplands - Areas in croplands where storm water runoff channelizes are planted with a dense grass cover to reduce runoff velocity and prevent channel erosion.

Conservation tillage agriculture – Tillage practices utilizing non-inversion plowing techniques leave significant quantities of crop residue at or near the soil surface. Crop residue reduces soil erosion and storm runoff, and helps maintain soil moisture through the growing season.

No-Till agriculture – A form of conservation tillage where no tillage is used to establish the seed bed. Former crop residues remain at the soil surface and reduce potential soil erosion.

Residue management on cropland – Residue from former or cover crops is maintained at or near the soil surface. Landowners manipulate (maximize) the amount of residue remaining after crop harvest or/and plant close-growing cover crops between harvested crop growing seasons.

Contour farming – Cropland planted parallel to elevation contours to reduce runoff velocity.

Contour strip-cropping – Croplands planted parallel to elevation contours with different crops in parallel strips reducing runoff velocity.

Parallel Terraces (newly constructed or repaired/refurbished) – Graded terraces across the slope reduce the effective land slope and reduce runoff velocity.

Pasture and hay land management – A system of practices designed to protect vegetative cover on improved pasture or range land which includes seeding or reseeding, brush management, proper stocking rates and grazing use, and deferred rotational systems. Maintaining permanent land cover with high quality vegetation decreases soil erosion.

Riparian (re)Vegetation and protection zones – Vegetated areas along water bodies or drainage channels are maintained or enhanced and can filter both surface and subsurface flows.

Cattle exclusion from waterways – Excluding livestock from areas where grazing, trampling, and watering denude stream banks. The practice reduces deposition of fecal material in streams, turbidity caused by in-stream trampling, and erosion of denuded stream banks.

Wetland development and/or restoration – Development or enhancement of wetland areas where increased retention time of runoff allows for pollutant settling and utilization of nutrients by wetland vegetation.

Pond development and/or restoration – Development and restoration of ponds that function as collection areas of runoff from fields for storage and pollution control by stopping water flow and allowing heavier suspended particles to settle.

Grade stabilization structures – Structures used to control grade and gully-head in drainage channels (fields, pastures, etc.) that reduce water velocity of runoff.

Stream channel stabilization – Structural and vegetative methods to reduce stream bank erosion using riprap, concrete, wood, rock gabions, and/or vegetation to stabilize stream banks. Vegetative methods have the additional benefits of shading the stream leading to decreased water temperatures, and increases in floodwater storage and hydrologic assimilative capacity. Efforts to identify areas of concern are required to effectively focus expensive stabilization strategies.

APPENDIX H

Upstream Impacts from Reallocation Alternatives

APPENDIX H

UPSTREAM IMPACTS FROM REALLOCATION ALTERNATIVES

Hulah Lake Upstream Impacts:

Reallocation of the flood pool would look at all impacts that would occur because of the conservation pool raise. A reallocation of the conservation pool for water supply would require that water supply users pay for the costs required by the reallocation of flood storage which is financed 100% from federal funds to water supply which is financed 100% by non federal funds. For planning purposes these costs were roughly estimated based on a 1%, 5% and 10% reallocation of the flood pool to water supply. If a reallocation was pursued these costs would be more accurately refined for repayment by eventual water supply users.

Physical Upstream Replacement Cost Impact for 1% Reallocation at Hulah

For Hulah Reservoir, a 1% reallocation would raise the Conservation pool from elevation 733.0 feet to elevation 733.9 feet and would increase the conservation pool by about 80 acres. Initial observations indicate that only a few oil facilities would be impacted. Total estimated costs are estimated at about $100,000.

Physical Upstream Replacement Cost Impact for 5% Reallocation at Hulah

For Hulah Reservoir, a 5% reallocation would raise the Conservation pool from elevation 733.0 feet to elevation 736.7 feet and would increase the conservation pool by about 400 acres. Initial observations indicate the following items that would need to be addressed.

1. Raise 1 mile of road 7 feet that runs along side the Waterfowl Refuge – $500, 000

2. Skull Creek – Either abandon all of the facilities or relocate all of the facilities to higher ground. The entrance road to this park area will go under water and render the entire park unusable. If abandoned – Removal and cleanup of old sites, toilets, and facilities - $150,000

3. If relocated – 24 campsites - $3000 per site = $72,000;1 boat ramp - $50,000;
   1 group shelter - $50,000;1 water system (hook to rural water) - $500,000;
   2 sets of pit toilets – $10,000

4. Turkey Creek - Either abandon part of the facilities or relocate them to higher ground. One road in the middle of the park will go under water. If abandoned – removal and cleanup of old sites, toilets, and facilities - $150,000

5. If relocated – 10 campsites - $3000 per site = $30,000;1 set of toilets - $5,000

6. Rural water intake structure will need to be raised to higher ground. - $100,000

7. An estimated 300 acres of State Waterfowl Refuge that is normally not covered with water at elevation 733.0 might go under water. This action may require mitigation with the State of Oklahoma. $800 per acre = $240,000

8. Oil and gas field related facilities may be affected. The superintendant with the Osage Indian Tribe will have to be contacted for current data on active oil and gas wells, tank batteries, pipelines, and electric lines. This action may require tribal coordination and compensation. A $200,000 value was estimated but by operations and could be significantly more than projected.

9. One A&G lessee will loose usage of an estimated 50 acres of substandard prairie grass. A $500 adjustment to the lease was estimated by operations.

10. To meet needs of the National Environmental Policy Act (NEPA) an environmental Assessment will be required for any future reallocation alternative. This estimate does not include fish & wildlife mitigation requirements which could be defined through the NEPA process.

11. A cultural resource survey would be required with any future reallocation alternative. These costs are outlined below under Table 13.

Based on the above estimate, a 5% reallocation would require $2,307,000 in upstream physical replacements within the Hulah Reservoir.

Physical Upstream Replacement Cost Impact for 10% Reallocation at Hulah

For Hulah Reservoir, a 10% reallocation would raise the conservation pool from elevation 733.0 feet to elevation 739.5 feet and would increase the conservation pool about 800 acres. Initial observations indicate the following items that would need to be addressed:

1. Raise 2.5 miles of road 10 feet that run along side the Waterfowl Refuge and near Elgin, KS – $1,500, 000
2. Skull Creek – Either abandon all of the facilities or relocate all of the facilities to higher ground. The entrance road to this park area will go under water and render the entire park unusable.
3. If abandoned – Removal and cleanup of old sites, toilets, and facilities - $150,000
4. If relocated – 24 campsites - $3000 per site = $72,000; 1 boat ramp - $50,000; 1 group shelter - $50,000; 1 water system (hook to rural water) - $500,000; 2 sets of pit toilets – $10,000
5. Turkey Creek - Either abandon all of the facilities or relocate them to higher ground.
6. If abandoned – Removal and cleanup of old sites, toilets, and facilities - $150,000
7. If relocated – 20 campsites - $3000 per site = $60,000; 2 set of toilets - $10,000;
8. Rural water intake structure will need to be raised to higher ground. - $100,000
9. An estimated 1800 acres of State Waterfowl Refuge that is normally not covered with water at elevation 733.0 might go under water. This action may require replacement with the State of Oklahoma. $800.00 per acre = $1,440,000 or total relocation.
10. Oil and gas field related facilities may be affected. The superintendant with the Osage Indian Tribe will have to contacted for current data on active oil and gas wells, tank batteries, pipelines, and electric lines. This action may require tribal coordination and compensation. - $2,000,000 conservatively

11. One A&G lessee will loose usage of an estimated 150 acres of prairie grass. A $900 adjustment to the lease was estimated.
12. Dry land hunters will loose use of an estimated 3,000 acres of hunting land. This issue may have to be mitigated with the State of Oklahoma. $800 per acre = $2,400,000
13. To meet needs of the National Environmental Policy Act (NEPA) an environmental Assessment will be required for any future reallocation alternative. This estimate does not include fish & wildlife mitigation requirements which could be defined through the NEPA process.
14. A cultural resource survey would be required with any future reallocation alternative. These costs are outlined below under Table 13.

Based on the above estimate, a 10% reallocation would require $8,627,000 in upstream physical replacements within the Hulah Reservoir.

Physical Upstream Replacement Cost Impact for 5% Reallocation at Copan
For Copan Reservoir, a 5% reallocation would raise the Conservation pool from elevation 710.0 feet to elevation 712.0 feet and would increase the conservation pool by 307 acres. Initial observations indicate that there would be few replacement items within the flood pool that would need to be addressed. The only major item needing attention would be shoreline erosion issues in the park area. Shoreline erosion was estimated at $5,000 at Copan Reservoir.

Physical Upstream Replacement Cost Impact for a 10% Reallocation at Copan
For Copan Reservoir, a 10% reallocation would raise the Conservation pool from elevation 710.0 feet to elevation 713.76 feet and would increase the conservation pool by 610 acres. The 10% reallocation would require additional erosion control on the face of the Copan Dam in addition to the other shoreline erosion issues estimated at $5,000 above. The face of the dam is partially rip rapped. Additional material may have to be added above the first level of riprap to control erosion. Estimated cost is $500,000. The total estimated upstream physical replacement cost is estimated to be $505,000 for a 10% reallocation. These costs do not include required NEPA environmental assessment costs, as well as cultural resource survey costs, that would also be required with any new reallocation.

Appendix B

Caney River QUAL2K Scoping Model Near Bartlesville, Oklahoma

August 2016

Tetra Tech

Caney River QUAL2K Scoping Model near Bartlesville, Oklahoma

August 15, 2016

PREPARED FOR

City of Bartlesville
Utility Services
401 S. Johnstone Ave.
Bartlesville, OK 74003

PREPARED BY

Tetra Tech
One Park Drive, Suite 200
PO Box 14409
Research Triangle Park, NC 27709
Tel 919-485-8278
Fax 919-485-8280
tetratech.com

Pictured: low-head dam on the Caney River underneath Oklahoma State Highway 123 (ODOT, 2016)

TETRA TECH

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EXECUTIVE SUMMARY

The City of Bartlesville, Oklahoma has conducted a Facility Planning study for which projected population growth will call for an increase in water usage. Projections indicate 8.5 million gallons per day (MGD) of wastewater will need to be treated at the existing Chickasaw wastewater treatment plant (WWTP) north of the City. The Chickasaw WWTP currently discharges treated effluent at the plant location into the Caney River downstream of a low-head dam. Although the majority of potable water supplies for Bartlesville are sourced from upstream reservoirs, there is an existing water intake located upstream of the dam and WWTP. Given the projected growth and wastewater discharge demands of Bartlesville, the City seeks to expand effluent treatment at Chickasaw and potentially add a second discharge location upstream. The City is exploring options to include a second discharge point approximately five to seven miles upstream of the existing intake location, providing that the Caney River has the assimilative capacity to handle this new inflow. The purpose of this desktop analysis is to provide a preliminary evaluation of the impact of effluent discharge relocation and/or reallocation upstream along the Caney River.

The Caney River along the reach of interest for this model is impaired for biology based on the results of fish bioassessments in the context of the river's Fish and Wildlife Propagation (FWP) classification of Warm Water Aquatic Community (WWAC) (ODEQ, 2014). The key water quality standard for FWP for the Caney River related to assimilative capacity evaluations is dissolved oxygen (DO) concentration. A scoping-level QUAL2K model was setup to support preliminary DO modeling analysis of adding a discharge point approximately five to seven miles upstream of the current Chickasaw outfall.

Model results indicate a likelihood of assimilative capacity along the Caney River to support a secondary effluent discharge location, although the scale of the capacity will require further field surveys and modeling to reduce existing uncertainty for key modeling assumptions. The preliminary results suggest that the Caney River may be capable of assimilating between 1.19 and 4.53 MGD of effluent at existing waste load allocation (WLA) limits when discharged five miles upstream of the intake. The model is quite sensitive to the prescribed reaeration model, with some sensitivity as well to various DO-related parameters such as carbonaceous biochemical oxygen demand (CBOD) decay rate, sediment oxygen demand (SOD), and net photosynthesis/respiration rate.

For the base QUAL2K model setup, this scoping analysis found that an additional discharge of 1.55 MGD located five miles upstream of the water supply intake could meet the summer period DO standard including a 5 percent margin of safety, and only result in 9 percent and 3 percent decreases in DO levels at the intake and above the existing WWTP respectively. For perspective, an effluent flow of 4.53 MGD at five miles upstream of the intake produces a 24 percent decrease in DO concentration at the intake and a 10 percent decrease in DO above Chickasaw. The true scale of assimilative capacity of the Caney River will depend on additional monitoring and modeling, but the weight of evidence from the QUAL2K model scoping analysis indicates a reasonable potential for assimilation of some additional flow above the existing Chickasaw outfall and supplemental water supply intake for the City of Bartlesville.

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1.0 BACKGROUND

1.1 PROJECT

The City of Bartlesville in northeastern Oklahoma has a population of approximately 35,750 (Census, 2010), and falls mostly within Washington County with a small portion located in Osage County. Bartlesville is bisected by the Caney River which flows south, joining the Verdigris River northeast of Tulsa. Bartlesville wastewater is processed through the Chickasaw Wastewater Treatment Plant (WWTP) north of the city, which discharges into the Caney River. Population growth projections for 2020 to 2050 have estimated future effluent discharge for Bartlesville at approximately 8.5 MGD total. Tetra Tech is supporting the City in conducting a facilities planning process to identify the optimal way to meet this future need.

The goal of this project is to provide a scoping-level desktop model analysis of the potential for using available assimilative capacity of the Caney River upstream of the existing Chickasaw WWTP. From a regulatory perspective, the Oklahoma Department of Environmental Quality (ODEQ) emphasizes modeling of dissolved oxygen (DO) kinetics to evaluate assimilative capacity. Previous studies (Tetra Tech, 2003; 2004; 2011) have estimated the maximum allowable effluent volume that can be assimilated successfully in the Caney River segment immediately downstream of the Chickasaw WWTP (i.e., to meet the water quality standard for DO for summer critical conditions) to be 3.97 MGD assuming effluent limits of 10 mg/L BOD_5 and 1 mg/L NH3 with an effluent DO of 6.0 mg/L. Given the population projection, Bartlesville seeks to explore the assimilative capacity of the Caney River to receive any or all of the remaining 4.53 MGD of effluent upstream. A number of scenarios were identified in which the Chickasaw WWTP discharges more effluent approximately five to seven miles upstream of an existing water intake located upstream of the OK-123 bridge north of the city. Bartlesville currently obtains potable water supplies from Hulah Lake and Hudson Lake north of the city, but the existing intake on the Caney River is used sparingly. If WWTP effluent discharge is feasible upstream of this intake in a capacity that would allow for potable reuse of water, that would be desirable for the City.

This feasibility analysis will provide a preliminary evaluation on the impact of effluent discharge relocation or reallocation between two discharges along the Caney River (the existing WWTP and a single additional upstream location). A scoping-level QUAL2K model was developed for this effort, which is built using available data for the area. Because this project is a scoping-level, the model was not calibrated to specific in-stream water quality or flow data. The results of this scoping-level analysis of stream assimilative capacity will be used to inform facility planning and decision making processes for the City of Bartlesville and ODEQ, including the potential need for and benefit of additional monitoring and calibrated modeling.

1.2 RECEIVING WATER AND WATER QUALITY STANDARDS

The Caney River flows from southern Kansas down to the Verdigris River south of Bartlesville. Two major reservoirs, Hulah Lake and Copan Lake, have a significant impact on flows in the Caney River near Bartlesville. The Caney River along the reach of-interest for this model (Waterbody ID 121400020010) is impaired for biology based on the results of fish bioassessments in the context of Fish and Wildlife Propagation (FWP) classification of Warm Water Aquatic Community (WWAC) (ODEQ, 2014).

Downstream of Bartlesville, the Caney River is also impaired for enterococcus, turbidity, and lead although the downstream extent of the Caney is not within the scope of this project.

From a regulatory perspective, a key water quality parameter of interest for assimilative capacity to support fish communities is DO. Additional parameters associated with DO need to be considered in the evaluation of assimilative capacity. In general, DO concentration increases at various rates due to plant photosynthesis and natural reaeration processes, and decreases due to fast carbonaceous biochemical oxygen demand (CBOD), nitrification, and plant respiration.

Numerical water quality criteria for DO are outlined by the Oklahoma Water Resources Board based on protection of beneficial uses (OWRB, 1991). For WWAC-classified rivers and streams, there are different criteria for DO at different times of year:

- Fishery Class: Early Life Stages (April 1 – June 15): minimum DO criteria is 6.0 mg/L for seasonal water temperature of 25 °C
- Fishery Class: Summer Conditions (June 16 – October 15): minimum DO criteria is 5.0 mg/L for seasonal water temperature of 32 °C
- Fishery Class: Winter Conditions (October 16 – March 31): minimum DO criteria is 5.0 mg/L for seasonal water temperature of 18 °C

Due to natural diurnal DO fluctuations, a 1.0 mg/L DO concentration deficit is allowed, although not for more than eight hours during any twenty-four hour period.

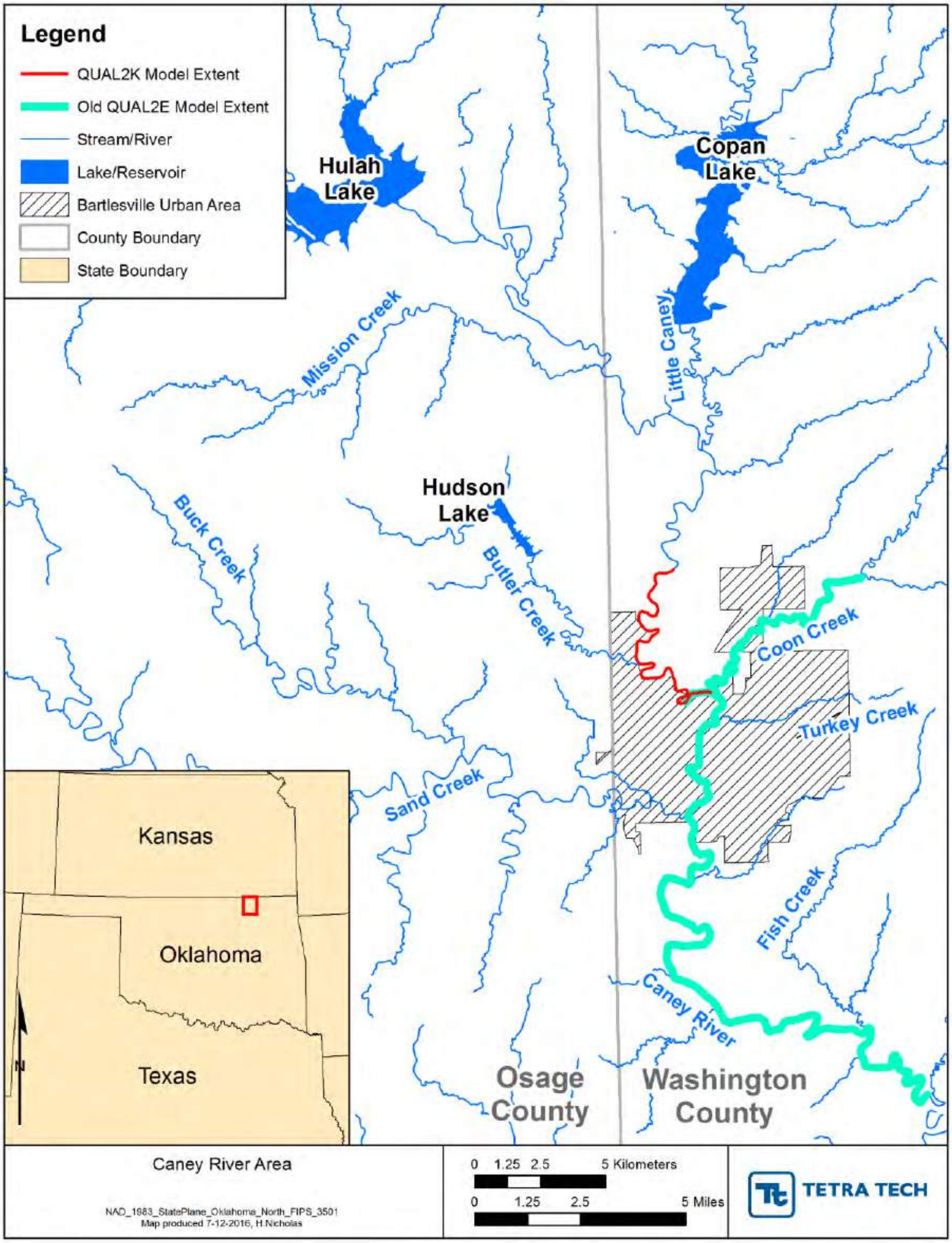

Figure 1. Caney River area and Bartlesville, Oklahoma area

2.0 PREVIOUS QUAL2E MODELING EFFORT

A project was undertaken in the early 2000s for the City of Bartlesville in order to evaluate the feasibility of expanding the Chickasaw WWTP discharge and/or building a second WWTP (Tetra Tech 2003; 2004; 2011). The Total Maximum Daily Load (TMDL) and Waste Load Allocation (WLA) efforts resulted in an estimation that the maximum allowable discharge at the existing WWTP was 3.97 MGD based on effluent concentrations of 10 mg/L BOD_5, 1 mg/L NH_3, and 6 mg/L DO. The quasi-dynamic QUAL2E model was used to estimate the sag point (low DO concentration) downstream of the local WWTPs (Chickasaw and Dewey) and provide a WLA to inform development of a TMDL for the existing Chickasaw facility.

Support for the TMDL and WLA project involved building a QUAL2E model which was a precursor to the QUAL2K model (Brown and Barnwell, 1987). This model was used to assess the assimilative capacity of the Caney River downstream of the Chickasaw WWTP, and included two model reaches which overlap with the area of interest of this current project. Field surveys, water quality sample data, and model parameterization and assumptions used for the QUAL2E model development were drawn upon for parameterization of the new QUAL2K model. The QUAL2E model was calibrated to critical low flow conditions on September 2, 2003. The total model extent was 36 miles (58 kilometers) long.

There were some issues with laboratory analyses in 2002, resulting in unreliable or unusable water quality sampling data associated with chlorophyll a, nitrate, NH_3, and long-term BOD. Despite some field sampling data issues, the modeling team was able to support a WLA for expanded discharge based on the combined monitoring and QUAL2E modeling effort. Following a series of written correspondence, meetings and calls with ODEQ in 2011, ODEQ provided resolution of concerns by applying a 7 percent margin of safety (MOS) to the final model allocations rather than the more traditional 5 percent MOS for calibrated model applications (letter from Mark Derichsweiler to Mike Hall, January 20, 2012). Therefore, the current QUAL2K scoping-level model was built upon the model results and parameterization established from this initial low flow WLA effort as a starting point.

3.0 QUAL2K MODEL SETUP

3.1 MODEL DETAILS

The QUAL2K model is a one-dimensional steady-state river water quality model (Chapra et al., 2012). QUAL2K was developed as a modernized and updated version of QUAL2E, the platform used for the previous Caney River modeling work (Brown and Barnwell, 1987). QUAL2K assumes well-mixed stream channels (both vertically and laterally), and can employ a diel, or 24-hour period, heat budget. The model interface operations are programmed in the Microsoft Office macro language Visual Basic for Applications (VBA) and is set-up and run using Excel.

The quasi-dynamic model developed in the early 2000s was created using QUAL2E. The list below provides some of the new elements which are found in QUAL2K and were not functions in QUAL2E:

1. Model segmentation allows for element size to vary from reach to reach
2. Point sources may be input to any single model element
3. Carbonaceous BOD speciation between the slow oxidizing form (slow CBOD) and the rapidly oxidizing form (fast CBOD)

4. Accommodation of anoxic conditions when applicable
5. Sediment-water interactions (such as fluxes of DO and nutrients) may be simulated
6. Bottom algae is explicitly simulated
7. Reach-specific kinetic parameterization is permitted
8. Weirs and waterfalls may be modeled explicitly, including associated hydraulics and gas transfer

The aforementioned QUAL2E model has two headwater reaches (QUAL2E model reaches 1 and 2) which correspond/overlap with the most downstream extent of this QUAL2K model. Parameterization for these two reaches will inform many of the assumptions associated with the QUAL2K model, although many new assumptions and parameterization choices were made based on the usage of this more robust model.

3.2 MODEL SEGMENTATION

QUAL2K model segments are called “reaches”, and represent lengths of stream for which hydraulic parameters are consistent. The full extent of the model is 7.85 miles (12.63 kilometers) of the Caney River from the crossing of W 1500 Rd down to the confluence with Coon Creek downstream of the Chickasaw WWTP. The mainstem of the river was segmented at key points of interest, such as road crossings, tributary and wastewater inflow points, and key hydraulic features such as the low-head dam under the OK-123 bridge. Each reach is broken up into computational “elements” within the model for hydraulic calculations, and physical parameters associated with upstream and downstream ends of each reach are used to inform the model domain (**Table 1**, **Figure 2**).

Note that model setup and results are presented in English and Metric units because the model uses Metric units, but English units may be more accessible for reference.

Table 1. Reach segmentation for Caney River QUAL2K scoping model

Reach	Description	Reach Length, mi (km)	Number of Elements	Upstream Location, mi (km)	Downstream Location, mi (km)	Upstream Elevation, ft (m)	Downstream Elevation, ft (m)
1	W 1500 Rd headwater reach	0.86 (1.39)	14	12.63 (7.85)	11.24 (6.98)	198.87 (123.57)	198.83 (123.55)
2	Headwater reach to W Durham Rd	0.60 (0.96)	10	11.24 (6.98)	10.28 (6.39)	198.83 (123.55)	198.79 (123.52)
3	W Durham Dr to Durham Rd	1.37 (2.20)	22	10.28 (6.39)	8.08 (5.02)	198.79 (123.52)	198.65 (123.43)
4	Durham Rd to development	0.58 (0.94)	9	8.08 (5.02)	7.14 (4.44)	198.65 (123.43)	198.61 (123.41)
5	Above Butler Tributary	0.58 (0.94)	9	7.14 (4.44)	6.20 (3.85)	198.61 (123.41)	198.57 (123.38)
6	Below Butler Tributary	1.59 (2.56)	26	6.20 (3.85)	3.64 (2.26)	198.57 (123.38)	198.53 (123.36)
7	USGS Gage 07174400 Segment	1.37 (2.20)	21	3.64 (2.26)	1.44 (0.89)	198.53 (123.36)	198.49 (123.33)
8	OK-123 Low-Head Dam*	0.01 (0.01)	1	1.44 (0.89)	1.43 (0.89)	198.49 (123.33)	198.49 (123.33)
9	Below Low-Head Dam to WWTP	0.50 (0.81)	8	1.43 (0.89)	0.62 (0.39)	198.49 (123.33)	194.76 (121.02)
10	WWTP to Coon Creek confluence	0.39 (0.62)	6	0.62 (0.39)	0.00 (0.00)	194.76 (121.02)	194.29 (120.73)

*Note that Reach 8 consists of a single element because that is a requirement to be modeled as a weir.

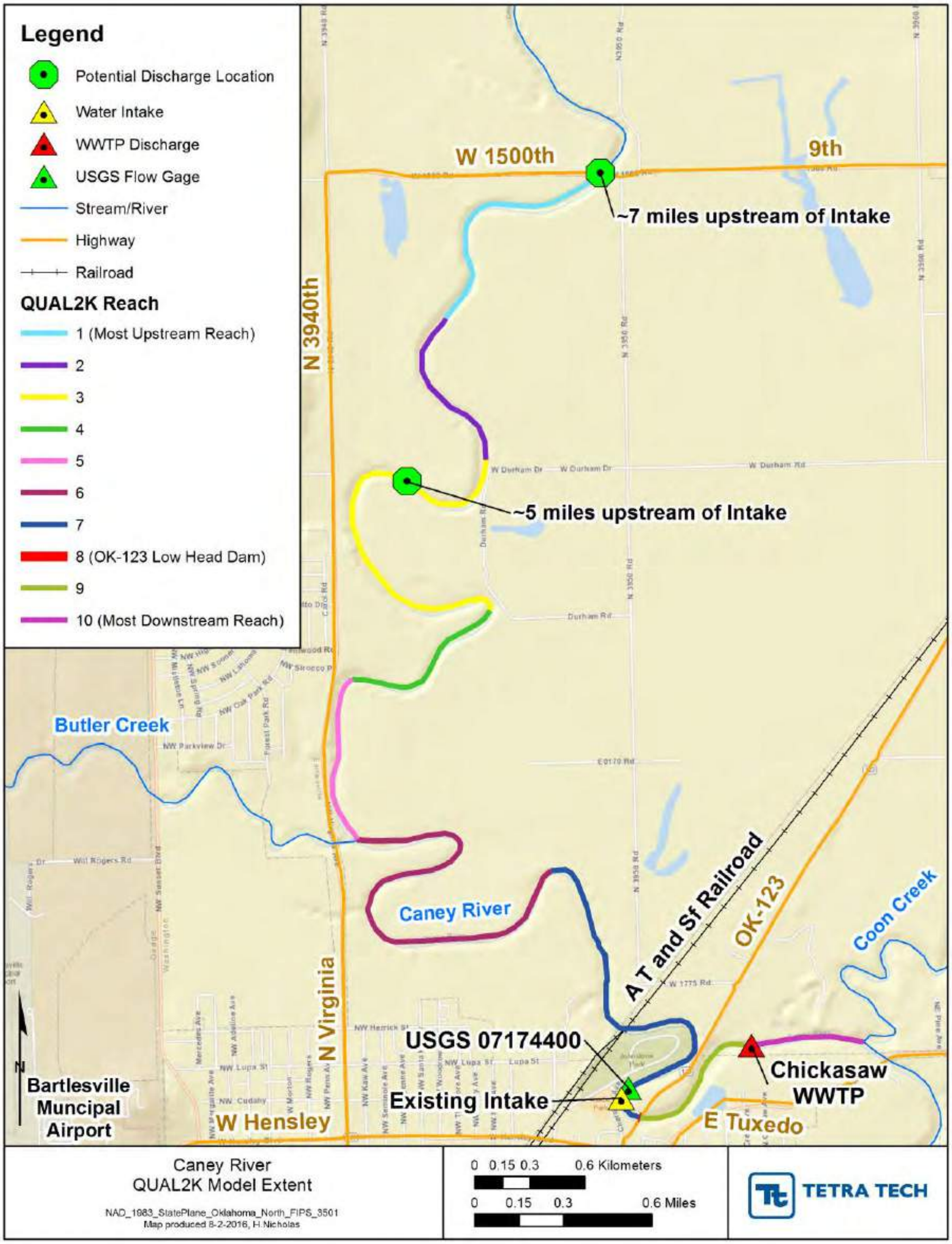

Figure 2. Model segmentation for Caney River QUAL2K scoping model

3.3 MODEL INPUTS

The model is set up to represent critical summer conditions (i.e., low flow and high temperature). Because lowest flows tend to occur in September in this area, and highest temperatures tend to occur in July and August, the representative critical date chosen for the model run is 7/21/2011. The model date was selected based on flow conditions recommended by DEQ and detailed below in “Flow Gaging”. The calculation step is set to the model-suggested 0.00293 hours to achieve stability, with a model run period of five days. The solution methods chosen for integration and pH are Euler and Brent respectively, the model defaults.

3.3.1 Flow Gaging

Flow along the Caney River north of Bartlesville is heavily influenced by three upstream lakes. Hulah Lake (3,570 acres) is located approximately 20 miles upstream of the model area of interest with its discharges being controlled by the US Army Corps of Engineers (USACE). Copan Lake (5,000 acres) is approximately 8 miles upstream of the model area along the Little Caney River and is also operated by the USACE. Daily discharges from these two lakes (stations HULO2 and CPLO2 respectively) are available online (USACE, 2016). The smaller Hudson Lake (268 acres) is located along the Butler Creek tributary, and regular discharge data are not available.

Within the model extent, there is a USGS flow gage (station number 07174400 - Caney River above Coon Creek at Bartlesville, OK) located near the existing water intake above the OK-123 dam and bridge. This flow gage monitors daily stream discharge and gage height from 2007 to present, representing a drainage area of 1,369 square miles (3,546 square kilometers). There are no water quality data of-interest for model setup collected at this gage, although there are some channel survey details which are useful.

ODEQ requires modeling analysis for WLAs be conducted for low-flow conditions referred to as 7Q2 (seven-day two-year annual low-flow statistics). 7Q2 flows can be calculated using existing flow data from the USGS gage as the annual 7-day minimum flow with a 2-year recurrence interval (non-exceedance probability of 50 percent). Correspondence with ODEQ directed that 7Q2 analyses be conducted for each regulatory-based season, and the agency provided estimates of annual and summer seasonal 7Q2 flows of 13.9 cfs and 20.6 cfs respectively (Derichsweiler, 2011). A quick analysis of the period of record produced a non-official summer 7Q2 flow estimate of 20.2 cfs as of summer 2016. The quick analysis result justifies using the summer 7Q2 flow recommended by ODEQ in 2011 for this scoping-level analysis.

3.3.2 Stream Hydraulics

Stream hydraulics are handled in QUAL2K using one of three available methods: Weir, Rating Curve, or Manning Formula. The methods were applied to the Caney River model reaches as follows:

3.3.2.1 Rating Curve

The majority of the model reaches (1 through 6, 9 and 10) were modeled using the rating curve method. QUAL2K employs power equations to relate mean velocity (*U*) and depth (*H*) to flow for the elements in a reach,

$$U = aQ^b$$

$$H = \propto Q^{\beta}$$

such that *a*, *b*, ∝, and β are empirical coefficients determined from velocity-discharge and stage-discharge rating curves. Rating curve parameters established under the previous QUAL2E modeling effort based on the results of time-of-travel dye studies were applied to the new modeling segment. It was assumed that the limestone bedrock-dominated reaches of the Caney River in the QUAL2E model were representative of the reaches upstream of the low-head dam since no better information was available to support a different representation. QUAL2E parameterization values for these coefficients were 0.063 for *a*, 0.786 for *b*, 0.563 for ∝, and 0.661 for β respectively.

3.3.2.2 Weir

The single low-head dam located at the end of Reach 8 was modeled with the following weir equation details: type is broad-crested, height and width are 4.9 ft (1.5 m) and 196.9 ft (60 m) respectively, and coefficients for aeration (*adam* and *bdam*) were set to 1.25 and 0.75 respectively. An *adam* value of 1.25 is the default for the water-quality coefficient for weirs, reflecting a “moderate” to “slight” polluted state, while the *bdam* value of 0.75 is the dam-type coefficient for a “flat broad-crested straight-slope face” dam. The bottom width was set equal to the weir width of 196.9 ft (60 m) to ensure that even the lowest flows would fall across the entire width, which is typical of a low-head dam. An image of the low-head dam below the OK-123 Bridge may be seen in **Figure 3**.

Figure 3. Photograph of the low-head dam at OK-123 along the Caney River (ODOT, 2016)

3.3.2.3 Manning Formula

Reach 7 is located immediately upstream of the OK-123 low-head dam represented by the weir formula in Reach 8. Reach 7 includes USGS gage 07174400, and channel surveys recorded at that gage informed model assumptions. Channel survey data during low flow conditions at the gage reflect a width of approximately 102 ft (31 m) during 7Q2 flows, so Manning Formula was chosen to model Reach 7 in order to specify bottom width, and build upon existing parameterization for Manning “n” used in the previous model. Channel slope was estimated using two-meter LiDAR-derived digital elevation maps obtained from OKmaps.org. The following model inputs were used for Reach 7: Channel Slope was 0.00003 (m/m), Manning “n” was 0.08 (unitless), bottom width was 102 ft (31 m), and channel side slopes were 0 (m/m). By allowing the Caney River to get deeper and less wide prior to reaching the dam, the model appears to better approach natural-seeming conditions related to reaeration, width, depth, and velocity.

3.3.3 Meteorological Inputs

Key metrological inputs to the QUAL2K model are air temperature, dew point temperature, wind, cloud cover, and shade. In order to represent critical conditions, the Caney River model was set up to assume clear skies (0% cloud cover for all reaches at all hours), stagnant air (0 m/s wind speed for all reaches at all hours), and full sun (0% shade for all reaches at all hours).

In the QUAL2E model, all water temperatures were maintained at 30 °C (86 °F), therefore air temperatures will be set to 30 °C in order to maintain that water temperature. The dew point temperature chosen (21.5 °C, 70.7 °F) was estimated based on the observed relationship between air and dew point temperatures at the nearby Bartlesville Municipal Airport (NOAA, 2016).

3.3.4 Headwater Inputs

For the QUAL2K model, headwater inputs reflect stream conditions immediately upstream of the first model reach. Inputs include: flow rate, elevation, hydraulics parameters, prescribed dispersion, and in-stream water quality conditions such as temperature, conductivity, DO, BOD, pH, alkalinity, and nitrogen and phosphorus species. Note that in-stream water quality data was extremely limited for parameter development, therefore details were used from the QUAL2E modeling effort and other applicable sources. These parameters can be input on an hourly basis, however without further information and due to the scoping level nature of this model, the same values were provided for each hour. Headwater inputs are summarized in the table below (**Table 2**). The downstream extent of the model was not a prescribed boundary.

Table 2. Headwater inputs for Caney River QUAL2K scoping model (metric units)

Parameter	Units	Model Input	Source
Flow Rate	m^3/s	0.570	Drainage area scaled flow estimated from USGS gage
Elevation	m	198.90	LiDAR estimated with assumed depth subtracted
Hydraulic Formula	N/A	Rating Curve	Same method as most model reaches, parameters from QUAL2E model
Prescribed Dispersion	m^2/s	3.00	QUAL2E model input for initial conditions, measured in the field based on dye studies
Water Temperature	°C	30	QUAL2E model input for initial conditions (90th percentile of observed critical period data)
Conductivity	µmhos	353.25	Average field-observed condition from 9/9/2002 from QUAL2E modeling effort
Inorganic Solids	mgD/L	1.50	Estimated using stoichiometric relationship from QUAL2K manual from phytoplankton
Dissolved Oxygen	mg/L	6.50	QUAL2E model input for initial conditions
CBODslow	mgO_2/L	1.00	Background labile slow CBOD present in the stream due to general decomposition, value estimated from QUAL2E model inputs
CBODfast	mgO_2/L	0.00	No residual fast CBOD estimated to be present
Organic Nitrogen	µgN/L	460.00	QUAL2E model input for initial conditions
NH4-Nitrogen	µgN/L	25.00	QUAL2E model input for initial conditions
NO3-Nitrogen	µgN/L	50.00	QUAL2E model input for initial conditions
Organic Phosphorus	µgP/L	40.00	QUAL2E model input for initial conditions
Inorganic Phosphorus	µgP/L	25.00	QUAL2E model input for initial conditions
Phytoplankton	µgA/L	15.00	Chlorophyll *a* for headwaters in QUAL2E model
Internal Nitrogen	µgN/L	Null	Calculated internally using stoichiometric relationship from QUAL2K manual from phytoplankton
Internal Phosphorus	µgP/L	Null	
Detritus	mgD/L	Null	
Alkalinity	$mgCaCO_3/L$	150	Estimate for freshwater (Murphy, 2007)
pH	unitless	7.82	Average field-observed condition from 9/9/2002 from QUAL2E modeling effort

3.3.5 Diffuse Sources

Flow rates were area-weighted based on drainage area in order to scale the estimated flow at the headwaters compared to the observed 7Q2 flow located at the USGS gage. The difference in flow between each reach was included in the model as diffuse groundwater inputs with prescribed slow CBOD concentrations of 1.00 mgO_2/L.

Table 3. Diffuse groundwater source inputs for Caney River QUAL2K scoping model

Reach	Diffuse Flow, cfs (m^3/s)	Slow CBOD (mgO_2/L)
1	0.0035 (0.0001)	1.00
2	0.0177 (0.0005)	1.00
3	0.0035 (0.0001)	1.00
4	0.0035 (0.0001)	1.00
5	0.0353 (0.0010)	1.00
6	0.0247 (0.0007)	1.00
7	0.0000 (0.0000)	1.00
8	0.0071 (0.0002)	1.00
9	0.1130 (0.0032)	1.00
10	0.0247 (0.0007)	1.00

3.3.6 Point Source Inputs

Point sources can be used in QUAL2K to represent flows into or out of the mainstem, from tributary or wastewater discharge inflows, to water withdrawals for potable supplies or otherwise. The water intake located above the OK-123 dam is not simulated in this model because it is not used regularly and would likely not be in use at all during critical low flow conditions. Point source inputs for this model have been set up for Butler Creek tributary and the Chickasaw WWTP, detailed below.

3.3.6.1 Butler Creek

Although Butler Creek is a relatively large tributary to the Caney River (containing Hudson Lake which is a potable water source for Bartlesville), it is ungaged. The Butler Creek drainage area of 25.6 square miles (66.4 km^2) was used to area-weight the estimated flow from this tributary based on the 7Q2 flow at the downstream Caney River USGS gage. Water quality parameterization for Butler Creek was estimated to be identical to the water quality of the Caney River (**Table 5**).

3.3.6.2 Chickasaw Wastewater Treatment Plant

Bartlesville treats city wastewater at the Chickasaw WWTP, which is operated by Veolia Water North America Operating Services. The Chickasaw WWTP is regulated under the Oklahoma Pollutant Discharge Elimination System (OPDES) permit OK0030333 (Facility ID S-21402) and is located 0.5 miles

(0.8 km) downstream of the OK-123 bridge and dam. This WWTP provides biological treatment of domestic sewage, and consists of three primary clarifiers, three activated sludge aeration basins, three secondary clarifiers, a chlorine contact basin for disinfection, sulfur dioxide removal of excess chlorine, and a step aerator to increase DO prior to discharge through Outfall 001 into the Caney River (ODEQ, 2013).

ODEQ has previously permitted an average facility design flow for Chickasaw WWTP of 7.0 MGD (10.8 cfs) with the effluent limits displayed in **Table 4**.

Table 4. Chickasaw WWTP discharge effluent limitations (ODEQ, 2013)

Effluent Characteristic	Discharge Limitations			Monitoring Requirement Frequency
	Monthly Average	Weekly Average	Daily Maximum	
Flow (MGD)	Report	---	Report	Daily
Biochemical Oxygen Demand (BOD_5, mg/L)	10	15	---	5/week
Total Suspended Solids (mg/L)	15	22.5	---	5/week
Ammonia (NH_3-N, mg/L)	2	3	---	5/week
pH (standard unit)	6.5-9.0			Daily
Total Residual Chlorine (mg/L)	Instantaneous samples should be <0.1			Daily
Fecal Coliform (colonies/100mL)	200	---	400	2/week
Total Lead (mg/L)	4.7	---	11.3	Quarterly

The previous WLA recognized in the current Chickasaw WWTP OPDES permit was not based on a calibrated model. During the facility planning process, the WLA analysis using the calibrated QUAL2E model found an appropriate maximum flow at this location to be 3.97 MGD (6.1 cfs) to allow for downstream waste assimilation. Daily reported effluent from the facility from 2010 to 2015 reveal average daily discharges to be 6.9 MGD (10.8 cfs), and monitoring data during the previous study showed DO standard violations from the existing plant effluent. Therefore, the Chickasaw WWTP input for this modeling analysis was held at 3.97 MGD with BOD_5 of 10 mg/L, NH_3-N of 1 mg/L, and an effluent DO of 6.0 mg/L which were shown to meet water quality standards under a summer 7Q2 critical condition. Since the QUAL2K model does not employ input of BOD_5 concentration but rather CBOD, details from the previous QUAL2E project was used to inform the relationship between these parameters. The CBOD-to-BOD_5 ratio established during QUAL2E model calibration was estimated to be 2:1, such that a concentration of 10 mg/L BOD_5 would be input to the model as 20 mg/L CBOD.

Parameterization of Chickasaw WWTP QUAL2K model inputs and sources are detailed in **Table 5**.

Table 5. Point source inputs for Caney River QUAL2K scoping model

Parameter	Units	Butler Creek Tributary	Chickasaw WWTP	Source
Location	km	6.20	0.62	Aerial imagery and GIS data
Inflow	m^3/s (MGD)	0.0123 (0.28)	0.1739 (3.97)	Tributary flow based on drainage area weighting from USGS gage flow, WWTP flow from WLA estimation
Water Temperature	°C	30	30	QUAL2E model input for initial conditions (90th percentile of observed critical period data)
Specific Conductance	µmhos	353.25	323.25	Same as mainstem
Alkalinity	$mgCaCO_3/L$	150	150	
Dissolved Oxygen	mg/L	6.50	6.00	Tributary same as mainstem, WWTP based on WLA estimation
Slow CBOD	mgO_2/L	1.00	0.00	Tributary same as mainstem, WWTP assumed zero (all fast CBOD)
Fast CBOD	mgO2/L	0.00	20.00	Tributary same as mainstem, WWTP set to BOD_5 permit limit based on BOD_5:CBOD ratio
Ammonia Nitrogen	µgN/L	25	1000	Tributary same as mainstem, WWTP based on WLA estimation
Organic Nitrogen	µgN/L	460	323	Tributary same as mainstem, WWTP from QUAL2E setup
Inorganic Suspended Solids	mg/L	1.50	3.20	
Nitrate+Nitrite Nitrogen	µgN/L	50	10300	
Organic Phosphorus	µgP/L	40	5	
Inorganic Phosphorus	µgP/L	25	2750	
pH	unitless	7.82	7.02	
Phytoplankton	µgA/L	15	0	Tributary same as mainstem, WWTP assumed zero
Internal Nitrogen	µgN/L	Null	Null	Calculated internally using stoichiometry
Internal Phosphorus	µgP/L	Null	Null	
Detritus	mgD/L	Null	Null	

3.3.7 Reach Rates, Rates, Light and Heat Inputs

Reach rates and velocities, water column rates, light parameters, and surface heat transfer model parameters were identical for all reaches in the model. Without sufficient field information to suggest otherwise, basic assumptions on driving kinetics consistency throughout the model extent are reasonable. All “light and heat” inputs were kept as QUAL2K model default values. All “water column rates” were also kept as model default values except for the reaeration model which was specified as the Churchill Method. Reaeration is a key parameter in DO modeling, and some of the key hydraulic-based formulas for reaeration in QUAL2K are the Churchill Method (Churchill et al, 1962), Owens-Gibbs Method (Owens et al, 1964), and O’Connor-Dobbins Method (O’Connor and Dobbins, 1958). The Internal Method (default for QUAL2K) uses a scheme developed by Covar (1976) such that depending on water depth and velocity, the model will employ O’Connor-Dobbins (depth >1.97 ft [0.6 m], velocity generally <1.64 ft/s [0.5 m/s]), Owens-Gibbs (depth <1.97 ft [0.6 m], all velocities), or Churchill (depth >1.97 ft [0.6 m], velocity generally >1.64 ft/s [0.5 m/s]). The Churchill reaeration model was chosen to be the likely best assumption for this system at critical low flows because of its use in the previous QUAL2E modeling and the fact that it represents one of the most conservative methods—thought to be appropriate for a scoping-level analysis. Sensitivity analyses were conducted during the model scenario runs to explore the dependence of model results on these assumptions (these analyses are detailed in Sections 5 and 6). The reach rate inputs for all reaches are summarized in **Table 6**.

Table 6. Reach rate inputs for all reaches (metric units)

Parameter	Units	Input	Source
Reaeration Model	N/A	Churchill	Conservative hydraulic-based option
Prescribed Dispersion	m^2/s	3.0	Based on QUAL2E model and longitudinal dye field studies from related field work
Bottom Algae Coverage	%	50%	Estimated
Bottom SOD Coverage	%	100%	Conservative estimate (full coverage)
Prescribed SOD	$gO_2/m^2/d$	0.42	Same as QUAL2E model input
Prescribed Evaporation	mm/d	2.0946	Estimated from Taghvaeian and Sutherland (2015)
ISS Settling Velocity	m/d	0.01	Assumed similar to Organic P velocity
Slow CBOD Hydrolysis Rate	/d	0.00	Assumed no conversion of slow CBOD to fast CBOD
Slow CBOD Oxidation Rate	/d	0.03	Estimated as average slow-bottle rate
Fast CBOD Oxidation Rate	/d	0.40	Estimated as BOD decay rate used in QUAL2E model
Organic N Hydrolysis Rate	/d	0.10	Same as QUAL2E model input
Organic N Settling Velocity	m/d	0.02	Same as QUAL2E model input
Ammonium Nitrification	/d	0.40	Same as QUAL2E model input
Nitrate Denitrification Rate	m/d	2.00	Same as QUAL2E model input
Organic P Hydrolysis Rate	/d	0.10	Same as QUAL2E model input
Organic P Settling Velocity	m/d	0.01	Same as QUAL2E model input
Inorganic P Settling Velocity	m/d	0.01	Assumed similar to Organic P velocity
Phytoplankton Max Growth Rate	/d	1.80	Same as QUAL2E model input
Phytoplankton Respiration Rate	/d	0.15	Same as QUAL2E model input
Phytoplankton Excretion Rate	/d	0.00	Assumed to be zero
Phytoplankton Death Rate	/d	0.00	Assumed to be zero
Phytoplankton Settling Velocity	m/d	0.00	Assumed to be zero
Bottom Algae Max Growth Rate	/d	1.8	Assumed same rates as phytoplankton
Bottom Algae Respiration Rate	/d	0.15	Assumed same rates as phytoplankton
Bottom Algae Excretion Rate	/d	0.00	Assumed same rates as phytoplankton
Bottom Algae Death Rate	/d	0.00	Assumed same rates as phytoplankton

4.0 BASELINE MODEL RESULTS

The first application of the QUAL2K model was set up to provide a baseline set of outputs representative of existing critical low flow summer conditions, and reflecting a prescribed discharge associated with the Chickasaw WWTP. In general, the baseline results should be consistent with existing observations where available, along with the previous QUAL2E modeling for the overlapping reaches.

For the baseline setup, the QUAL2K model predicts that DO in the portion of the Caney River above the low-head dam reaches an equilibrium concentration of about 5.9 mg/L under critical summer period low flow conditions (**Figure 4**). There is a small sag in predicted DO concentration immediately before the low-head dam associated with slowing and deepening of the water volume, followed by a predicted sharp DO increase of roughly 0.6 mg/L due to reaeration over the dam associated with the sheer drop in elevation from the dam height of 4.9 ft (1.5 m). This predicted increase in DO across the dam is well within the documented range of observed low-head dam reaeration potential, which has been measured as high as 2.25 mg/L in the Midwest (Butts and Evans, 1978), and the resulting DO level is consistent with the concentrations observed below the dam during the previous Tetra Tech summer low flow study. DO concentration decreases below the dam due to in-stream kinetics until a steeper drop in DO is predicted immediately below the Chickasaw WWTP effluent outfall (again consistent with observations from the previous monitoring study conducted by Tetra Tech, and a direct result of using the same model setup assumptions for this segment to be consistent with the previous QUAL2E modeling effort).

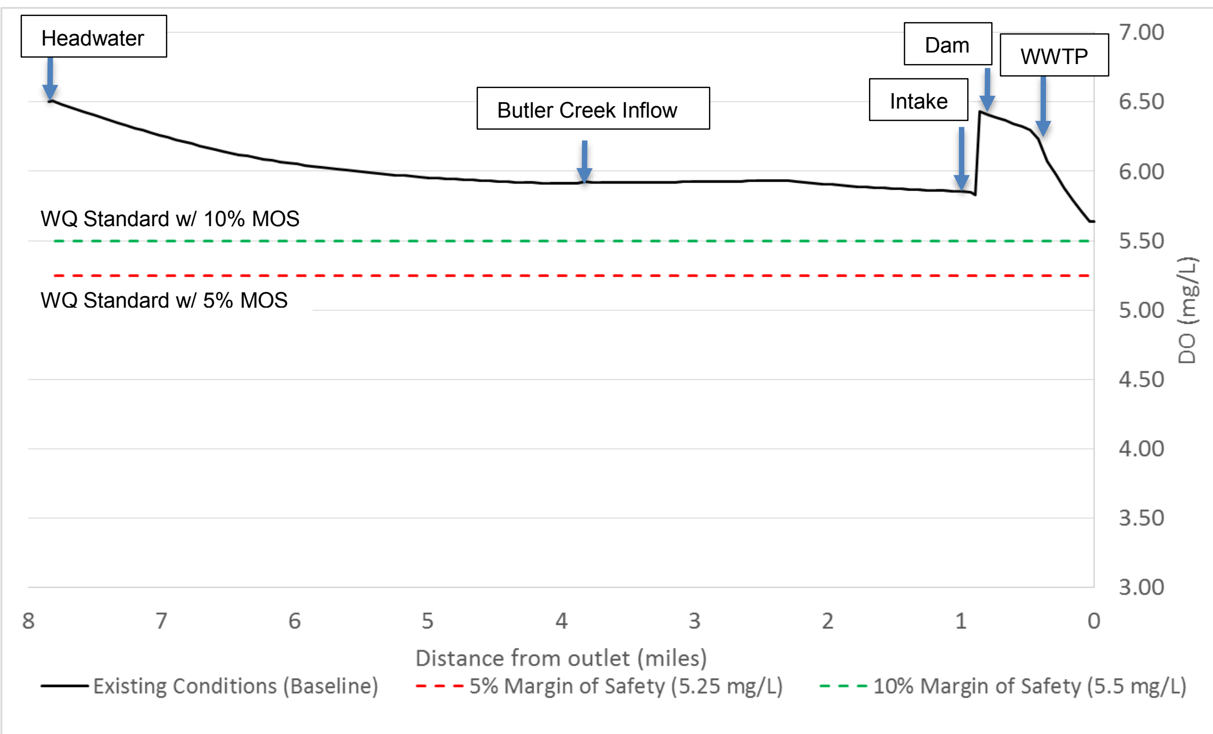

Figure 4. Longitudinal graph of modeled dissolved oxygen concentrations along Caney River, critical conditions baseline scenario

Under the existing conditions of the QUAL2K model, flow velocity is 0.13-0.16 ft/s (0.04-0.05 m/s) for all reaches, except for immediately before the dam where velocity is slowed due to assumed pooling. Under the specified Churchill method for estimating reaeration, the resulting predicted reaeration rates are generally 0.90-0.99 /d. One exception is for the portion of stream in Reach 7 immediately before the low-head dam where reaeration is predicted to drop to 0.72 /d due to an increase in depth, which is indicative of the anticipated backwater effect of the dam. The reaeration rate before the dam is a bit low compared with reported rates which are suggested to naturally occur at or above 1 /d (EPA, 1985; Thomann and Mueller, 1987), but the impact of reaeration from the dam itself is on the order of magnitude anticipated for relative change in stream DO concentrations.

In-stream DO was observed during the QUAL2E modeling effort field work in 2002 downstream of the dam. Below the dam and upstream of Chickasaw WWTP average DO on 9/9/2002 during relatively low flows was observed throughout the day as ranging from 5.98 to 7.20 mg/L. The baseline model estimates average DO concentration between the dam and WWTP as 6.35 mg/L which is within this range. There are no existing field measurements of DO upstream of the dam. Scenarios and sensitivity analysis will be considered in order to address the potential assimilative capacity of the river to receive addition upstream WWTP discharge. Because this scoping level model is not calibrated, there are a number of model sensitivities to be considered in the face of limited field data. For example, if the Caney River above the OK-123 dam has very low natural reaeration and very high sediment oxygen demand, its natural assimilative capacity will be greatly diminished.

5.0 QUAL2K MODEL APPLICATION SCENARIOS

The main questions this scoping analysis seeks to answer are:

1. Is there any existing assimilative capacity in the Caney River upstream of the Chickasaw WWTP which would support a secondary effluent discharge location?
2. If so, what is the general magnitude of that assimilative capacity, and what impacts do various upstream discharge scenarios have on the stream and the Chickasaw WLA?
3. How sensitive are the scoping-level predictions to key parameter assumptions and what are the implications for facility planning?

A total of 13 scenarios were run to address these questions, which are described by grouping below and summarized individually in **Table 7**.

The first suite of wasteflow scenarios (1 through 4) test the new upstream discharge location and various discharge rates. The facility planning study for Bartlesville indicates a long-term projected flow need of 8.5 MGD. The previous QUAL2E modeling effort was calibrated to monitoring data and indicated a maximum effluent discharge at the current Chickasaw outfall of 3.97 MGD, so that level was maintained for all modeled scenarios. The difference between the projected 8.5 MGD flow and the existing maximum discharge is 4.53 MGD. Scenarios 1 and 2 were set up as the initial runs to test discharging 4.53 MGD of the same water quality as the Chickasaw effluent at five miles or seven miles upstream of the water supply intake respectively. Scenarios 3 and 4 included the upstream discharge at five miles, although flow was decreased in order to meet the DO water quality standard of 5.0 mg/L with a 5 percent margin of safety (5.25 mg/L) and with a 10 percent margin of safety (5.50 mg/L) respectively.

The condition for which effluent may be discharged at a distance five miles upstream of the intake rather than seven miles upstream is preferable from a financial cost standpoint, therefore the assimilative capacity of the stream from this location was used to explore model sensitivity to specific parameters in Scenarios 5 through 11. Using Scenario 3 as a starting position, the following parameters were altered iteratively to explore the impact on the model results (i.e., model sensitivity): reaeration rate, sediment oxygen demand (SOD) rate, fast CBOD rate, and net photosynthesis/respiration rate.

Scenarios 12 and 13 employ the maximum flow scenario (4.53 MGD) at five miles upstream, exploring model response to decreases in BOD_5 and NH_3 concentration in the upstream effluent discharge respectively. These runs are meant to provide a scoping level answer to the question of how much would effluent limits need to be reduced at the new discharge point in order to meet water quality standards.

All scenario runs are detailed by specific changes made to the baseline QUAL2K model in **Table 7**.

Table 7. Summary of individual model scenarios

Run	Description of Scenario and Change in Model Inputs
Baseline	Existing conditions, Chickasaw WWTP existing location discharge at 3.97 MGD with no addition upstream discharge.
Scenario 1	From Baseline, add new discharge of 4.53 MGD at 5 miles upstream of intake (model location 9.61 km)
Scenario 2	From Baseline, add new discharge of 4.53 MGD at 7 miles upstream of intake (model location 12.63 km)
Scenario 3	From Baseline, add new discharge at 5 miles upstream of with maximum flow which allows for the in-stream DO minimum not to fall below 5.25 mg/L (5% margin of safety on 5.0 mg/L WWAC standard)
Scenario 4	From Baseline, add new discharge at 5 miles upstream of intake with maximum flow which allows for the in-stream DO minimum not to fall below 5.50 mg/L (10% margin of safety on 5.0 mg/L WWAC standard)
Scenario 5	From Scenario 3 (discharge at 5 miles, flow of 1.55 MGD), change reaeration formula from Churchill to Covar (Internal Method)
Scenario 6	From Scenario 3 (discharge at 5 miles, flow of 1.55 MGD), decrease SOD rate from 0.42 to 0.30 $mgO_2/m^2/d$
Scenario 7	From Scenario 3 (discharge at 5 miles, flow of 1.55 MGD), change reaeration formula from Churchill to Covar (Internal Method), and decrease SOD rate from 0.42 to 0.30 $mgO_2/m^2/d$
Scenario 8	From Scenario 3 (discharge at 5 miles, flow of 1.55 MGD), increase SOD rate from 0.42 to 0.50 $mgO_2/m^2/d$
Scenario 9	From Scenario 3 (discharge at 5 miles, flow of 1.55 MGD), fast CBOD rate increased from 0.40 to 0.50 /d
Scenario 10	From Scenario 3 (discharge at 5 miles, flow of 1.55 MGD), fast CBOD rate decreased from 0.40 to 0.30 /d
Scenario 11	From Scenario 3 (discharge at 5 miles, flow of 1.55 MGD), photosynthesis and respiration turned off

Run	Description of Scenario and Change in Model Inputs
Scenario 12	From Baseline, add new discharge at 5 miles upstream of intake, decrease new effluent BOD_5 to 5 mg/L (10 mg/L CBOD) with maximum flow which allows for the in-stream DO minimum not to fall below 5.25 mg/L (5% margin of safety on 5.0 mg/L WWAC standard)
Scenario 13	From Baseline, add new discharge at 5 miles upstream of intake, decrease new effluent BOD_5 to 5 mg/L (10 mg/L CBOD), and decrease NH_3 from 1 mg/L to 0.05 mg/L with maximum flow which allows for the in-stream DO minimum not to fall below 5.25 mg/L (5% margin of safety on 5.0 mg/L WWAC standard)

6.0 QUAL2K MODEL SCENARIO RESULTS

Based on model results, there appear to be a range of scenarios in which the assimilative capacity of the Caney River can support an upstream discharge while in-stream water quality numeric criteria are met.

6.1 ASSIMILATIVE CAPACITY PREDICTIONS UNDER SCOPING-LEVEL QUAL2K MODEL SETUP

The first suite of wasteflow scenarios (1, 2, 3, and 4) represent conditions for which existing model conditions are held constant, and a new discharge is added at either five miles upstream or seven miles upstream of the existing intake (Figure 5). Scenario 1 (5mi@4.53MGD) shows the full projected expansion of 4.53 MGD discharge at the closest modeled location of five miles upstream. This scenario produces a drop in DO concentration (relative to the baseline condition) of 1.68 mg/L at mile marker 4, with a minimum DO of 4.23 mg/L which is below the WWAC criteria considering either a 5 percent or 10 percent MOS.

In Scenario 1, the model predicts DO, fCBOD, NH_3-N, and TN concentrations at the backup water supply intake (i.e., above the low-head dam) of 4.45, 1.11, 0.005, and 0.35 mg/L respectively. The concentrations of the same parameters immediately upstream of the Chickasaw outfall were estimated as 5.60, 1.18, 0.070, and 0.44 mg/L respectively. The DO minimum decreases by 1.40 mg/L between the Baseline and Scenario 1. Water quality above of the Chickasaw WWTP outfall reveals a DO decrease of 0.63 mg/L relative to the baseline, and an increase in fBOD of 0.41 mg/L. The general conclusion from this run is that there is not enough assimilative capacity at a point five miles upstream of the water supply intake for an effluent discharge of 4.53 MGD with concentrations of 10 mg/L BOD5,1 mg/L NH3 and a DO of 6 mg/L. Therefore, additional scenarios were run to determine whether a location further upstream would be preferable from a water quality perspective, as well as exploration of what small amounts of wasteflow might be assimilated at the five mile location.

When the full 4.53 MGD flow is discharged seven miles upstream (Scenario 2), the assimilative capacity of the stream is similar, with a minimum DO of 4.38 mg/L, which is below the water quality standard for DO considering either a 5 percent or 10 percent MOS. In Scenario 2, the model predicts DO, fCBOD, NH_3-N, and TN concentrations at the downstream water supply intake of 4.85, 0.75, 0.005, and 0.33 mg/L respectively. The concentrations of the same parameters immediately above the Chickasaw WWTP outfall were estimated as 5.81, 0.87, 0.072, and 0.42 mg/L respectively. Although stream water quality is predicted to be slightly better at the downstream intake and Chickasaw WWTP outfall for the seven mile discharge compared to the five mile discharge, in-stream water quality standards are not met, and there appears to be a likely impact on the downstream water quality for the Chickasaw WLA.

Scenarios 3 and 4 show the maximum effluent flows which can be discharged at the five mile location to meet in-stream DO concentrations of 5.25 mg/L and 5.50 mg/L respectively. If the basic model setup is reasonably accurate, then it appears that the Caney River has the assimilative capacity to receive an upstream discharge at five miles of 1.55 MGD (equivalent of Chickasaw WWTP limits) to meet the 5 percent MOS, and a flow of 0.91 MGD to meet the 10 percent MOS. In Scenario 3, the model predicts DO, fCBOD, NH_3-N, and TN concentrations at the intake of 5.33, 0.45, 0.006, and 0.29 mg/L respectively. The concentrations of the same parameters above Chickasaw were estimated as 6.02, 0.91, 0.077, and 0.54 mg/L respectively. In Scenario 4, the model predicts DO, fCBOD, NH_3-N, and TN concentrations at the intake of 5.55, 0.26, 0.006, and 0.29 mg/L respectively. The concentrations of the same parameters above Chickasaw were estimated as 6.12, 0.84, 0.080, and 0.57 mg/L respectively.

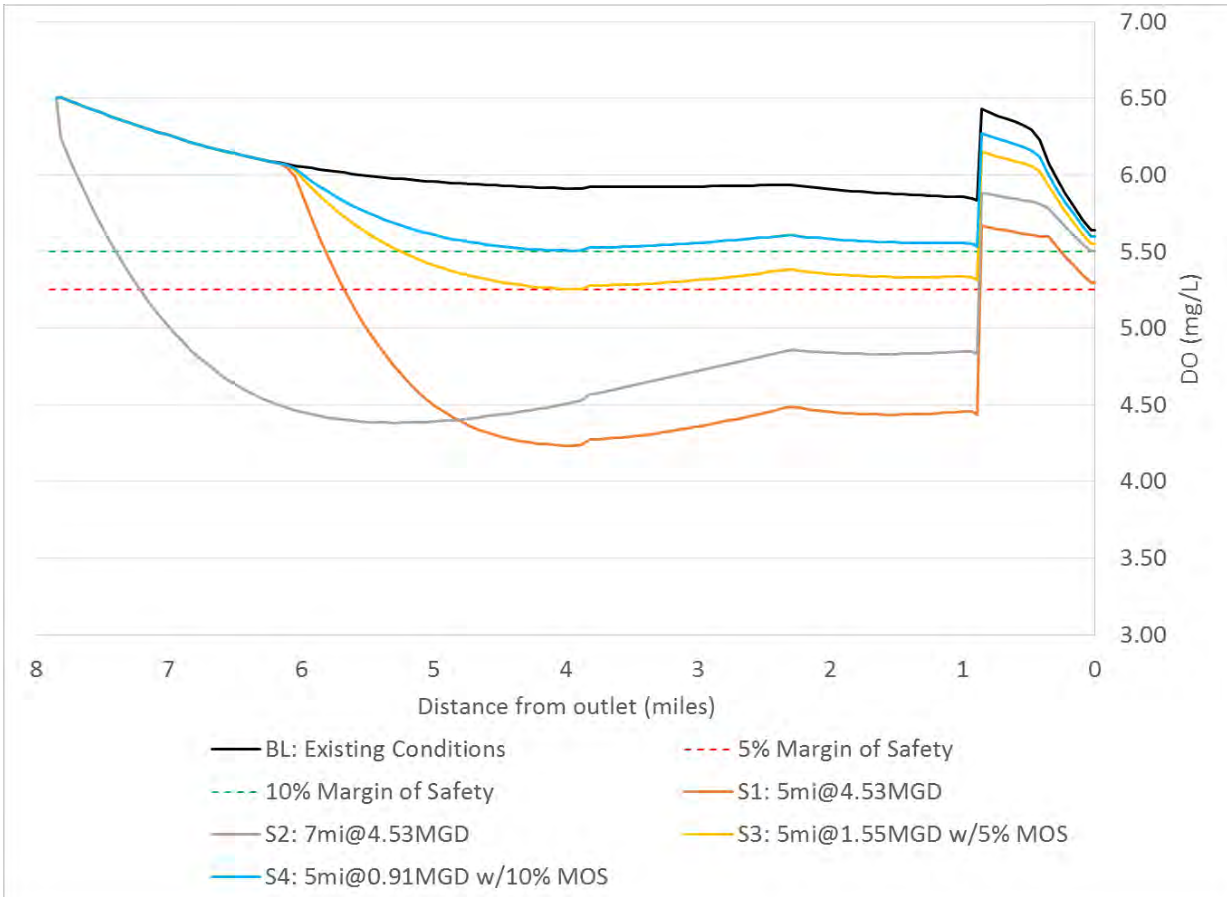

Figure 5. Longitudinal graph of modeled dissolved oxygen concentrations along Caney River for baseline and scenarios 1-4

6.2 MODEL SENSITIVITY TO KEY PARAMETER ASSUMPTIONS

The next suite of scenarios (5 through 13) explore model sensitivity to a number of key parameters which can impact DO in-stream. All of these scenarios are based on adding a 1.55 MGD discharge five miles upstream of the intake (Scenario 3). Scenario 3 was selected as a basis for comparison because it represents the highest possible discharge (with quality equivalent to the Chickasaw WWTP effluent WLA concentrations) that may be assimilated. The sensitivity analysis provides insight into how robust the estimates are for how much effluent volume can likely be assimilated in this portion of the Caney River.

Reaeration

Scenarios 5 and 7 stand out (see **Figure 6**) in showing the model sensitivity to reaeration by changing the model method for estimating reaeration to “Internal” (i.e., default application of the Covar method). Reaeration rates in the majority of stream reaches under the Covar method predict reaeration of approximately 3.3–3.6 /day, well above the approximate average of 0.9/day predicted using the Churchill method. Although reaeration rates vary heavily across stream conditions (e.g. from 1 to >100 per the EPA Rates manual, 1985), these differences are significant as the DO profiles in Figure 6 illustrate. The profiles reflecting a Covar method show increased assimilative capacity would be available.

Without in-stream data to indicate otherwise, however, application of the Churchill method is likely more appropriate for estimating reaeration in the Caney River. Results from the previous QUAL2E modeling analysis found that this method yielded a better match to observed DO concentrations in the stream, and the method also represents a more conservative approach appropriate for scoping-level analysis that supports regulatory decision-making. However, if stream studies reveal a higher reaeration rate in the river than what is estimated using the Churchill method, then assimilative capacity for oxygen-consuming waste would be higher than estimated by the scoping analysis if other assumptions hold true as represented.

Sediment Oxygen Demand

Scenarios 6 and 8 show the relative impact of decreasing and increasing sediment oxygen demand (SOD) by 0.12 and 0.8 $mgO_2/m^2/d$ respectively (**Figure 6**). Decreasing SOD from a rate of 0.42 to 0.3 is predicted to raise the DO minimum by 0.20 mg/L, whereas increasing SOD rate to 0.5 is predicted to decrease the DO concentration at the sag point by 0.13 mg/L. Thus if SOD rate is significantly lower than assumed for the current baseline model setup, there would be more assimilative capacity available than predicted. For this case, the allowable effluent flow would increase from 1.55 MGD to 2.10 MGD. The reverse would be true if SOD is higher than that assumed. For the assumption of increasing SOD to 0.5, allowable effluent would decrease to 1.19 MGD.

BOD Decay

Scenarios 9 and 10 show model sensitivity to the in-stream fast CBOD decay rate (increase and decrease by 25 percent), which had a smaller absolute impact on minimum DO concentrations than changes in SOD (-0.08 mg/L and +0.08 mg/L respectively). The higher CBOD rate scenario would impact the allowable effluent volume negatively, reducing it from 1.55 MGD to 1.37 MGD. Conversely, a 25 percent lower CBOD decay rate would allow effluent volume to increase to 1.80 MGD.

Net Photosynthesis/Respiration

Scenario 11 shows the net impact on the model prediction for DO when photosynthesis and respiration are not modeled. DO concentrations increase on the order of 0.12 mg/L along the entire reach, demonstrating that current model assumptions represent a slight dominance of respiration over photosynthesis. If the net impact of photosynthesis and respiration is zero (i.e., neither dominates) as scenario 11 assumes, the allowable effluent volume would increase to 1.96 MGD.

Effluent BOD_5 (as CBOD) and NH_3

Scenario 12 revealed that if effluent BOD_5 were reduced from 10 mg/L to 5 mg/L, effluent flow could be increased to 3.13 MGD and still meet the DO water quality standard with a 5 percent MOS. This decrease in BOD_5 is reflected in the model as a decrease in CBOD concentration from 20 mg/L to 10 mg/L. Scenario 13 shows when effluent BOD_5 is decreased to 5 mg/L and effluent NH_3 concentration is reduced from 1 mg/L to 0.5 mg/L (i.e., cut in half), then effluent flow could be increased to 3.81 MGD at this location and still achieve compliance with the DO water quality standard with 5 percent MOS. The impact of these effluent concentration reductions and increased allowable flows at the five mile distance location are seen as compared to Scenario 3 (BOD_5 at 10 mg/L and NH_3 at 1 mg/L) in **Figure 7**. These results suggest that the assimilative capacity of the stream is greater and can handle higher effluent flow volumes if the associated water quality is treated at a higher level (e.g. advanced tertiary).

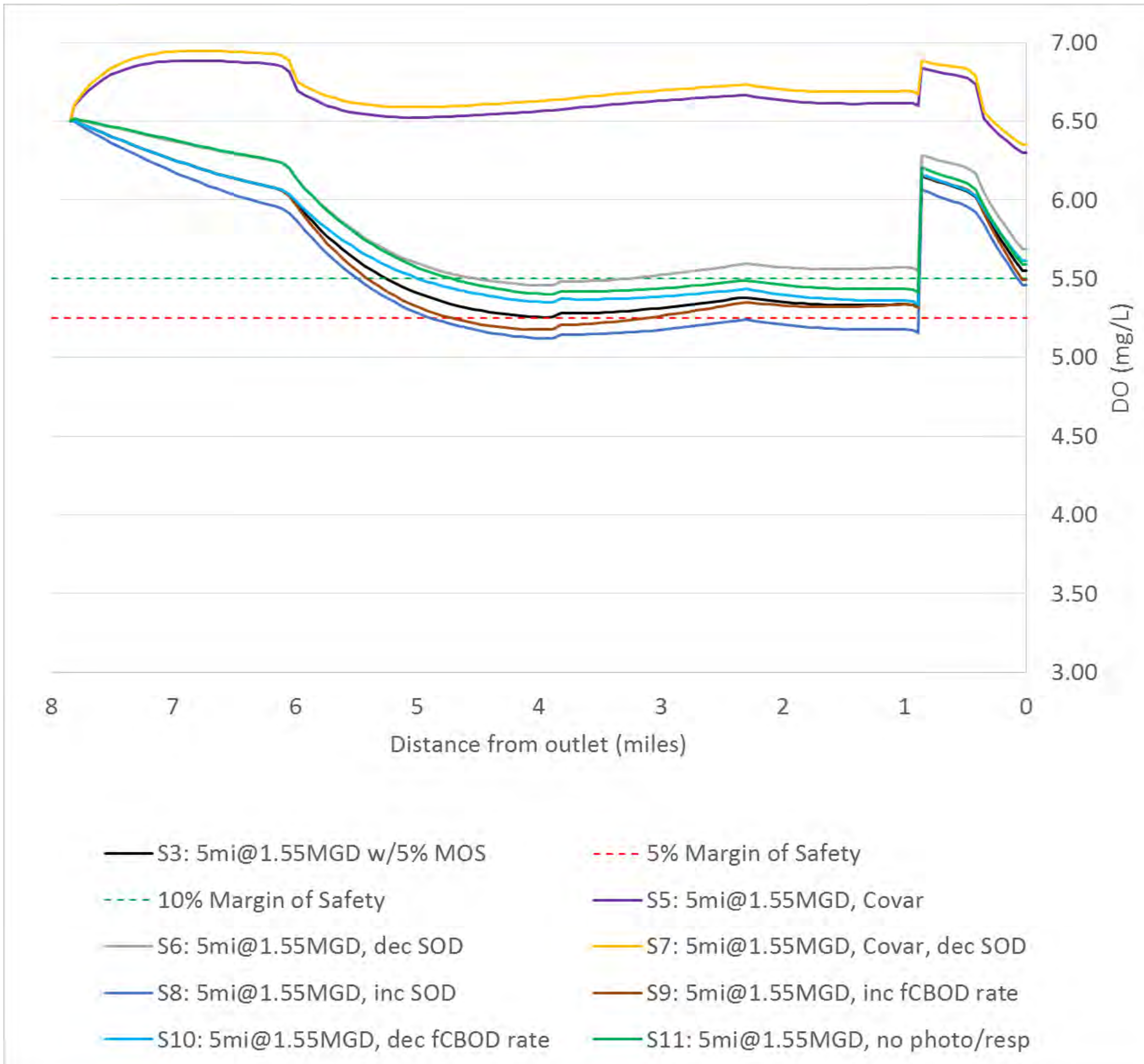

Figure 6. Longitudinal graph of modeled dissolved oxygen concentrations along Caney River showing sensitivity to a number of parameters (scenarios 3, 5-11)

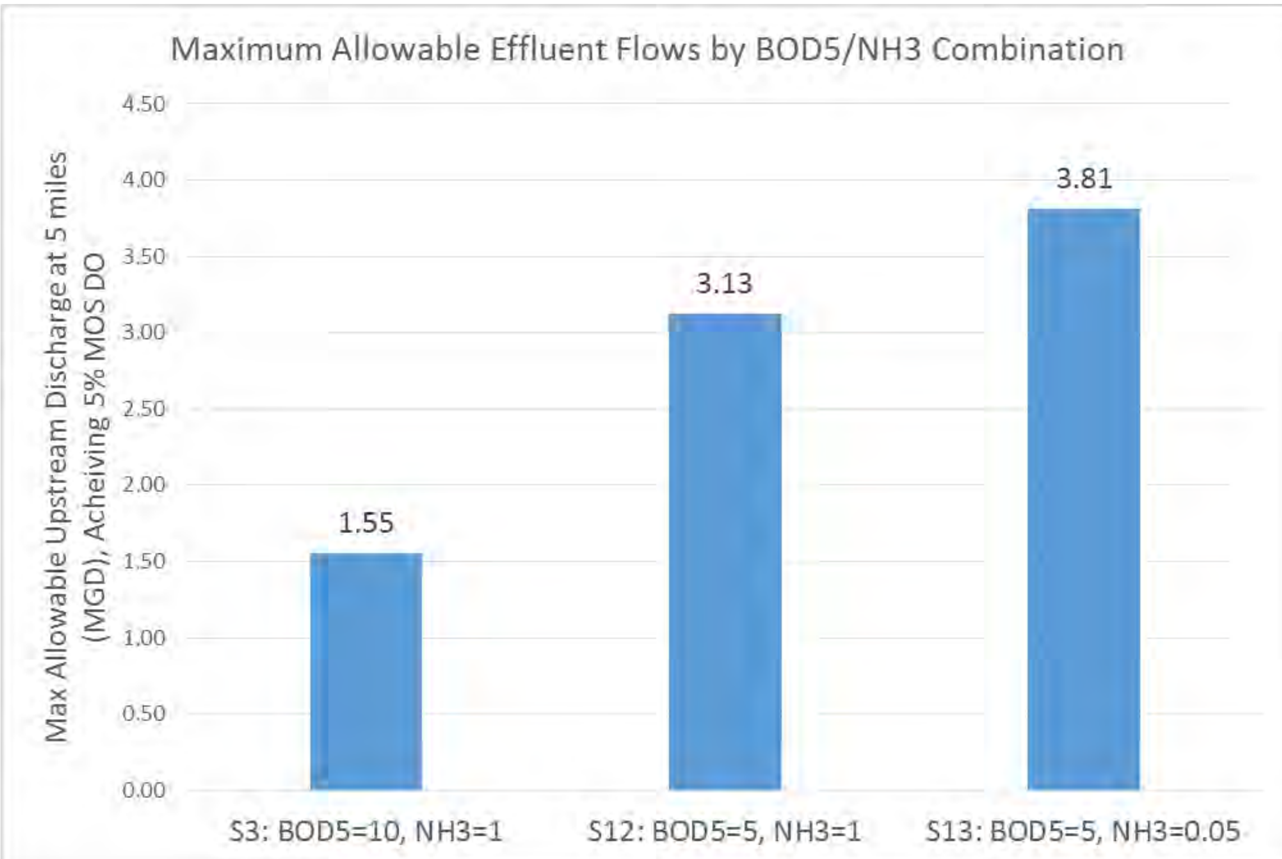

Figure 7. Bar graph showing maximum allowable effluent flows for combinations of BOD_5 and NH_3 concentrations in upstream effluent

Table 8. Modeled QUAL2K scenario results (all results are mg/L unless otherwise listed)

Run	Quick Reference Description	Water Quality at the Intake				Water Quality above Chickasaw				Min DO	Median Reaeration Rate (/d)
		DO	fCBOD	NH_3-N	TN	DO	fCBOD	NH_3-N	TN		
Baseline	Existing Conditions	5.85	0.00	0.008	0.27	6.23	0.77	0.089	0.62	5.64	0.99
Scenario 1	5mi@4.53MGD	4.45	1.11	0.005	0.35	5.60	1.18	0.070	0.44	4.23	0.90
Scenario 2	7mi@4.53MGD	4.85	0.75	0.005	0.33	5.81	0.87	0.072	0.42	4.38	0.90
Scenario 3	5mi@1.55MGD, 5% MOS	5.33	0.45	0.006	0.29	6.02	0.91	0.077	0.54	5.26	0.95
Scenario 4	5mi@0.91MGD, 10% MOS	5.55	0.26	0.007	0.28	6.12	0.84	0.080	0.57	5.50	0.97
Scenario 5	5mi@1.55MGD, Covar	6.61	0.46	0.012	0.29	6.73	0.92	0.133	0.54	6.30	3.30
Scenario 6	5mi@1.55MGD, dec SOD	5.57	0.45	0.006	0.29	6.17	0.91	0.077	0.54	5.46	0.95
Scenario 7	5mi@1.55MGD, Covar & dec SOD	6.69	0.46	0.012	0.29	6.79	0.92	0.132	0.54	6.35	3.30
Scenario 8	5mi@1.55MGD, inc SOD	5.17	0.45	0.006	0.29	5.92	0.91	0.077	0.54	5.12	0.95
Scenario 9	5mi@1.55MGD, inc fCBOD rate	5.33	0.31	0.006	0.29	6.02	0.78	0.077	0.54	5.18	0.95
Scenario 10	5mi@1.55MGD, dec fCBOD rate	5.35	0.64	0.006	0.29	6.03	1.10	0.077	0.54	5.34	0.95
Scenario 11	5mi@1.55MGD, no photo/resp	5.43	0.45	0.007	0.30	6.07	0.91	0.085	0.55	5.40	0.95
Scenario 12	5mi@3.13MGD, dec eff BOD_5	5.27	0.42	0.007	0.33	6.01	0.71	0.082	0.48	5.25	0.92
Scenario 13	5mi@3.81MGD, dec eff NH_3 & BOD_5	5.27	0.51	0.005	0.31	6.02	0.72	0.071	0.44	5.25	0.91

Table 9. Modified flow regimes for each scenario to meet water quality standards (DO concentration with 5% margin of safety: 5.25 mg/L)

Run	Quick Reference Description	Notes for maximum flow achieving DO standard
Baseline	Existing Conditions	Standard met/exceeded for existing conditions
Scenario 1	5mi@4.53MGD	Standard met for flow of 1.55 MGD (see Scen 3)
Scenario 2	7mi@4.53MGD	Standard met for flow of 1.80 MGD
Scenario 3	5mi@1.55MGD, 5% MOS	Standard met for purposes of initial setup
Scenario 4	5mi@0.91MGD, 10% MOS	Standard met for purposes of initial setup
Scenario 5	5mi@1.55MGD, Covar	Standard met for flows greater than 4.53 MGD
Scenario 6	5mi@1.55MGD, dec SOD	Standard met for flow of 2.10 MGD
Scenario 7	5mi@1.55MGD, Covar&dec SOD	Standard met for flows greater than 4.53 MGD
Scenario 8	5mi@1.55MGD, inc SOD	Standard met for flow of 1.19 MGD
Scenario 9	5mi@1.55MGD, inc fCBOD rate	Standard met for flow of 1.37 MGD
Scenario 10	5mi@1.55MGD, dec fCBOD rate	Standard met for flow of 1.80 MGD
Scenario 11	5mi@1.55MGD, no photo/resp	Standard met for flow of 1.96 MGD
Scenario 12	5mi@3.13MGD, dec eff BOD_5	Standard met for purposes of initial setup
Scenario 13	5mi@3.81MGD, dec eff NH_3 & BOD_5	Standard met for purposes of initial setup

6.3 SCOPING MODEL RESULTS SUMMARY AND CONCLUSIONS

Modeling results and conclusions are summarized under each primary study question below:

1. Is there any existing assimilative capacity in the Caney River upstream of the Chickasaw WWTP which would support a secondary effluent discharge location?

The scoping level model would indicate that there is some assimilative capacity assuming that background assumptions for water quality are correct and the representation of the Caney River stream hydraulics and kinetic processes in the QUAL2K model are reasonably accurate. The baseline run for existing conditions predicts that summer critical DO would be expected to remain above 5.85 mg/L above the dam which would provide for some waste assimilation considering a 5 percent or 10 percent MOS on the DO standard (e.g., 5.25 mg/L DO or 5.50 mg/L DO respectively).

2. If so, what is the general magnitude of that assimilative capacity, and what impacts do various upstream discharge scenarios have on the stream and the Chickasaw WLA?

The scoping model results would suggest that there is not enough assimilative capacity to discharge 4.53 MGD of treated wastewater with effluent concentrations of 10 mg/L BOD_5 and 1 mg/L NH_3 (i.e., equivalent to modeled WLA for existing Chickasaw outfall). For these equivalent effluent concentrations, the base model setup predicted that a maximum of 1.55 MGD could be assimilated to meet the DO standard with a 5 percent MOS. The allowable effluent volume would drop to 0.91 MGD for these effluent concentrations if a 10 percent MOS is applied.

3. How sensitive are the scoping-level predictions to key parameter assumptions and what are the implications for facility planning?

Because the model has not been calibrated to monitored data, there is considerable uncertainty in model assumptions. Some of that uncertainty is reduced by having monitoring and modeling data for the downstream portion of the Caney that may also be representative of what to expect upstream. The sensitivity analysis scenarios that were run in the QUAL2K provide insight into how robust the model predictions are to key parameter assumptions.

Since model assumptions for reaeration using the Churchill method were as low (or lower) as is recommended for modeling, reaeration would likely only be expected to be the same or higher than assumed if measured in the field. If reaeration is higher than assumed, then it may be possible to assimilate more volume of discharge above the water supply intake and meet the water quality standard. For example, application of the Covar reaeration estimation method led to prediction that the full 4.53 MGD could be discharged without violating the DO standard with either level of MOS. However, the potential increase would need to be evaluated with subsequent potential impact on the Chickasaw outfall WLA downstream.

Results of the model were sensitive to other key parameters such as SOD, CBOD decay, and net photosynthesis/respiration rates. However, it should be noted that in all cases tested for these parameters the model predicted some level of available assimilative capacity. Allowable effluent flows that protect the DO standard (with at least a 5 percent MOS recognized) above the water supply intake ranged from a low of 1.19 MGD to a high of 2.10 MGD.

Scenarios 12 and 13 indicated that reductions in effluent CBOD and NH_3 (i.e., reflecting a higher level of treatment than the base assumption) would allow for some additional assimilative capacity should the facility planning process want to examine that further down the road.

Finally, the DO sag identified by the QUAL2E model below Chickasaw occurred 1.2 miles downstream at the end of the current modeled river segments. Therefore it would be advantageous to extend a calibrated QUAL2K model downstream in order to consider expansion of discharge at both upstream and existing outfall locations, and well as updating the WLA for the Chickasaw outfall for effluent flows beyond the 3.97 MGD currently recognized. Since we are now using a steady-state model with MOS recognized for the DO standard rather than the original quasi-dynamic model, there may be additional flexibility at the current site to assimilate more volume than 3.97 MGD.

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Appendix C

Monitoring Study Plan, Caney River TMDL Study

July 2017

Tetra Tech

Monitoring Study Plan
Caney River TMDL Study
For
Chickasaw Wastewater Treatment Plant
Bartlesville, Oklahoma

For:

CITY OF BARTLESVILLE

401 S. Johnstone Avenue

Bartlesville, OK 74003

Prepared By:

TETRA TECH, INC.

7645 E. 63rd Street, Suite 301

Tulsa, OK 74133

CA 2388, EXP. 06/17

July 27, 2017

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Prepared by:

Peter Cada, GISP
Environmental Scientist/Planner

Reviewed & Submitted by:

Srini Sundaramoorthy, P.E.
Sr. Project Manager
Tetra Tech, Inc.
CA 2388, Exp. 06/2017

Seal & Date

1.0 INTRODUCTION

The City of Bartlesville, Oklahoma, has conducted a Facility Planning study for which projected population growth will call for an increase in water usage. Projections indicate 8.5 million gallons per day (MGD) of wastewater will need to be treated at the existing Chickasaw wastewater treatment plant (WWTP) north of the City. The Chickasaw WWTP currently discharges treated effluent at the plant location into the Caney River downstream of a low-head dam. Given the projected growth and wastewater discharge demands of Bartlesville, the City seeks to expand effluent treatment at Chickasaw and potentially add a second discharge location upstream of the existing outfall. The City is exploring options to allocate the second discharge point approximately five to seven miles upstream of the existing water supply intake location above the OK-123 bridge, providing that the Caney River has the assimilative capacity to handle this new inflow (Figure 1). A monitoring and modeling study is being conducted to develop a TMDL and wasteload allocation for the City in this portion of the Caney River.

The Caney River along the reach of interest for this study is impaired for biology based on the results of fish bioassessments in the context of the river's Fish and Wildlife Propagation (FWP) classification of Warm Water Aquatic Community (WWAC) (ODEQ, 2014). The key water quality standard for FWP for the Caney River related to assimilative capacity evaluations is dissolved oxygen (DO) concentration. A desktop QUAL2K model was set up to support preliminary DO modeling analysis of adding a discharge point approximately five to seven miles upstream of the current Chickasaw outfall (Tetra Tech, 2016).

The preliminary desktop modeling analysis indicates a likelihood of assimilative capacity along the Caney River to support a second effluent discharge location, although clarifying the magnitude of the assimilative capacity more accurately will require further field surveys and modeling to reduce existing uncertainty for key modeling assumptions. The preliminary results suggest that the Caney River may be capable of assimilating between 1.19 and 4.53 MGD of effluent at existing waste load allocation (WLA) limits when discharged five miles upstream of the intake. The model is quite sensitive to the prescribed reaeration model, with some sensitivity as well to various DO-related parameters such as carbonaceous biochemical oxygen demand (CBOD) decay rate, sediment oxygen demand (SOD), and net photosynthesis/ respiration rate.

This Study Plan describes the monitoring and modeling study details including objectives, methods, scheduling and quality assurance aspects to ensure high quality data that will accurately represent the existing water quality in this river, support model development, and allow for development of the TMDL for the river and the waste load allocation for the proposed plant expansion. The plan will define the procedures required to collect, handle, and analyze field monitoring data required to characterize existing water quality and parameterize the QUAL-2E model.

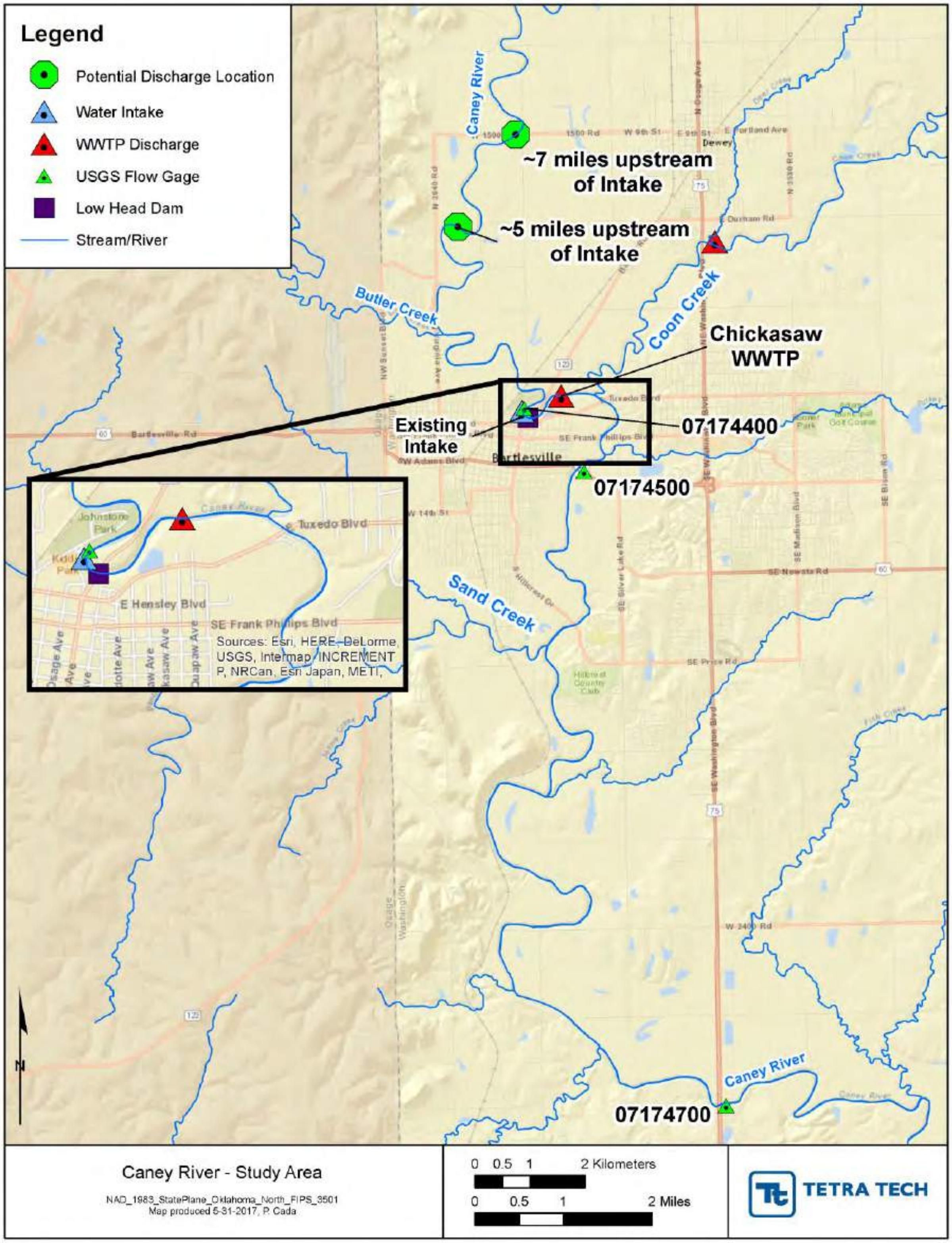


Figure 1. Caney River – Study Area

2.0 PROJECT DESCRIPTION

The study shall be initiated approximately 7 miles upstream of the Chickasaw water intake on the Caney River, and extend downstream approximately 10 to 15 miles below the existing Chickasaw wastewater outfall as necessary (to the Highway 75 Crossing of Caney River) to address the potential impacts of the existing and the possible second effluent outfall discussed above. The annual 7-day minimum flow with a recurrence interval of 2 years (7Q2) condition as estimated by the USGS is considered to represent the critical flow by ODEQ. Sampling will be performed to quantify the instream water quality for model calibration under as close to 7Q2 conditions (recommended by DEQ in 2011 as 20.5 cfs at the USGS gage above the existing Chickasaw outfall) as possible. Additional sampling will be done under higher flow conditions during the late summer or early fall season to address hydrological and seasonal variability (for model validation purposes) and to consider time of travel implications above the existing water supply intake. Recent desktop analysis indicated a median flow condition for Caney River at the USGS gage of approximately 100 cfs providing a target for hydrologic conditions permitting.

Potential nonpoint source (NPS) contributors observed during the field sampling will be noted in the survey logs. Monitoring is scheduled for the summer and fall of 2017 and will involve discrete sampling at multiple locations. The study objectives, sampling locations, monitoring dates, sampling intervals, and monitored parameters are discussed in detail in the following sections.

2.1 MONITORING OBJECTIVES

Objectives of this study are to gather data—observational and measured—to support refinement of the recently developed, scoping-level QUAL2K model (Tetra Tech, 2016). This QUAL2K model provided a preliminary evaluation on the impact of effluent discharge relocation or reallocation between two discharges along the Caney River (the existing WWTP outfall and a single additional upstream location). In addition to the refinement of the recent QUAL2K model, an older QUAL2E model developed in the early 2000s (Tetra Tech, 2003; 2004; 2011) will be updated and transformed into the QUAL2K modeling platform.

The QUAL2K model is a one-dimensional steady-state river water quality model (Chapra et al., 2012). QUAL2K was developed as a modernized and updated version of QUAL2E, the platform used for the previous Caney River modeling work (Brown and Barnwell, 1987). QUAL2K assumes well-mixed stream channels (both vertically and laterally), and can employ a diel, or 24-hour period, heat budget. The model interface operations are programmed in the Microsoft Office macro language Visual Basic for Applications (VBA) and is set-up and run using Excel.

To meet these objectives, the following will be performed:

- Obtain physical measurements to refine QUAL2K model input assumptions that represent Caney River channel width and depth under different baseflow regimes.
- Obtain flow and velocity measurements to calibrate QUAL2K hydraulics components and provide a basis for predicting stream reach velocities in the Caney River under two different baseflow regimes.
- Sample instream water quality under the two different flow conditions in the vicinity of the existing Chickasaw outfall as well as potential future discharge locations that will support development of a dissolved oxygen TMDL for the Caney River:

- Obtain general field measurements for basic water quality indicators such as temperature, DO, conductivity, and pH at multiple locations throughout the study area to characterize stream reaches and tributary conditions.
- Perform detailed "DO sag" study to determine the response of instream dissolved oxygen concentrations downstream of the existing WWTP discharge.
- Collect field samples for lab analysis to characterize parameters associated with assimilative capacity and initial modeling conditions at key locations.
- Collect field measurements and samples for lab analysis to determine diurnal variation in DO concentrations and the extent that algal kinetics influence the River.
- Collect insitu measurements of reaeration and sediment oxygen demand to reduce uncertainty in those parameters shown to be key during preliminary desktop analysis.

2.2 SAMPLING COMPONENTS

Five primary monitoring components are required to meet the objectives of the monitoring plan: 1) hydraulic studies, 2) sediment oxygen demand and reaeration studies, 3) general water quality characterization, 4) DO sag point analysis, and 5) detailed water quality characterization.

2.2.1 Hydraulic Studies

Hydraulic studies are required to estimate the velocity of the Caney River throughout the study area. Physical channel measurements will be performed at transects throughout the study area to determine the physical channel dimensions. Additionally, dye studies will be performed to estimate stream velocities for use in the estimation of flow/velocity relationships and prediction of travel times. Distribution of dye concentrations will help calculate longitudinal dispersion; peak-to-peak time will support velocity estimates.

Three separate dye sampling events will be performed to estimate the velocities under summer 7Q2 critical conditions (approximately 20.5 cfs at USGS gage 07174400) and during typical fall flows (targeting roughly 100 cfs to represent median flow). The timing of these studies will require that no significant rainfall events (> 0.5 inches) have occurred in the previous seven days, the Army Corps of Engineers can maintain the desired discharge releases from the upstream reservoirs, and the river has reached steady-state flows during the sampling period.

Multiple individual dye studies will be performed during each time-of-travel sampling event. Due to the large study area (> 22 miles in length) and the slow travel time during low flow conditions, it is not practical to track one dye injection through the entire study segment. For this reason, sampling will occur in several distinct sections using YSI Autologger to measure time to peak concentration of the different dye releases; 1) from the crossing of W 1400 Rd (west of Dewey) down to the low head dam and exiting water intake, 2) from just below the low head dam (but above the Chickasaw Plant outfall) to the Rice Creek Road crossing of Caney River, and 3) from the Rice Creek road crossing of Caney River to the Highway 75 crossing of Caney River. Past reconnaissance surveys and review of the City of Bartlesville Flood Insurance Study (FEMA, 1992) show significantly different flow velocities upstream and downstream of the confluence of the Caney River and Sand Creek. The dye study sites have been selected to represent the different velocities found in the study area along the entire stretch of Caney River (Figure 2).

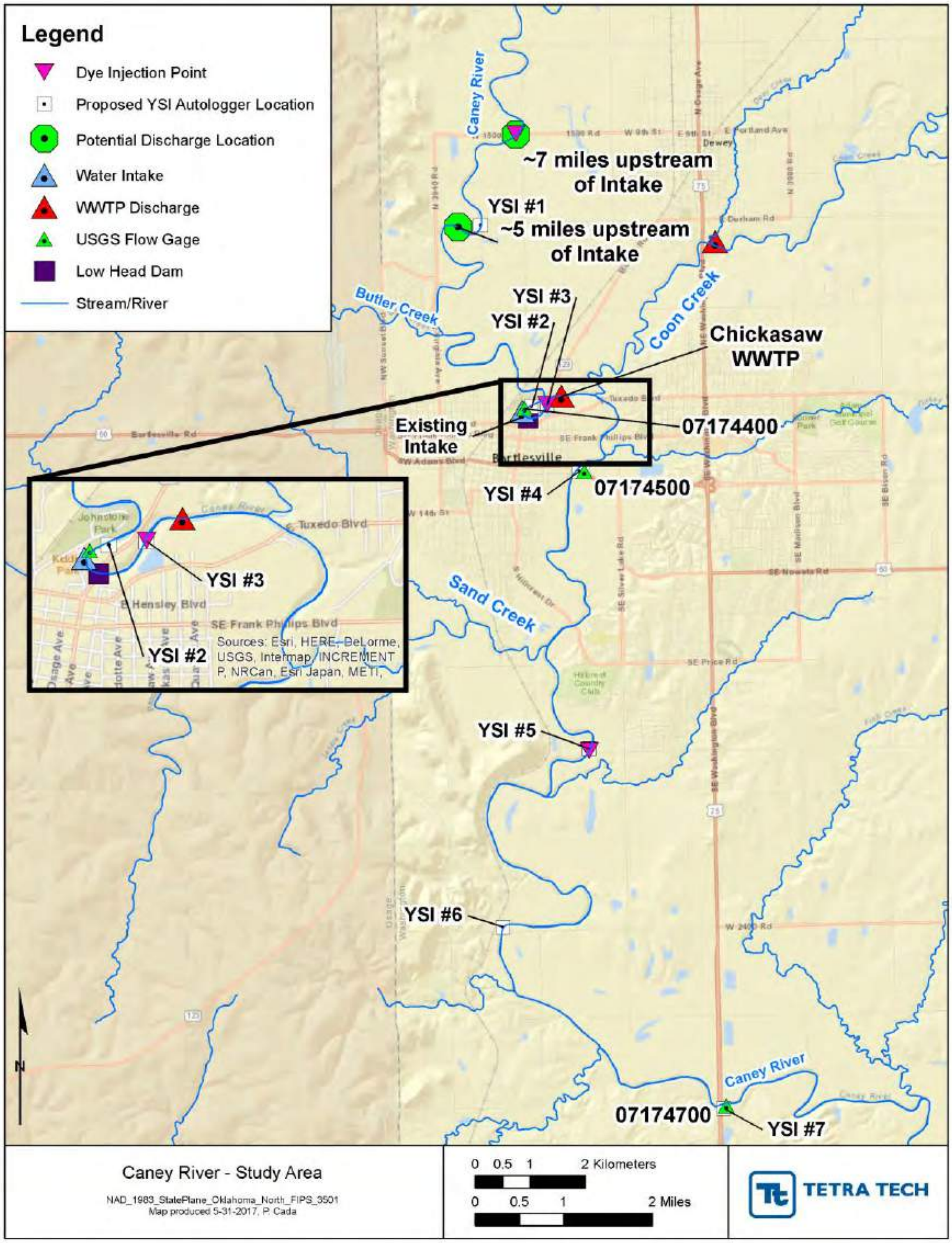


Figure 2. Caney River – Proposed Dye Injection and YSI Autologger Locations

2.2.2 Sediment Oxygen Demand (SOD) and Re-Aeration Studies

SOD will be measured insitu by an SOD specialist, Dr. Paul Gantzer of Gantzer Water Resources Engineering LLC, at up to 6 locations based upon available resources, accessibility, and water level conditions in the field. Tetra Tech staff will provide assistance to Dr. Gantzer for deployment and collection of SOD chambers to the creek. At each proposed study location, insitu SOD chambers will be deployed in triplicate with a fourth chamber deployed to measure water column oxygen demand (WOD). An example of SOD and WOD chamber deployment is provided in Figure 3. See Appendix A for further details on SOD study and deployment methods.

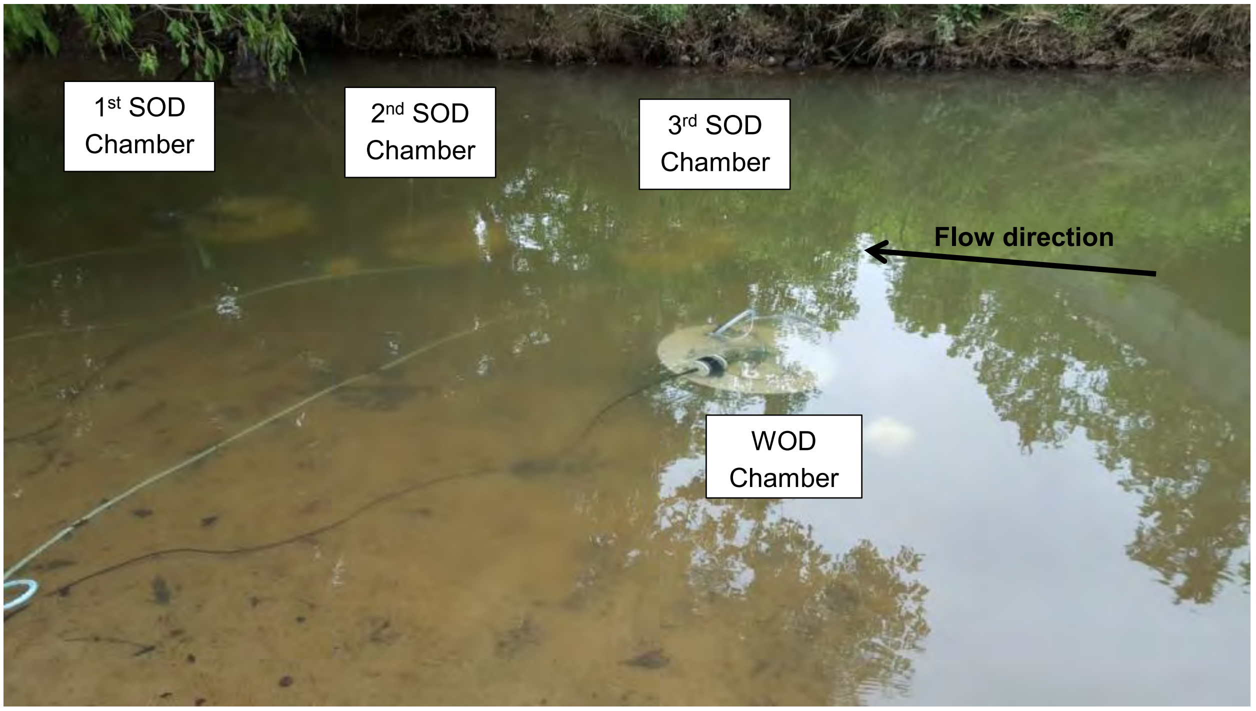


Figure 3. SOD chambers deployed outside of Thomasville, North Carolina - May 2017

Reaeration will be measured by Dr. Mark Koenig and Phil Murphy, expert consultants to Tetra Tech, at strategic locations based upon available resources, accessibility, and water level conditions in the field. Reaeration studies will be conducted working in coordination with Dr. Gantzer and other Tetra Tech field team members.

Reaeration will be measured using a floating diffusion dome (see Figure 4) to make direct and independent evaluation of stream reaeration rate coefficients. The diffusion dome technique was developed by Dr. Koenig and provides for direct measurement of gas exchange between the water column and the atmosphere in a stream reach. The floating diffusion dome uses a forced dissolved oxygen deficit to measure oxygen flux. Accurate and defensible knowledge of stream reaeration rate coefficients are paramount to successful water quality modeling. See Appendix B for further details on the Reaeration study, deployment methods, and standard operating procedures (SOP).

Figure 4. Floating Dome Reaeration Measurement Device (Credit: Koenig)

Proposed SOD measurement locations include the following also seen spatially as black-colored squares in Figure 5:

1. Just upstream of Butler Creek's confluence with Caney River
2. Just downstream of Butler Creek's confluence with Caney River.
3. Just upstream of the Water Intake and USGS gage (07174400).
4. Just up- or downstream of the Tuxedo Blvd. crossing of Caney River.
5. Just up- or downstream of the Hillcrest Drive crossing of Caney River.
6. Just up- or downstream of the W. 2400 Rd. access point of Caney River.

Proposed reaeration measurement reaches will be the following also seen spatially as yellow-colored hexagons in Figure 5:

1. Just upstream of Butler Creek's confluence with Caney River
2. Just downstream of Butler Creek's confluence with Caney River.
3. Just upstream of the Water Intake and USGS gage (07174400).

Preliminary field reconnaissance indicates that it may be challenging to access any or all of these locations. Every effort will be made to perform SOD measurements at all 6 proposed SOD measurement locations, and the three proposed reaeration locations. In the event that reaeration measurements are not achievable in this system due to accessibility by required boat and trailer and/or water level constraints (too shallow for boat and motor), other field data (BOD5 from grab samples and diurnal DO measurement curves) can be used to calibrate subsequent modeling to reaeration dynamics observed in the field data sets.

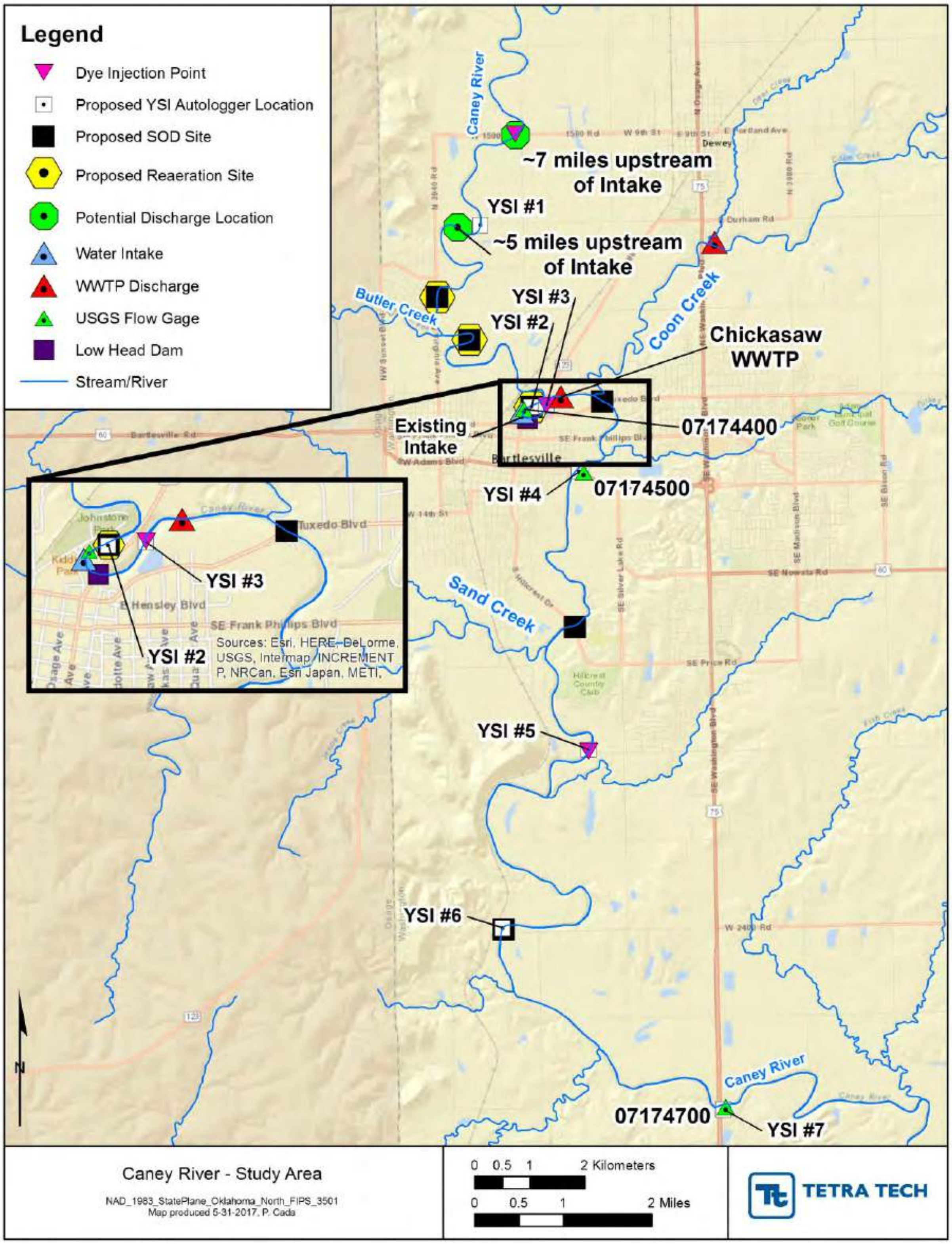


Figure 5. Proposed Dye Injection, YSI Autologger, and SOD and Reaeration Study Locations

2.2.3 General Water Quality Characterization

Field measurements for general water quality parameters (temperature, pH, conductivity, DO) will be performed at approximately 300 locations, intensive survey sites, and at the mouth of significant tributaries to the Caney River under two different flow conditions. Measurements will be performed using handheld instruments and all pertinent data will be recorded in a field log. This sampling will be used to characterize the overall water quality in the study area, identify potential areas of DO sag along the study reach, and to identify changes in water quality which would indicate previously unidentified pollutant sources. Sampling for these parameters will begin approximately 7 miles upstream of the existing water intake where Caney River crosses under West 9th St. (W. 1500 Rd) and will continue all the way to Caney River's crossing under Highway 75 south of Bartlesville at USGS Gage (07174700).

2.2.4 DO Sag Point Analysis

The point in a stream below a WWTP outfall where instream dissolved oxygen concentrations reach their lowest level is referred to as the DO sag point. Field measurements can be used to identify the location of the sag point and observe the distance required for the dissolved concentrations to return to ambient levels. Field results for general water quality parameters will be collected at more frequent locations than for the General Water Quality Characterization (approximately every 250 yards) in the section of the river between the CWWTP discharge, at minimum, to the Adams Road Bridge to verify what was observed during a previous intensive survey in 2002 while supporting the model calibration/ validation efforts.

2.2.5 Detailed Water Quality Characterization (Intensive Survey)

Detailed intensive surveys are required to gain a more complete understanding of the water quality in the Caney River. These surveys combine field observations with the collection of water samples for analysis of parameters such as ammonia and biological oxygen demand to characterize the complete cycle of oxygen demanding wastes. Two separate intensive sampling events will be performed to provide a detailed understanding of instream water quality. As with the hydrological studies, timing will require that no significant rainfall events (> 0.5 inches) have occurred in the previous seven days, the Army Corps of Engineers can maintain the desired discharge releases from the upstream reservoirs, and the river has reached steady-state flows during the sampling period. A total of 10 sampling locations (Figure 6) were selected for the intensive surveys to characterize instream water quality data needed to calibrate and validate the water quality component of the QUAL2K model:

- 7 miles upstream of intake, where Caney River crosses under West 9th St. (W 1500 Rd)
- Butler Creek (at N Virginia Ave) before its confluence with Caney River
- At Water Intake (USGS 07174400, above the low head dam)
- Upstream Chickasaw WWTP (downstream of the low head dam)
- Chickasaw effluent
- Downstream of Chickasaw WWTP
- Coon Creek before its confluence with Caney River (near Tuxedo Blvd and Caney River)
- Near USGS 07174500, at Highway 60 (SE Adams Blvd)
- Sand Creek before its confluence with Caney River (at Keeler Ave, N 3946 Rd)
- Downstream end of study reach at USGS 07174700 (Caney River crossing under Hwy 75)

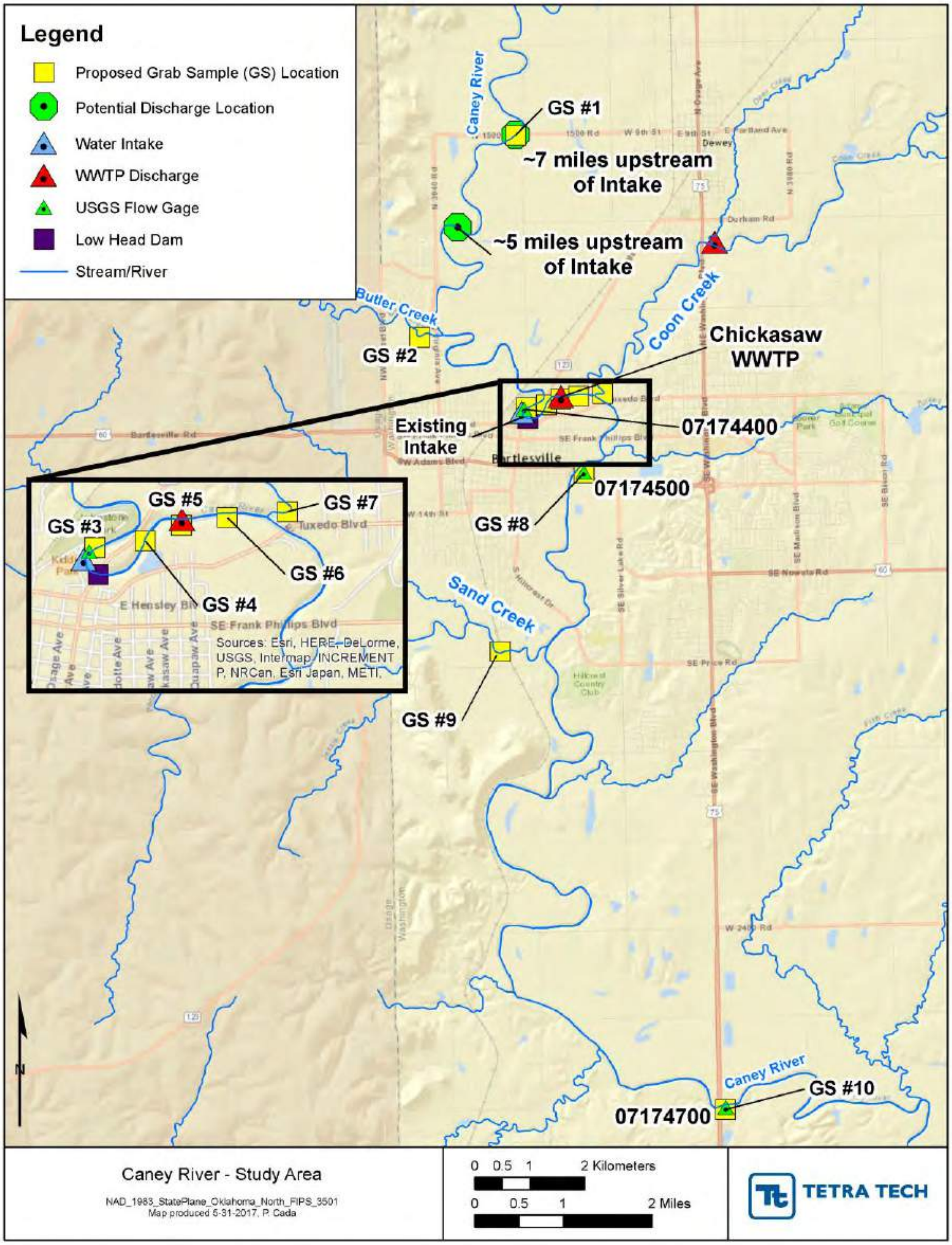


Figure 6. Proposed Water Quality Grab Sample Locations

Sampling locations were selected based on proximity to stream reaches of interest, existing and potential point source loads and tributaries, safety and accessibility, significant changes in channel morphology, and knowledge of where the dissolved oxygen sag occurs.

The intensive surveys will consist of field measurements as well as the collection of water quality samples for lab analysis (Table 2-1). The field measurements include the same general water quality monitoring performed at the transect locations as well as long-term sequential samples of pH, DO, temperature, and conductivity at YSI Autologger locations to help characterize diurnal fluctuations.

Table 2-1. Intensive Survey Parameter List

Variable	Number of Surveys	Sampling Frequency	Sampling Locations	Total Samples
Temperature	2	2/Day[1] 4/Day[1]	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
Dissolved Oxygen	2	2/Day[1] 4/Day[1]	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
Conductivity	2	2/Day[1] 4/Day[1]	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
pH	2	2/Day[1] 4/Day[1]	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
BOD5	2	2/Day 4/Day	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
BOD20	2	2/Day 4/Day	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
CBOD5 (Filtered)	2	2/Day 4/Day	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
CBOD20 (Filtered)	2	2/Day 4/Day	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
CBOD5 (Unfiltered)	2	2/Day	All	40
CBOD20 (Unfiltered)	2	2/Day	All	40
Kjeldahl-N	2	2/Day 4/Day	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
NH_3	2	2/Day 4/Day	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
NO_3 - NO_2	2	2/Day 4/Day	GS#1, #2, #3, # 9, #10 GS#4, #5, #6, #7, #8	20 40
Total Phosphorus	2	2/Day	All	40
Orthophosphorus	2	2/Day	All	40
TSS	2	2/Day	GS#1, #3, #5, #6, #7, #8	24
TDS	2	2/Day	GS#1, #3, #5, #6, #7, #8	24
TOC	2	2/Day	GS#1, #3, #5, #6, #7, #8	24
Chlorophyll *a*	2	2/Day	GS#1, #3, #4, #6, #8	20

[1] – Grab samples at general stations will be supplemented by continuous (diurnal) monitoring at YSI logger stations

2.3 MONITORING SCHEDULE

Water quality monitoring of the Caney River is expected to occur during the period from late July 2017 to September 2017. As discussed, two sampling surveys will be performed to collect data for the calibration and validation stages of the model development. The QUAL2K predicts water quality under steady-state conditions. The monitoring events specified in this plan are designed to capture instream water quality to aid in the calibration of the model during base flow conditions.

The first survey is likely to occur in late July/early August to capture instream conditions during critical conditions. The second survey will be performed in the late summer or early fall to provide estimates of water quality under higher flow conditions to afford statistical power to the study. For purposes of this study, no appreciable rainfall (> 0.5 inches) shall have occurred in the seven days prior to the sampling events. Rainfall recorded at the Frank Philips Airport will be monitored to determine whether this limit has been exceeded. Rainfall events which occur during the sampling events will be considered on a case-by-case basis to determine whether they will interfere with the objectives of the study.

2.4 HEALTH AND SAFETY PLAN

A Health & Safety Plan (HASP) will be developed for each planned field sampling event. The purpose of the HASP is to guide appropriate actions while conducting field assessments. It includes emergency contact information, local hospital and emergency room information, and daily HASP-related checklists to be completed prior to field work. It is the responsibility of each Tetra Tech employee participating in the field reconnaissance to implement and familiarize yourself with the contents of HASP to keep safe in the field and to know which procedures to follow and forms to fill out if necessary.

3.0 PROJECT ORGANIZATION AND RESPONSIBILITY

Srini Sundaramoorthy is the overall Project Manager for the full water and wastewater planning project for the City of Bartlesville. His responsibilities for this monitoring and modeling study will include coordinating with the technical lead and field leader for all phases of the project. He will provide senior review of all deliverables, and maintain contact with local and state agencies regarding the study. He will coordinate contact with local and state organizations for notification of dye studies, and synchronization of sampling and lake releases with the Corps of Engineers, as well as discussions with the City and DEQ regarding the study.

Trevor Clements is the Technical Lead for the monitoring and modeling study. He will coordinate closely with the overall Project Manager and oversee the technical aspects of the field sampling and modeling activities.

Peter Cada, as Field Study Coordinator, will lead the field team responsible for the dye tracing studies, operation of YSI Autologger data sondes, and collection general water quality longitudinal profiling data. He will provide direction to the teams conducting physical channel measurements, and conducting water chemistry grab sampling. He will act as the Quality Assurance Officer (QAO) and will determine the standard operating procedures used, specify and check calibration procedures, oversee data recording and reporting, and perform internal QA/QC.

Hillary Nicholas will be the Lead Modeler for the project. She will be responsible for coordination with the sampling team to ensure monitoring meets the model input and calibration/validation requirements. She will act as a second QA/QC level to ensure monitoring data meet quality criteria.

Jon Butcher will provide senior review of the monitoring plan, modeling report, and other project deliverables to assure technical accuracy and completeness.

4.0 QA OBJECTIVES FOR MEASUREMENT DATA

The primary data quality objective of this sampling and analysis effort is to ensure that the data collected provide results that are representative of the sampled environment and are scientifically and legally defensible. Criteria commonly used to quantify the quality of measurement data include accuracy, precision, completeness, representativeness, and comparability. Accuracy is the degree of difference between the reported values and the true value. Precision refers to degree of consistency among separate measures of a uniform parameter, substance, or object, i.e., the ability to obtain equivalent results when analyzing replicate samples. Completeness is defined as the percentage of measurements judged to be valid compared with the total number of measurements made. Representativeness refers to the ability to extrapolate from measurements on the parameters for a system to the range of properties typically occurring for the system. Comparability expresses the confidence for comparing one data set with another, which can be affected by sample collection and handling techniques, sample matrix type, and analytical methods used. Data sets can be compared with a high degree of confidence only when their precision and accuracy are known.

Dissolved oxygen, temperature, conductivity, and pH, will be measured in the field using a YSI multi-parameter instrument, and a YSI Optical DO (ODO) meter as well for sampling replication. Precision of the measured parameters using the instruments will be evaluated by completing duplicate shortly spaced timed measurements on at least 5 percent of the individual measurements completed for each analyzed water quality parameter. Precision will be determined to be adequate if the results from the pairs of samples agree within 10 percent.

The analytical methods and detection limits for remaining water quality parameters are shown in Table 4-1. These objectives are based on regulatory and technical requirements of the project, existing method validation studies that include replicates, standards, and calibrations procedures, and knowledge of the measurement system used.

In general, precision and accuracy objectives are specified in method descriptions provided by manufacturers for monitoring equipment and test kits that will be used during this study. These specifications indicate the relative analytical performance required for this project. (Field values of precision and accuracy are usually lower than those obtained by laboratory studies because of matrix interference and the confounding effects of other pollutants.) Standard instrument procedures will be used to eliminate potential interference with water testing procedures.

Table 4-1. Summary of Analysis Methods and Detection Limits

Measured Parameter	Method	Detection Limit
Carbonaceous Biological Oxygen Demand – 5 day (CBOD5), filtered (mg/L)	Standard Methods 5210B	2 mg/L
Carbonaceous Biological Oxygen Demand – 5 day (CBOD5), unfiltered (mg/L)	Standard Methods 5210B	2 mg/L
Carbonaceous Biological Oxygen Demand – 20 day (CBOD20), filtered (mg/L)	Standard Methods 5210B	2 mg/L
Carbonaceous Biological Oxygen Demand – 20 day (CBOD20), unfiltered (mg/L)	Standard Methods 5210B	2 mg/L
Biological Oxygen Demand – 5 day (BOD5), filtered (mg/L)	Standard Methods 5210B	2 mg/L
Biological Oxygen Demand – 5 day (BOD5), unfiltered (mg/L)	Standard Methods 5210B	2 mg/L
Biological Oxygen Demand – 20 day (BOD20), filtered (mg/L)	Standard Methods 5210B	2 mg/L
Biological Oxygen Demand – 20 day (BOD20), unfiltered (mg/L)	Standard Methods 5210B	2 mg/L
Nitrate Nitrogen (mg/L)	EPA 353.2	0.1 mg/L
Nitrite Nitrogen (mg/L)	EPA 353.2	0.1 mg/L
Ammonia Nitrogen	Standard Methods 4500 NH3	0.1 mg/L
Total Kjehldahl Nitrogen (mg/L)	EPA 351.2	0.5 mg/L
Organic N	EPA 350.1 + 351.2	0.2 mg/L
Total Phosphorus (mg/L)	EPA 365.4	0.1 mg/L
Orthophosphorus (mg/L)	EPA 365.2	0.1 mg/L
Total Organic Carbon (mg/L)	Standard Methods 5310C	1 mg/L
Total Dissolved Solids (mg/L)	Standard Methods 2540C	5 mg/L
Total Suspended Solids (mg/L)	Standard Methods 2540D	5 mg/L
chlorophyll *a* (ug/L)	Standard Methods 10200H	5 mg/m3

5.0 SAMPLING PROCEDURES

5.1 HYDRAULIC STUDY

The hydraulic study will consist of characterization of the physical channel and flow/velocity estimation using a Rhodamine-WT dye tracer.

5.1.1 Physical Channel Measurements

The hydraulic studies include physical channel measurements (width/depth/flow) at transects in the Caney River and major tributaries. This will involve choosing transect locations, establishing benchmarks on the streambanks such that measurements can be repeated at the exact location, and stringing a temporary line across the stream channel to ensure channel depth measures are maintained along the transect. All stream profile locations will be identified by wooden stakes or flagging on both sides of the stream transect. These locations will be cross-referenced by natural features of the site, relative position to bridge crossings, flagging on trees, and GPS as needed. The specific tools used at each profile site will be dictated by the site conditions. All transect sites will be photographed to further document their locations. Twelve locations along the creek will be measured for channel physical characteristics (Figure 7).

The first step will be to place a measuring tape perpendicular to the flow over the stream from one top of stream bank to the other. The height from ground to the measuring tape will be measured at frequent distances across the channel to map a cross-sectional profile of the creek at this location. Height from water surfaces will also be recorded (Figure 8). Time of day shall be noted at the beginning of height measurements and at the conclusion of measurement.

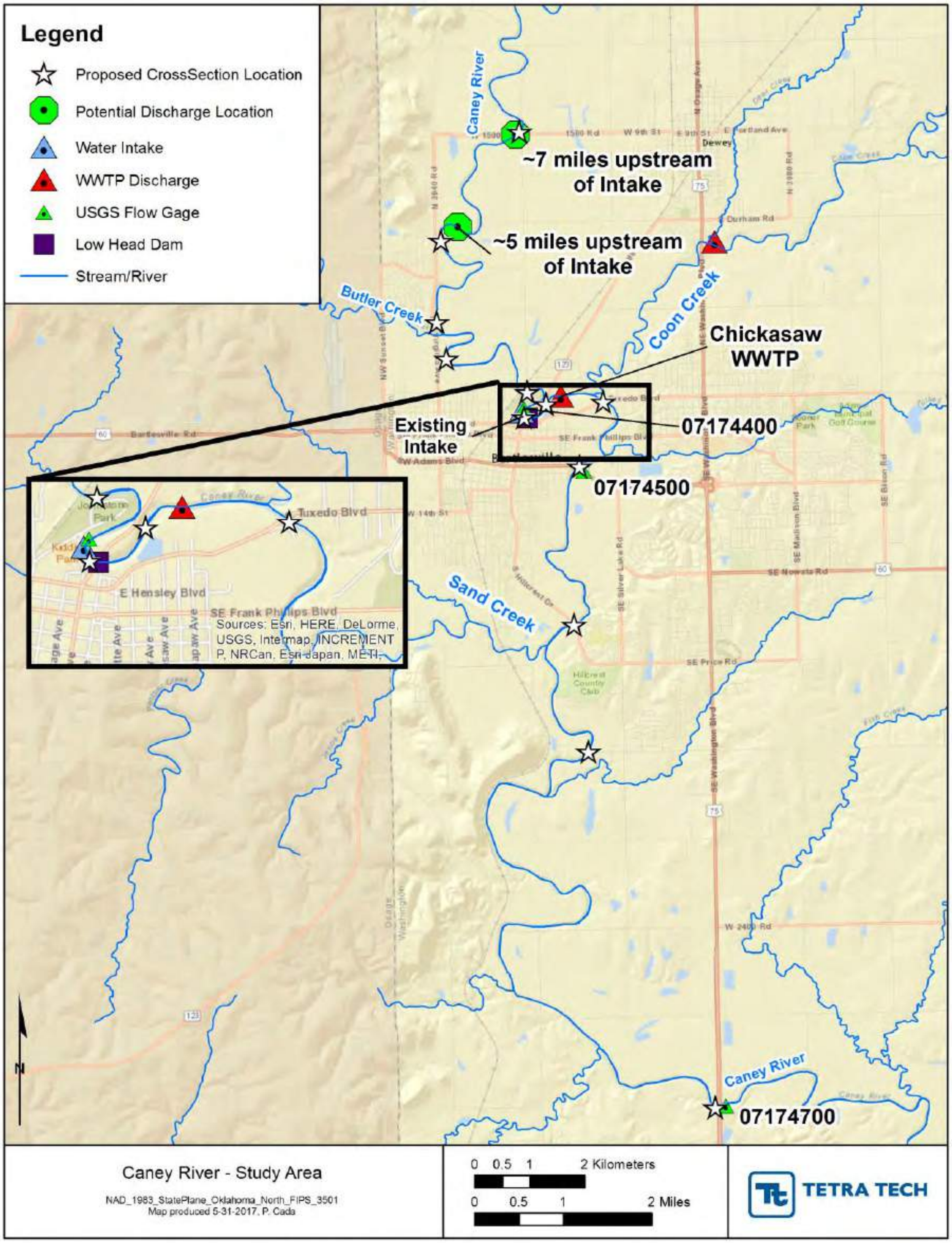
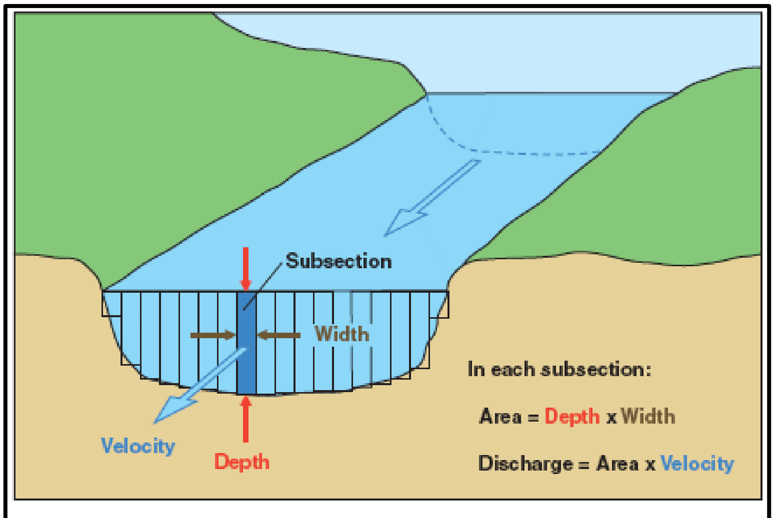


Figure 7. Proposed Physical Cross-section Measurement Locations (excluding velocity)

Figure 8. Example of Cross-section Field Measurements

Before dye injections commence for time-of-travel estimates, a similar cross-section measurement will be made to estimate velocity and discharge (see Figures 2 and 6 for dye injection locations). Velocity will be measured at the same locations across the channel where heights of water surface and stream bed were recorded. All of this measured data provides a detailed velocity profile across the stream cross-section. The velocity profile and water depth profile measurements allow estimates of flow (discharge) at the time of field measurement. A conceptual diagram of this type of effort provided by the USGS (https://water.usgs.gov/edu/images/streamflow2-1.gif) is provided below (Figure 8).



Current-meter discharge measurements are made by determining the discharge in each subsection of a channel cross section and summing the subsection discharges to obtain a total discharge.

Figure 9. Conceptual Diagram of Field measurements Used for Stream Discharge Estimates

5.1.2 Rhodamine-WT Dye Studies

In order to define the river hydraulics, time-of-travel study using Rhodamine-WT dye tracer will be needed. An instantaneous release of the dye will be made at 3 locations previously identified (see Figures 2 and 5). The presence of dye will be followed by visual observation and by the use of deployable, portable fluorometers. The initial time the dye is released and its arrival at key reference sites along the Caney River will be recorded. Measurements from specified downstream locations will be recorded and analyzed to determine the travel time of the leading edge, peak, and trailing edge of the dye plume. The downstream dye injection site will be released first to avoid contamination from the dye injection release from upstream injection sites.

The Project Manager will notify key local and state agencies to prepare for public inquiries or complaints which may arise as a result of temporary stream discoloration from the dye study.

5.2 WATER QUALITY SAMPLING

Water quality sampling will include the measurement of field parameters as well as the collection of stream samples for more detailed lab analysis. Field measurements will be made by a multi-parameter probe measuring dissolved oxygen, percent saturation, temperature, conductivity, and pH. In areas of shallow flow, the field measures will be made at 0.6 the total depth of flow. In areas where transects are wide, multiple 0.6 measurements across the width of the stream will be made to characterize the stream. Instantaneous velocity readings will accompany these measurements. In deeper water, the probe and velocity measurements will be made at 0.2 and 0.8 of the depth of flow. All field measurements will be documented in field notes and submitted and discussed with the modeling team.

5.2.1 General Water Quality Sampling

General water quality parameters will be measured at each location for the parameters described in Section 2.2.3. These parameters will be measured in the field using handheld equipment and will be taken at all transect locations as well as intensive study locations to provide general indicators for water quality throughout the watershed. As previously described in Section 4, at least 5 percent of the general water quality measurements will be repeated within a short interval as duplicate measurements to ensure precision of the readings.

5.2.2 DO Sag Point Estimation

General parameters will also be measured at several locations (approximately 250-yard increments) in the section between the Hwy 123 Bridge and the Adams Road Bridge (2.15 river miles). This will provide verification of the “DO sag” below the WWTP discharge point compared with intensive survey results observed in 2002. The dissolved oxygen profile produced by analysis of this data will refine estimates of the rate at which the instream dissolved oxygen returns to ambient levels.

At low flow, the field team will wade upstream and downstream to obtain this measurement. At high flow, a small boat may be required. The 250-yard increments will be referenced by natural features and GPS handheld units. At critical increments and areas of rapidly changing DO concentrations, supplemental determinations will be made. The diurnal effects on dissolved oxygen will be recorded at 7 sampling locations as detailed above in Section 2.2 and Figures 2 and 5 by use of YSI multi-parameter probes.

5.2.3 Intensive Water Quality Surveys

Two intensive surveys will be performed to provide a more detailed understanding of the water chemistry under different flow conditions to: 1) model the river using the first set of survey results, and 2) validate the model using the second set of survey results. Water samples for the parameters listed in Table 2-1 will be collected at 10 locations under two flow regimes.

Sediment along the bed and banks will be inspected and characterized to inform the QUAL2K model. For example, notation will be made at cross-sections and other field sampling locales to determine the sediment characteristics of the bed and banks at each location (e.g., cobble, sandy, muck, silt/clay, etc.).

All samples will be collected, preserved, and transported to the laboratory by the field team. Representative samples will be collected in accordance with methods described in 40 CFR 136 considering the location depth, width, and flow. At least 10 percent of the samples collected for each analyte will be submitted for duplicate analyses to confirm field procedures and laboratory precision. The laboratory will perform QA/QC procedures consistent with NPDES sample requirements.

5.3 FIELD LOG BOOKS

One or more waterproof field log books will be maintained for recording data collection activities performed during the study. The general principle of information recording is that the entries be sufficient to reconstruct the site investigation without reliance on memory. All field measurements from samples collected will be recorded. Wherever a sample is collected or a measurement is made, a detailed description of the sample location will be recorded. Log book entries will include the location of the sampling point, the depth of sample, observed character of the sampled material, any field measurement analyses taken at the site, and other appropriate observations and information.

The following minimum information will be recorded:

- Calibration of field instruments.
- Field observations.
- Sample collection locations (with Unique ID).
- Date and time.
- Field measurements and analysis results.
- Sampling or analysis problems.

To ensure consistency across all sampling sites with all personnel, standardized forms will be used at each sampling location (Appendix A). Each section of the form will be completed on site at each location with an ink pen. Any information not applicable to a certain site will be flagged “na.”

5.4 PHOTOGRAPHS

Digital photographs will be taken to document each sampling location. These photographs will show the orientation to the surrounding area and nearby objects. Photographs will also be taken to document any unusual environmental conditions encountered. For each photograph, the photograph number assigned by the camera, field date, and subject will be logged in the field notebook. Digital photographs will be downloaded to archive folders.

The photographs will also be identified by photographing, on the first image, an identifying sheet containing the Work Assignment No., project name, archive folder, date, and photographer's name.

Digital cameras should always be carried in their cases, and only removed at the time of photographing. A spare set of batteries and backup memory cards should be carried as well.

5.5 CORRECTIONS TO DOCUMENTATION

All original data will be recorded into field logbooks. No field data shall be destroyed or thrown away, even if they are rendered illegible or contain inaccuracies that require a replacement document. The original author will correct errors by crossing a single line through the error, entering the correct information, and initialing the correction. The erroneous information shall not be obliterated. Any subsequent error discovered on a field document will be corrected by crossing out the error with one line, by the person who made the entry. All subsequent corrections must be initialed and dated.

6.0 SAMPLE CUSTODY

A chain-of-custody form will be maintained as the normal procedure to ensure samples are traceable from collection to receipt at the analyzing laboratory. The custody sheet will include the name of the person delivering the samples, the date and time of delivery, project number, collection location, sample ID, date and time of collection, and the number of bottles per set (Appendix B). The chain-of-custody sheet will accompany samples, and a copy of the sheet will be delivered to the project manager.

7.0 CALIBRATION PROCEDURES AND FREQUENCY

A bound instrument logbook will be maintained for recording calibration information for field instruments. This logbook will contain chronological entries that include the identity (name and serial number) of the device being calibrated, and describe routine maintenance, calibration, operational deficiencies, performance notes, and repairs (by reference if appropriate). At a minimum, field equipment will be checked and calibrated at intervals recommended by the manufacturer. If subsequent calibration reveals that the equipment is not operating within accuracy requirements, the calibration frequency will be increased to enhance data reliability. The Field Study Coordinator also may increase the frequency of equipment checking and calibration if faulty readings are suspected. Any equipment that will not calibrate satisfactorily will be removed from the field for repairs, and studies at that site will be terminated until such time that repaired or replacement instruments can be installed.

The multi-parameter probe will be calibrated (conductivity, pH, and dissolved oxygen) once per week, at a minimum. The barometric pressure reading will be checked and adjusted before dissolved oxygen calibration. The dissolved oxygen sensor calibration will be checked at least once per day in the field. At all calibrations and calibration checks, the operator will first record the pre-calibration reading, calibrate, and then verify that the post-calibration reading is correct.

The dissolved oxygen membrane and electrolyte solution will be replaced once per week, at a minimum. The pH electrode junction will be replaced when discolored or at the end of a run, whichever occurs first. The conductivity sensors will be cleaned and serviced if conductivity calibrations indicate any “drift” or inaccuracy in readings.

The fluorometer used in the dye tracing studies will be calibrated to ambient fluorescence values according to the manufacturer’s protocols prior to the collection of any samples containing dye tracer.

8.0 ANALYTICAL PROCEDURES

Materials that may be analyzed under this Work Assignment include routine water quality parameters analyzed using conventional techniques, including field sensors and manufactured test kits. The manufacturer’s SOPs for the instruments and water quality test kits will be followed.

9.0 DATA REDUCTION, VALIDATION, AND REPORTING

The Project Manager and the QAO will be responsible for verification checks for internal consistency, transmittal errors, laboratory protocols, and QC measures specified in this plan. Results of all analyses will be checked for compliance with instrument calibration, relevant instrument tuning and performance information, method blanks, quantification, and expected concentrations based on historical monitoring data from this river system.

The integrity of collected data will be maintained and validated following several procedures, depending on the source of the data. All pages of the field logs will be copied and these copies mailed by the Field Study Coordinator, Peter Cada, to the Project Manager prior to departure of the field personnel for their home offices. The Field Study Coordinator will be responsible for entering all data from the field logs into appropriate data files. These entries will be validated by a manual review of all entries by an independent data reviewer not involved in the original data entry, as identified by the Project Manager. Upon entry and validation of the study data into appropriate computer files, these files will be transmitted by the Field Study Coordinator to the Project Manager or a person designated by the Project Manager for final data summary and analysis.

All raw data collected during the study will be included in appropriate appendices accompanying the final report for this study. The summary and analyses of the collected study data will include routine statistical summaries for the sample results (i.e., means, ranges, standard deviations), individual plots, and qualitative assessments of changes in concentrations for the monitoring parameters over the time of the study at each sampling station.

10.0 INTERNAL QUALITY CONTROL CHECKS

Duplicate samples will provide precision information about the measurement system as a whole. That is, they will provide an integrated, precision measurement for the sample collection, handling, and analysis procedures as a combined system. For each analyte, at least one duplicate sample will be submitted for each 10 field samples collected. Duplicate laboratory analyses with a relative percent difference (RPD) greater than 10 percent will initiate a corrective action sequence (see Section 14.0).

11.0 PERFORMANCE AND SYSTEM AUDITS

The project manager will review field documentation to ensure that the project adhered to the procedures outlined in this Design Plan and to standard practices. The Quality Assurance Officer (QAO) will review work product quality and will ensure that the project is performed in accordance with approved quality control procedures. In addition, the project team or his designate will review work for technical accuracy and completeness.

12.0 PREVENTATIVE MAINTENANCE

Laboratory instruments and equipment will be maintained according to the schedule and procedures established by the analytical method. Maintenance of field instruments will be based on the manufacturer's instructions and the amount of use that they receive. Maintenance records on each piece of equipment or instrument will be maintained in the logbook specific for this study.

Field sampling and analytical equipment will receive preventative maintenance and calibration in accordance with procedures and guidelines provided by the equipment manufacturer. The frequency, acceptance criteria, and source of standards will be performed in accordance with the manufacturer's recommendations. If a particular piece of equipment cannot be calibrated to the acceptance criteria in the field, it will be repaired or replaced, if possible. Critical replacement parts will be maintained, as possible, for unexpected equipment malfunction. Backup equipment will be available in case of failure.

13.0 SPECIFIC ROUTINE PROCEDURES USED TO ASSESS DATA PRECISION, ACCURACY, AND COMPLETENESS

Data quality parameters for precision, accuracy, and completeness have been discussed in previous sections, as have quality control samples, and the frequency with which they will be collected. The quality of data collected during this project will be reviewed by the QAO or by staff designated by the QAO to evaluate its attainment of project DQOs. Data validation will be performed by the Tetra Tech Project Manager or his designate working independently from the field study team.

For each field parameter analyzed, measurements of precision (RPDs) will be evaluated. The results from these analyses will be presented in the study report.

14.0 CORRECTIVE ACTION

A corrective action may be initiated because of the results from QA/QC data quality evaluations or the identification of a problem by the study team. A formal corrective action will include problem identification, responsibility assignment, investigation, action to eliminate the problem, monitoring of the effectiveness of the corrective action, and verification that the problem has been eliminated.

The person identifying a potentially significant problem will notify the project QAO, either directly or through the project manager, regarding the nature of the problem and the action undertaken to correct the problem. Corrective action will include determination of the root cause of the problem, determination of the potential implications to previously completed work, documentation of actions taken to preclude repetition, and correction of the particular problem identified. The QAO will evaluate the problem and its corrective action. The QAO will then assign a sequential number to the action and add it to a status log that also lists the date issued, addressee, date response due, date corrective action due, and date closed. A copy of the action will be transmitted to the program manager, or other responsible authority for corrective action. All such actions will be included as an appendix to the study report.

15.0 QUALITY ASSURANCE REPORT TO MANAGEMENT

The purpose of the Quality Assurance Report is to document the implementation of the QA efforts. Following completion of this field study, all field QA documentation will be submitted for review by the QAO. The QAO will report to the Project Manager the findings of his review. This report will include:

- An assessment of measurement data accuracy, precision, and completeness;
- Results of performance audits;
- Results of system audits; and
- Any significant QA problems and recommended solutions.

These reports will be provided to the Project Manager and to EPA Region 6. Tetra Tech will strive to ensure that these reports contain useful information that is both accessible and actually used to add to the overall quality of the project, and not simply paperwork generated to fulfill a requirement. This report will be included as an appendix with the final study report presented to DEQ/EPA Region 6.

16.0 REFERENCES

Brown, L.C., and T.O. Barnwell, Jr. 1987. The Enhancement Stream Water Quality Models QUAL2E and QUAL2E-UNCAS: Documentation and User Manual. United States Environmental Protection Agency (USEPA). Cooperative Agreement No. 811883. Department of Civil Engineering, Tufts University, Medford, MA.

Chapra, S.C., Pelletier, G.J. and Tao, H. 2012. QUAL2K: A Modeling Framework for Simulating River and Stream Water Quality, Version 2.11: Documentation and User's Manual. Civil and Environmental Engineering Dept., Tufts University, Medford, MA. Steven.Chapra@tufts.edu

ODEQ. 2014. Water Quality in Oklahoma. Prepared by Oklahoma Department of Environmental Quality, pursuant to Section 303(d) and Section 305(b) of the Clean Water Act.

Tetra Tech. 2003. Model Calibration for the Caney River QUAL2E Model: Low Flow Total Maximum Daily Load (TMDL) Study for the Caney River. Prepared for City of Bartlesville, Oklahoma.

Tetra Tech. 2004. Refinement and Application of the Caney River QUAL2E Model: Low Flow Waste Load Allocation (WLA) Study for the Caney River. Prepared for City of Bartlesville, Oklahoma.

Tetra Tech. 2011. Bartlesville WLA Study Technical Addendum. Prepared for City of Bartlesville, Oklahoma.

Tetra Tech. 2016. Caney River QUAL2K Scoping Model near Bartlesville, Oklahoma. Prepared by Tetra Tech, Inc., Research Triangle Park, NC for City of Bartlesville Utility Services. August 23, 2016.

APPENDIX A – SOD STUDY AND DEPLOYMENT METHODS

Chamber Deployment

SOD chambers will be set and flushed for at least 30 minutes before incubation begins. This method has been observed to be a very reliable method to ensure the chambers had consistent water in them while allowing the sediments to settle that were re-suspended during initial placement of the chambers.

The biggest component of achieving successful SOD measurements is to have the chambers sealed, both at the sediments as well as every connection on the chambers themselves. When deploying the chambers in shallower applications, it is important to look for bubbles exiting the chamber, indicating potential leaks that would affect results. It can be difficult making sure all air is removed from the chamber before incubation can begin. Because chamber lids are bolted down in advance, air issues are managed by placing the chambers in the water, inverting them to completely fill with water, then rotating them while underwater to be upright, for subsequent placement on the water body bed surface.

Dissolved Oxygen Probes

Because the probes used to monitor oxygen content are fluorescent DO probes, traditionally designed for waste water applications, they are quite stable and do not experience drift for several months, unlike traditional Clark cell technology. As a result of the very stable nature of the probes, calibration checks will be performed upon completion of the final site.

Operation

Incubation time periods are dependent on each unique site's characteristics and will be run for a period that will allow for adequate time to detect a stable oxygen depletion rate. Data loggers are programmed to collect data every two (2) minutes. Flow and oxygen content will be manually verified every 30 minutes.

Materials

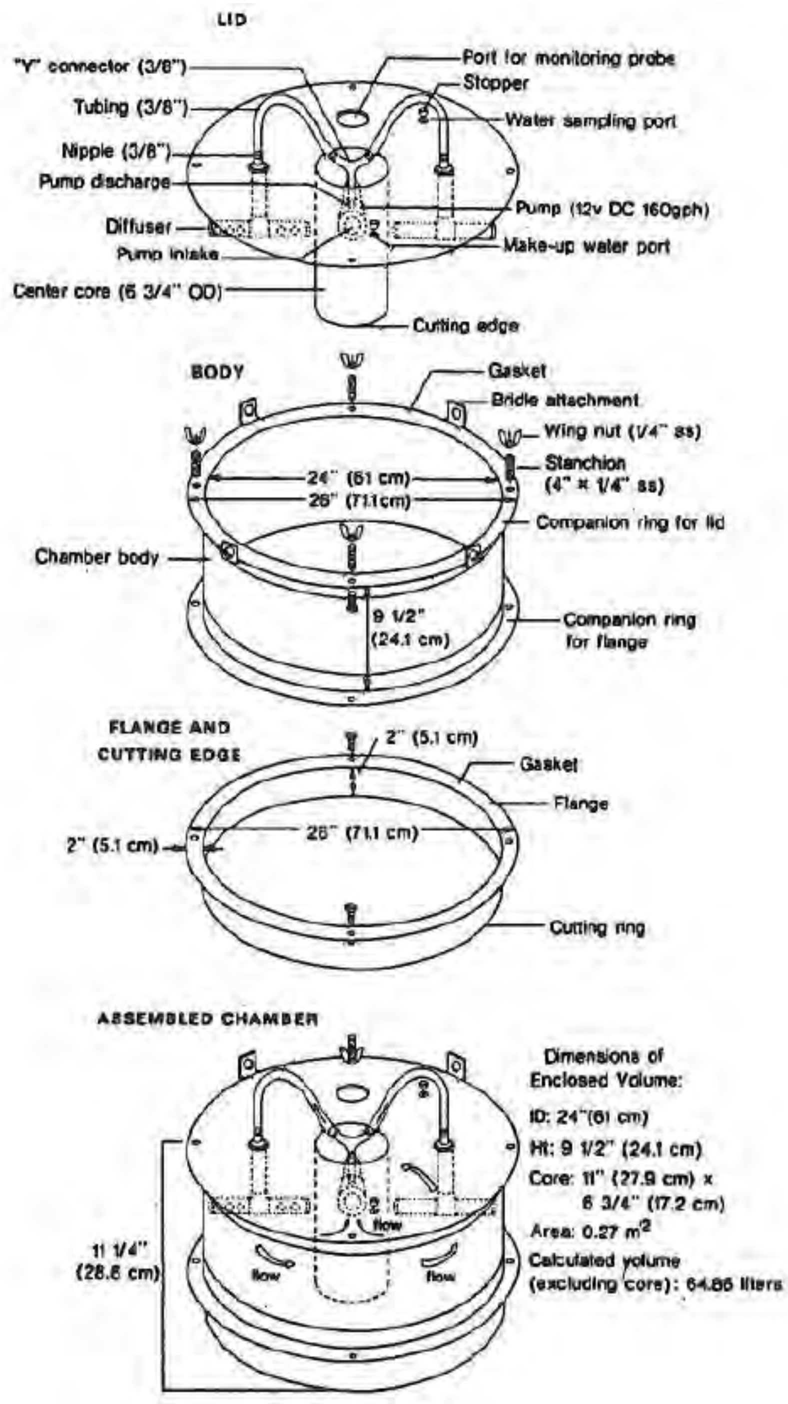
The in-situ SOD chambers are the same chambers and components used by Murphy and Hicks (1985) (see picture below) with the exception of the DO probes, which will be InSite IG optical DO probes connected to an Instrumentation North West (INW) data logger to record DO readings on a two (2) minute interval.

The chambers will be configured to allow suction from the water column with chamber ports open during the flushing phase. For the circulation period, suction will be drawn from inside the chamber and the chamber ports previously opened will be closed. DO in the chamber will be continuously monitored during the flushing phase to ensure adequate DO levels are achieved before placing the chamber into the circulation (incubation) mode. Once the DO is observed to be stabilized in the chamber, they will be left unattended for at least one hour. After the first hour of incubation, data will be collected and evaluated. The chambers will be left in an incubation state until sufficient data identified a linear DO depletion rate.

References

Murphy, P.J., and D.B. Hicks 1985. In-situ Method for Measuring Sediment Oxygen Demand. "Sediment Oxygen Demand, Processes, Modeling, and Measurement", Institute of Natural Resources, Athens, GA.

Schematic of in-situ SOD chamber



APPENDIX B – RE-AERATION METHODS AND SOP

Tetra Tech's consulting experts will use a floating diffusion dome to make direct and independent evaluation of stream reaeration rate coefficients. A copy of the Standard Operating Procedure that will be applied for the reaeration study using this technique follows.

COPY

Region 4 **U.S. Environmental Protection Agency** **Science and Ecosystem Support Division** **Athens, Georgia**
OPERATING PROCEDURE
Title: **Reaeration Measurement by Diffusion Dome**
Effective Date: May 30, 2013 / **Number:** SESDPROC-505-R3
Authors
Name: Hunter Johnson Title: Environmental Engineer Signature: [signature] Date: 5/28/13
Approvals
Name: John Deatrick Title: Chief, Ecological Assessment Branch Signature: [signature] Date: 5/29/13
Name: Bobby Lewis Title: Field Quality Manager, Science and Ecosystem Support Division Signature: [signature] Date: 5/28/13

Revision History

The top row of this table shows the most recent changes to this controlled document. For previous revision history information, archived versions of this document are maintained by the SESD Document Control Coordinator on the SESD local area network (LAN).

History	Effective Date
SESDPROC-505-R3, *Reaeration Measurement by Diffusion Dome*, replaces SESDPROC-505-R2. **General:** Corrected any typographical, grammatical, and/or editorial errors. Throughout the document, the term "stream" was omitted. **Title Page:** Changed the EAB Chief from Bill Cosgrove to John Deatrick. Changed the FQM from Laura Ackerman to Bobby Lewis. **Revision History:** Changes were made to reflect the current practice of only including the most recent changes in the revision history **Section 1.2:** Added the following statement to the first paragraph: "Mention of trade names or commercial products in this operating procedure does not constitute endorsement or recommendation for use." In the second paragraph, "stream" was replaced with "water body." **Section 3.2:** In the first paragraph, the second and third sentence was combined. Added EAB acronym in the first sentence of second paragraph. Omitted the second sentence in the second paragraph and replaced it with the following language: "Currently, EAB uses a luminescent (LDO) probe connected to a digital display but other technologies are available and can be utilized." **Section 4.1**: Omitted the last sentence in the first paragraph. Replaced "every 15 – 30 minutes" with "concurrent with monitoring data readings" in the second sentence of the fourth paragraph.	May 30, 2013
SESDPROC-505-R2, *Reaeration Measurement by Diffusion Dome*, replaces SESDPROC-505-R1.	November 6, 2009
SESDPROC-505-R1, *Reaeration Measurement by Diffusion Dome*, replaces SESDPROC-505-R0.	November 1, 2007
SESDPROC-505-R0, *Reaeration Measurement by Diffusion Dome*, Original Issue	February 05, 2007

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1 General Information

1.1 Purpose

The purpose of this operating procedure is to document both general and specific methods and considerations to be used when measuring reaeration using a diffusion dome.

1.2 Scope/Application

This document describes both general and specific methods to be used by field investigators when obtaining data for the purposes of determining reaeration using a diffusion dome. In the event that Science and Ecosystem Support Division (SESD) field investigators determine that any of the procedures described in this section are either inappropriate, inadequate or impractical for a given site or station or that another procedure must be used to obtain a representative measurement, the variant procedure will be documented in the field log book, along with a description of the circumstances requiring its use. Mention of trade names or commercial products in this operating procedure does not constitute endorsement or recommendation for use.

Reaeration is the rate at which atmospheric oxygen diffuses across the air-water interface of the surface of a water body.

1.3 Documentation/Verification

This procedure was prepared by persons deemed technically competent by SESD management, based on their knowledge, skills and abilities and has been tested in practice and reviewed in print by a subject matter expert. The official copy of this procedure resides on the SESD local area network (LAN). The Document Control Coordinator (DCC) is responsible for ensuring the most recent version of the procedure is placed on the LAN and for maintaining records of review conducted prior to its issuance.

1.4 References

American Public Health Association (APHA), American Waterworks Association (AWWA), and the Water Environment Federation (WEF). 1998. Standard Methods for the Examination of Water and Wastewater, 20th Edition. Washington, D.C.

Buck, A. L. 1981. New Equations for Computing Vapor Pressure and Enhancement Factor. National Center for Atmospheric Research. Boulder, Colorado.

Cavinder. 2002. Reaeration Rate Determination with a Diffusion Dome. United States Environmental Protection Agency, Region 4 Science and Ecosystem Division, Ecological Assessment Branch.

Chapra, S.C. and Canale R.P. 1998 Numerical Methods for Engineers With Programming and Scientific Applications. The McGraw-Hill Companies, Inc., New York.

Copeland, B.J. and Duffer, W.R. 1963. Use of a Clear Plastic Dome to Measure Gaseous Diffusion Rates in Natural Waters. Limnol. Oceanogr. 9:494-499.

Juliano, D. W. 1969. Reaeration Measurements in an Estuary. Journal of the Sanitary Engineering Division, ASCE. 95(SA6):1165-1178.

SESD Operating Procedure for Logbooks, SESDPROC-010, Most Recent Version.

SESD Operating Procedures for Measurement of Dissolved Oxygen, SESDPROC-106, Most Recent Version.

SESD Operating Procedure for Global Positioning System, SESDPROC-110, Most Recent Version.

SESD Operating Procedure for *In situ* Water Quality Monitoring, SESDPROC-111, Most Recent Version.

SESD Operating Procedure for Reaeration Measurement using Krypton Gas, SESDPROC-506, Most Recent Version.

USEPA. Safety, Health and Environmental Management Program Procedures and Policy Manual. Science and Ecosystem Support Division, Region 4, Athens, Georgia. Most Recent Version.

1.5 General Precautions

1.5.1 Safety

Proper safety precautions must be observed when conducting reaeration studies. Refer to the SESD Safety, Health and Environmental Management Program Procedures and Policy Manual and any pertinent site-specific Health and Safety Plans (HASP) for guidelines on safety precautions. These guidelines, however, should only be used to complement the judgment of an experienced professional. For example, these methods may be employed during periods of high stream flow or in conjunction with boating operations.

1.5.2 Procedural Precautions

The following precautions should be considered when conducting reaeration measurements studies:

- All instrumentation should be in good condition and operating within the manufacturer's recommended tolerances.
- All instrumentation should be calibrated and deployed in accordance with the manufacturer's requirements.

2 Special Sampling Considerations

2.1 Quality Control

The reaeration rate coefficient is expressed as a rate in 1/day corrected to 20° Celsius (C). Dissolved oxygen (DO) meters should be calibrated according to SESD Operating Procedure for Field Measurement of Dissolved Oxygen (SESDPROC-106).

2.2 Records

Information generated or obtained by SESD field investigators will be organized and accounted for in accordance with SESD records management procedures. Field notes, recorded in a bound field logbook, in accordance with SESD Operating Procedure for Logbooks (SESDPROC-010), will be generated, as well as chain-of-custody documentation. All measurements shall be thoroughly documented in field records. All measurements shall be traceable to the personnel making the measurements and the equipment utilized.

3 General Considerations

3.1 General

The techniques and equipment described in Section 4 of this procedure document are designed to provide representative measurements of reaeration rates. Care should be applied in the selection of measurement sites and/or reaches to ensure personnel and equipment safety.

Highly productive waters may cause ambient DO to rise significantly during the day. If it is known ahead of time that this may be the case, diffusion dome measurements may be conducted at night or in the late evening/early morning, as safety considerations allow, to minimize ambient DO changes associated with algal production.

3.2 Equipment Selection Considerations

Diffusion domes currently in use are constructed of stainless steel fitted with a ring of foam insulation for floatation. The domes have two inlet/outlet ports for purging of the dome volume and each dome is equipped with a spinning baffle on a post running through the dome to allow external manual operation. In addition, each dome has an internal bracket to hold a DO probe and external brackets for securing cooling water tubing which is supplied by a submersible pump.

Ecological Assessment Branch (EAB) diffusion domes are custom designed to hold a DO probe without a stirrer. Currently, EAB uses a luminescent (LDO) probe connected to a digital display but other technologies are available and can be utilized. A digital display is preferable in low turbulence systems to better define small changes in DO through the measurement period.

The dome number and DO meter serial number or other identifier should be recorded in the field log book. For DO meters, the log book should include a notation indicating which meter was used in the dome and which provided ambient data.

If measurements are made in a saline environment, a salinometer or other instrumentation should be deployed to allow for correction of dissolved oxygen measurements.

4 Diffusion Dome Reaeration Measurement

The determination of the reaeration rate coefficient is a measure of the rate at which atmospheric oxygen can move across the air water interface. This is a measure of the rate of potential oxygen transfer. The actual quantity of oxygen transferred to the water column is a function of the water column dissolved oxygen deficit and the reaeration rate coefficient.

The diffusion dome technique for measuring reaeration is based on the work of Copeland and Duffer (1963) and Juliano (1969). In general, the method involves purging the volume of a floating dome with nitrogen gas and monitoring the recovery of oxygen within the dome. While applicable to most systems, this method is especially useful in areas where the gas tracer method (SESDPROC-506) may not feasible (e.g., very shallow streams or large water bodies) or too resource intensive. Where site conditions allow, the dome should be free-floating in the water body. Usually, the dome is tethered to an unanchored boat to allow the field crew access to the dome for purging and mixing during the float. If necessary, based on the site, a "static" float may be conducted, where the dome is tethered to a stationary object.

4.1 Field Measurement Method

The diffusion dome method requires two DO meters. One meter is installed inside the dome to measure DO and temperature in the dome air space. The second meter is used to measure ambient water column DO and temperature. The ambient DO probe should be located just below the water surface, deep enough to represent the entire water column for a non-stratified system. If DO probes are equipped with attachable stirrers, the stirrer should be installed on the ambient probe, but not on the dome probe. The dome is equipped with a manual baffle for circulating air inside the dome.

Once the DO probe is installed in the dome, the dome is placed on the water surface and the nitrogen gas line is connected to an inlet valve on the dome. The circulation pump should then be place in the water and started and DO/temperature monitoring initiated. The ambient probe is deployed in the water column and ambient DO, temperature and salinity (if appropriate) are also monitored. The temperature inside the dome should stabilize relatively close to the ambient temperature before purging is conducted. Based on the ambient data, the DO deficit is calculated and the dome is purged with nitrogen to create a DO deficit between the water column and the dome atmosphere which approximates the water column DO deficit. If a method for calculating the deficit has not been predetermined, the following example method may be used.

Example Deficit Calculation:

Ambient DO - DO Saturation Concentration = Deficit

Where:

$$\text{DO Saturation Concentration} = 0.0035T^2 - 0.3369T + 14.407$$

Then:

Ambient DO – Deficit = Dome Purged DO Value

Ambient and dome monitoring data should be recorded at 15 minute or more frequent intervals throughout the measurement period. If possible, monitoring should continue for a period sufficient to recover at least 5% of the initial DO deficit imposed in the dome. Depending on the magnitude of the deficit and environmental conditions affecting the measurement (e.g., debris blocking channel, rapids affecting dome seal), a 5% recovery may not be possible. In such cases, the measurement should continue for a minimum of 30 minutes or until conditions prevent continued monitoring. Locational data (latitude/longitude) and depth should also be recorded concurrent with monitoring data readings, in accordance with SESD Operating Procedure for Global Positioning System (SESDPROC-110).

Wind data should always be collected during diffusion dome measurements on open water bodies (e.g., lakes, estuaries) and may be desirable on river or stream systems. Wind speed from a hand-held wind meter should be recorded, concurrent with monitoring data readings, with an approximation of wind direction. Alternatively, a weather station or stationary logging wind meter deployed in the study area can provide wind data. If the hand held meter or weather station is so equipped, barometric pressure should also be recorded.

The circulation pump helps maintain a constant temperature in the dome and should be checked frequently throughout the diffusion measurement period. If temperatures rise significantly even with proper operation of the circulation pump, the dome should be shaded and/or a small amount of ice placed on top of the dome.

4.2 Reaeration Rate Coefficient Calculation

Following field data collection, a reaeration rate coefficient is calculated for each diffusion measurement period and corrected to a base temperature of 20°C as follows:

The amount of oxygen diffused into the dome, D, during the test is represented by:

$$\textbf{(1) } D\ (g/m^3/hr) = \frac{(V)(32\ g/mole)(0.0446\ moles/liter)}{(CA)(t)(Z)}$$

where V = change in volume of O_2 in chamber (liters),

CA = diffusion Dome area at water-surface interface (meters $[m]^2$)

t = period of measurement (hours)

Z = average depth of unstratified water column (m)

The change in O_2 chamber volume, V, is calculated as follows:

$$\textbf{(2) } V\ (liters) = \{(273.15V_1)/(273.15 + T_1) - (273.15V_0)/(273.15 + T_0)\}(CV)(f)$$

where V_1 = final dome DO as percent saturation (as fraction)

V_0 = initial dome DO as percent saturation (as fraction)

T_1 = final temperature in dome (°C)

T_0 = initial temperature in dome (°C)

CV = dome (chamber) volume (liters)

f = % O_2 in ambient atmosphere (atm) (as fraction)

$$f = \frac{0.2095(P - P_{wv})}{P}$$

where P = barometric pressure (atm)

P_{wv} = water vapor partial pressure (atm)

When barometric pressure is not available, local pressure, P, can be estimated from altitude and air temperature as:

$$P = \{(273.15T - 0.0065Z)/(273.15T)\}^{5.2559}$$

where T = ambient temperature (°C)

Z = local elevation (m)

When water vapor partial pressure is not available, $P_{wv,}$ can be estimated by the Arden Buck Equation:

$$P_{wv} (hPa) = 6.1121 exp\{(17.502T)/(240.97+T)\}$$

where T = ambient temperature (ºC)
1 (hPa) = 9.8692 e -4 (atm)

The reaeration rate, K_a (Base e), is then calculated as:

(3) $$\mathbf{K_a\ (1/day) = \frac{(D)(24\ hrs/day)}{(S_{def})(C_s)}}$$

where D = oxygen diffusion from equation 1 ($g/m^3/hr$)

S_{def} = average saturation deficit between dome and water column (as fraction)

= {1 – (average dome DO/average water column DO)} C_s = average water column saturation DO (g/m^3)

The reaeration rate, K_a (Base e, @ 20 ºC), is then calculated as:

(4) $$\mathbf{K\ (1/day) = (1.024)^\wedge(20 - T_a)}$$

where T_a = average ambient temperature (Celsius)

Appendix D

Bartlesville WLA Studies – Caney River Monitoring and Modeling Report

November 2018

Tetra Tech

Scott Thompson
Executive Director

OKLAHOMA DEPARTMENT OF ENVIRONMENTAL QUALITY

Mary Fallin
Governor

OKLAHOMA DEPARTMENT OF ENVIRONMENTAL QUALITY

November 13, 2018

Terry Lauritsen, P.E.
Water Utilities Director
401 South Johnstone Ave.
Bartlesville, OK 74003

Re: Caney River WLA- TMDL Study
Proposed New Secondary Discharge to Caney River
City of Bartlesville Wastewater Treatment Facility
Facility ID: S-21402, OPDES OK0030333

Dear Mr. Lauritsen:

DEQ received your request on June 22, 2017, for the State Water Quality Management Plan modification with new design flow (8.206 MGD) and additional location of point of discharge (POD). Based on modified WLA report dated on November 9, 2013, Tetra Tech has proposed the following waste load allocation (WLA).

The proposed discharge limits for City of Bartlesville Wastewater Treatment Facility (WWTF) are shown below:

City of Bartlesville WWTF

At current POD (Latitude: 36° 45' 25.965" N, Longitude: 95° 57' 54.406" W) with design flow of 8.206 MGD,

Year-round: 10.0 mg/l BOD_5; 1.0 mg/l NH_3-N; 6.0 mg/L DO; 15.0 mg/L TSS

At 50-50 split discharge at current POD and 7 miles upstream of intake (Latitude: 36° 48' 1.27" N, Longitude: 95° 58' 20.72" W),

Year-round: 10.0 mg/l BOD_5; 1.0 mg/l NH_3-N; 6.0 mg/L DO; 15.0 mg/L TSS
Exception for spring at 7 miles upstream of intake: no discharge from 4/1 to 6/15.

If you wish to proceed with these limits, please advise us and we will transmit this information to EPA for a technical review. Upon completion of their review, the limits will be submitted for public comment.

After all public comments are addressed and the request is approved by EPA, your discharge limits will be placed in the State Water Quality Management Plan (the 208 Plan). If you have any questions, please contact Soojung Lim at (405) 702-8195 or soojung.lim@deq.ok.gov

Sincerely,

Joe A. Long, Manager
Watershed Planning Section
Water Quality Division

cc: Elizabeth Denning, ECLS Engineer, ODEQ
Greg Carr, P.E., WQD Chief Engineer, ODEQ
Patrick Rosch, P.E., WQ Planning Manager, ODEQ
Srini Sundaramoorthy, P.E., Tetra Tech

Bartlesville WLA Studies – Caney River Monitoring and Modeling Report

November 9, 2018

city of bartlesville
CONNECTED / CREATIVE / VIBRANT

PREPARED FOR

City of Bartlesville
401 S. Johnstone Ave.
Bartlesville, OK 74003

PREPARED BY

Tetra Tech
7645 E 63rd Street, Suite 301
Tulsa, OK 74133
OK CA 2388, Exp. 6/30/19
Tel 918-249-3909

TETRA TECH

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EXECUTIVE SUMMARY

The City of Bartlesville in northeastern Oklahoma currently supports a population of 36,647 with a permitted wastewater treatment discharge capacity of 7 MGD. With a projected population increase of approximately 13 percent by the year 2050, water usage and treatment demands are anticipated to rise significantly. A Facility Plan Amendment Study (Tetra Tech, 2017) projected an effluent discharge of 8.206 MGD will be required to provide service to the growing population of Bartlesville. The City is seeking an expansion of their City of Bartlesville Wastewater Treatment Facility (WWTF) and approval from the Oklahoma Department of Environmental Quality (ODEQ) to increase their discharge to the Caney River to meet this need.

The WWTF currently discharges to the Caney River downstream of a run-of-river low-head dam. The State of Oklahoma places the Caney River under the Fish and Wildlife Propagation (FWP) classification of Warm Water Aquatic Community (WWAC) (ODEQ, 2014). ODEQ is requiring that a wasteload allocation (WLA) study in the context of a Total Maximum Daily Load (TMDL) under the Federal Clean Water Act be conducted to evaluate Caney River assimilative capacity and demonstrate that the Caney River can receive the expanded wasteflow at a particular WLA and still support designated uses and water quality standards. The City is exploring options to allocate the expanded WWTF discharge between the existing location and a point approximately seven miles upstream of an existing water supply intake location to increase potential water supply volume in future years, providing that the Caney River has the assimilative capacity to handle this new inflow. A monitoring and modeling study was conducted by Tetra Tech under a plan approved by ODEQ in support of developing a TMDL and WLA for the City in this portion of the Caney River.

To evaluate the assimilative capacity of the Caney River to receive additional treated effluent at the current discharge point, Tetra Tech previously developed a simulation of the existing conditions along the river below the low-head dam using a one-dimensional steady state QUAL2E (Brown and Barnwell, 1987) receiving water model (Tetra Tech, 2011). A more recent desktop analysis was conducted by Tetra Tech to explore expansion of discharge upstream by converting the existing QUAL2E model to a more user-friendly and modernized version known as QUAL2K (Chapra et al., 2012) and adding the seven-mile segment upstream of the low-head dam (Tetra Tech, 2016). The desktop analysis suggested that assimilative capacity may be available along the Caney River if key assumptions regarding existing conditions and reaction rates instream could be validated. Following ODEQ approval of the monitoring plan developed by Tetra Tech, the City of Bartlesville approved moving forward with the comprehensive monitoring program with intensive monitoring studies performed in September and October 2017 to support calibration and corroboration of the QUAL2K model.

The September and October 2017 intensive monitoring studies were conducted to measure and observe existing conditions under two different flow regimes. In coordination with the U.S. Army Corps of Engineers (USACE) which manages upstream reservoirs in the Caney River basin, flows were controlled for the two sampling trips and held at approximately the historical critical 7Q2 flow (20 cfs) in September and historical median flow (100 cfs) for the October study. Monitoring included surveys of channel cross sections, flow and time-of-travel measurements, water quality grab sampling at key points of interest, and synoptic water quality sampling of parameters including dissolved oxygen (DO), water temperature, pH, and conductivity using data sondes. Diel variations were observed in both water temperature and DO to inform the extent of algal kinetics influencing the Caney River. *In situ* measurements of both reaeration and sediment oxygen demand (SOD) were collected to reduce uncertainty in those parameters shown to be key components of the DO balance during the preliminary desktop analysis.

The QUAL2K model was calibrated and corroborated based on the field monitoring from 2017. Both calibration and corroboration simulations provided strong agreement with observed hydraulics, thermal conditions, nutrients, pH, and DO. The Caney River QUAL2K model appropriately captures observed conditions under two different flow regimes, so the ability of the model to capture application scenarios is very good and does not include significant uncertainty. Based on the tested parameters, the model simulation of DO concentration is most sensitive to SOD and boundary conditions.

The calibrated and corroborated QUAL2K model provides a reasonably accurate representation of water quality in the Caney River, particularly during low flow conditions, and can thus support WLA scenarios to assess the capacity of river to assimilative additional wasteload discharge. Critical conditions of low flow and warm temperatures were simulated as part of the WLA analysis for each season: summer, spring, and winter. This extreme seasonal baseline conditions meets the minimum daily mean DO water quality criteria of 5.0 mg/l for summer and winter, and 6.0 mg/l for spring. WLA scenario results suggest that for summer and winter, the Caney River model extent likely has the assimilative capacity to support additional effluent discharge either upstream of the dam at a point seven miles above the existing intake location, and/or expanded discharge at the existing outfall. WLA scenarios in the spring season suggest that assimilative capacity for the expanded flow at the existing outfall is possible, although it may not be possible to incorporate the upstream discharge location during that season. Results suggest that instream DO concentrations are likely to meet existing water quality standards by season with at least a 5% margin of safety with the exception of the upstream discharge during the spring season.

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1.0 INTRODUCTION

The City of Bartlesville is located in northeastern Oklahoma, bisected by the Caney River which flows south, joining the Verdigris River northeast of Tulsa. Bartlesville wastewater is processed through the City of Bartlesville (Plant #1) Wastewater Treatment Facility (Permit OK0030333) north of the City, which discharges into the Caney River downstream of a low-head dam. A Facility Plan Amendment Study (Tetra Tech, 2017) projected population growth for Bartlesville to be 41,441 persons by the year 2050, which will increase water usage and treatment demands from current plant capacity of 7.0 MGD to a projected 8.206 MGD. Given the anticipated growth and increased wastewater discharge demands for Bartlesville, the City seeks to expand effluent treatment at the Bartlesville Wastewater Treatment Facility (WWTF) and potentially add a second discharge location upstream of the existing outfall. The City is exploring options to allocate the second discharge point approximately seven miles upstream of the existing water supply intake location which is upstream of run-of-river low-head dam, providing that the Caney River has the assimilative capacity to handle this new inflow (Figure 1-1). The purpose of this report is to document the monitoring and modeling study conducted during the summer and fall of 2017 in support of TMDL development and wasteload allocation (WLA) assessment for the City along this portion of the Caney River.

From a regulatory perspective, the Oklahoma Department of Environmental Quality (ODEQ) emphasizes modeling of dissolved oxygen (DO) kinetics to evaluate assimilative capacity under low flow and high temperature conditions. The segment of the Caney River of interest for this study includes a portion of waterbody 121400020010 (Caney River from Hulah Reservoir to Rice Creek) and waterbody 12140010010 (Caney River from Rice Creek to Verdigris River), although specifically the extent is Caney River from the W 1500 Road crossing down to the Highway 75 crossing.

This extent is currently impaired for biology based on the results of fish bioassessments in the context of the existing Fish and Wildlife Propagation (FWP) classification of Warm Water Aquatic Community (WWAC) (ODEQ, 2014). To address the assimilative capacity of the Caney River to receive additional treated effluent, Tetra Tech previously developed a simulation of the existing conditions along the river below the low-head dam using a one-dimensional steady state QUAL2E (Brown and Barnwell, 1987) receiving water model (Tetra Tech, 2011). A more recent desktop analysis was conducted by Tetra Tech to explore the expansion of discharge upstream by converting the existing QUAL2E model to a more user-friendly QUAL2K model (Chapra et al., 2012) and adding the seven-mile segment upstream of the low-head dam (Tetra Tech, 2016). The desktop analysis suggested that assimilative capacity may be favorable along Caney River if key assumptions regarding existing conditions and reaction rates instream could be validated. This report covers a more robust, calibrated, and corroborated QUAL2K model which includes the seven-mile segment of the Caney River upstream of the low-head dam and extends south of the City to Highway 75. The report details the QUAL2K model setup and how the monitoring data were used to parameterize model calibration and corroboration runs.

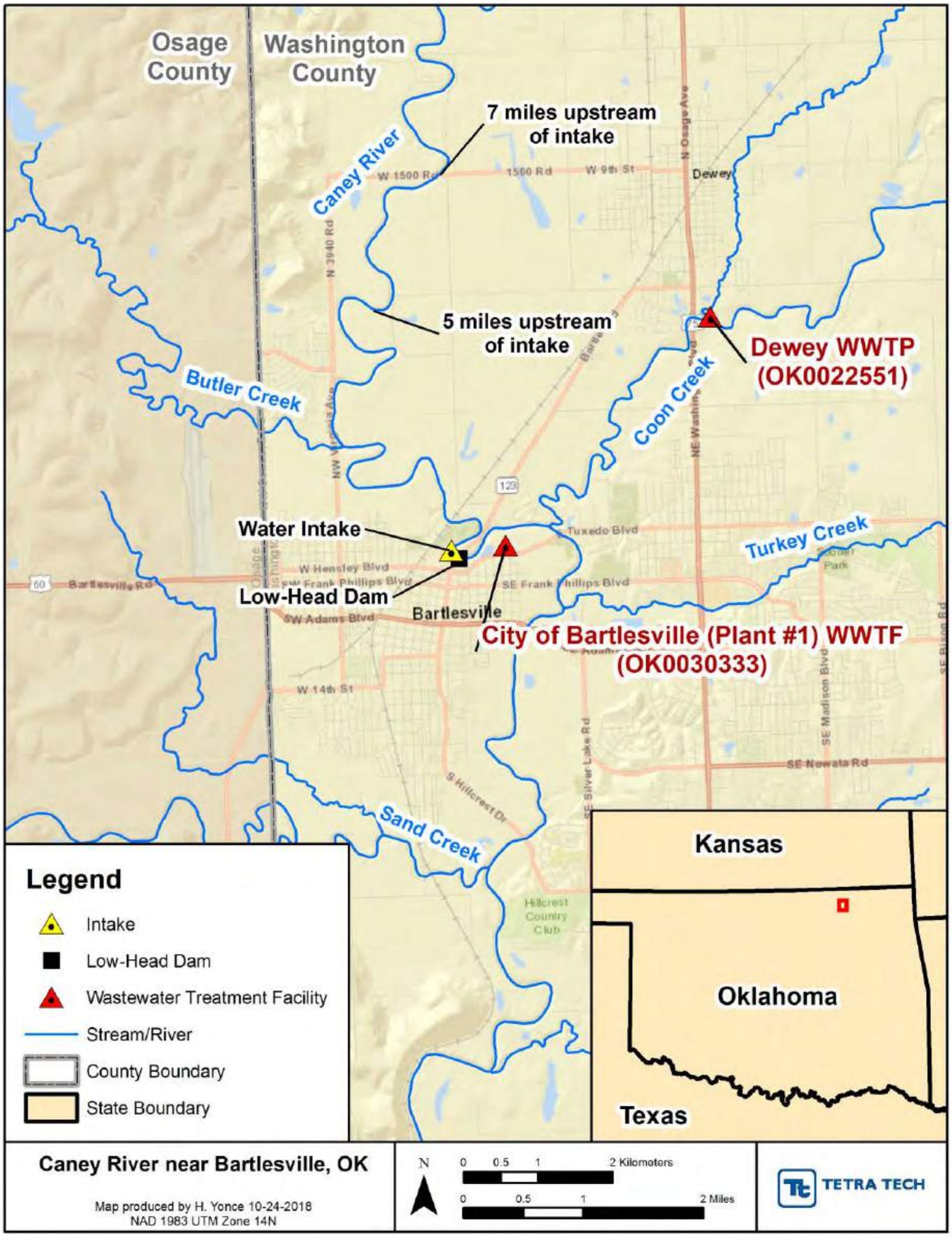

Figure 1-1. Caney River near Bartlesville, Oklahoma

2.0 FIELD WORK AND MODEL DEVELOPMENT PLAN SUMMARY

The QUAL2K model is a one-dimensional steady-state river water quality model (Chapra et al., 2012). QUAL2K was developed as a modernized and updated version of QUAL2E, the platform used for the previous Caney River modeling work (Tetra Tech, 2011). QUAL2K assumes well-mixed stream channels (both vertically and laterally), and employs a diel, or 24-hour period, heat budget. The desktop QUAL2K model was developed for a 7.9-mile portion of the Caney River and simulated basic instream conditions in the absence of observed field data (Tetra Tech, 2016). While the desktop model provided preliminary results for stream hydraulics, water temperature, and water quality kinetics, field data is required to validate the results of such a model. The refined QUAL2K model was extended to a total of 21.5 miles (34.7 kilometers) of the Caney River, and incorporates observed data for a more robust, calibrated simulation.

To obtain the necessary data to refine and expand the desktop QUAL2K model to support TMDL and WLA estimation, two different flow conditions were monitored during September and October of 2017. Conducting field monitoring under two different flow conditions (critical low flow and median flow) allows for the model to be more robust in its ability to capture receiving stream system variability. Critical low flows are defined by ODEQ to be the annual 7-day minimum flow with a recurrence interval of 2 years (7Q2) and are used in TMDL and WLA analyses to simulate the most critical period during which water quality standards must be maintained. Additional sampling was conducted under median flow conditions during the early fall season to address hydrological and seasonal variability (for model corroboration purposes) and to consider time-of-travel implications above the existing water supply intake. The USGS flow gage on the Caney River upstream of Coon Creek at Bartlesville (ID# 07174400) was used to develop summer 7Q2 and annual median flow statistics which were tabulated as 20.2 cfs and 98.3 cfs respectively based on calendar years 1986 through 2015 (complete years of data available prior to sampling).

Flow to the Caney River near Bartlesville is dominated by two upstream reservoirs (Hulah Lake and Copan Lake) which are managed by the U.S. Army Corps of Engineers (USACE). With cooperation from the USACE, reservoir discharges were controlled during sampling periods to maintain approximately 7Q2 and median flow conditions for the two separate sampling trips respectively. The first sampling trip was conducted September 6 – 11, 2017 and the second field sampling trip was conducted October 2 – 6, 2017. The following field monitoring efforts were undertaken to meet the objections of refining, parameterizing, and calibrating the QUAL2K model:

- Obtained physical measurements to refine QUAL2K model input assumptions that represent Caney River channel under different baseflow regimes, including cross-section, width, and depth measurements.
- Obtained flow and time-of-travel measurements to develop and calibrate QUAL2K hydraulics components and provide a basis for predicting stream reach velocities in the Caney River under two different baseflow regimes.
- Sampled instream water quality under the two different flow conditions along the entire area-of-interest to support model development:

- Collected *in situ* measurements of reaeration and sediment oxygen demand to reduce uncertainty in those parameters shown to be key to the DO balance during the preliminary desktop analysis.
- Obtained general field measurements for basic water quality indicators such as temperature, DO, conductivity, and pH at multiple locations throughout the study area to characterize stream reaches and tributary conditions.
- Performed detailed “DO sag” study to determine the response of instream DO concentrations downstream of the existing WWTF discharge.
- Collected water chemistry field samples for lab analysis to characterize parameters associated with assimilative capacity and initial modeling conditions at key locations.
- Collected field measurements instream over time and water chemistry samples for lab analysis to determine diel variation in DO concentrations and the extent that algal kinetics influence Caney River.

Field monitoring results are summarized throughout this report as they pertain to each element of QUAL2K model development and/or calibration and corroboration efforts. The complete Study Plan is provided in Attachment A.

3.0 QUAL2K MODEL SETUP

3.1 MODEL PERIOD AND EXTENT

The Caney River QUAL2K model was set up, parameterized, and calibrated based on data collected during the critical low flow period observed during the first field trip conducted September 6 – 9, 2017. The simulation date for the calibration model was selected as the central date of the longitudinal surveying data (September 7 – 9), or September 8, 2017. The corroboration model run was set up using data collected largely October 2 – 6, 2017 to test that the calibrated model parameterization holds true under different flow and meteorological conditions. The simulation date for the corroboration model run was selected as the central date of the longitudinal surveying data (October 2 – 4), or October 3, 2017. The models were run for these simulation dates and a run time of 30 days to allow the model to reach steady state conditions. The flow conditions for these simulations were set by controlled releases of the upstream reservoirs in coordination with USACE to be approximately equal to 7Q2 and median flow conditions at the USGS gage location. Flows used for model development were based on the average flows observed on the simulation date chosen and the day prior. Calculated 7Q2 and median flows were 20.2 and 98.3 cfs respectively, and average observed flows for the calibration and corroboration models were 24.0 cfs and 96.7 cfs respectively.

The simulation extent along the Caney River was from the W1500 Road crossing down to the crossing of Highway 75 south of Bartlesville (Figure 3-1). The QUAL2K model upstream extent was chosen because it is 7 miles upstream of the intake, which is the most upstream location being considered for the future expanded outfall. The downstream extent was chosen due to road accessibility from Highway 75, and the distance being sufficiently far away from the existing outfall to fully capture the sag point downstream of both outfall locations of interest. The Highway 75 crossing is also the location of USGS gage 07174700 (Caney River near Ochelata, OK) which is no longer active.

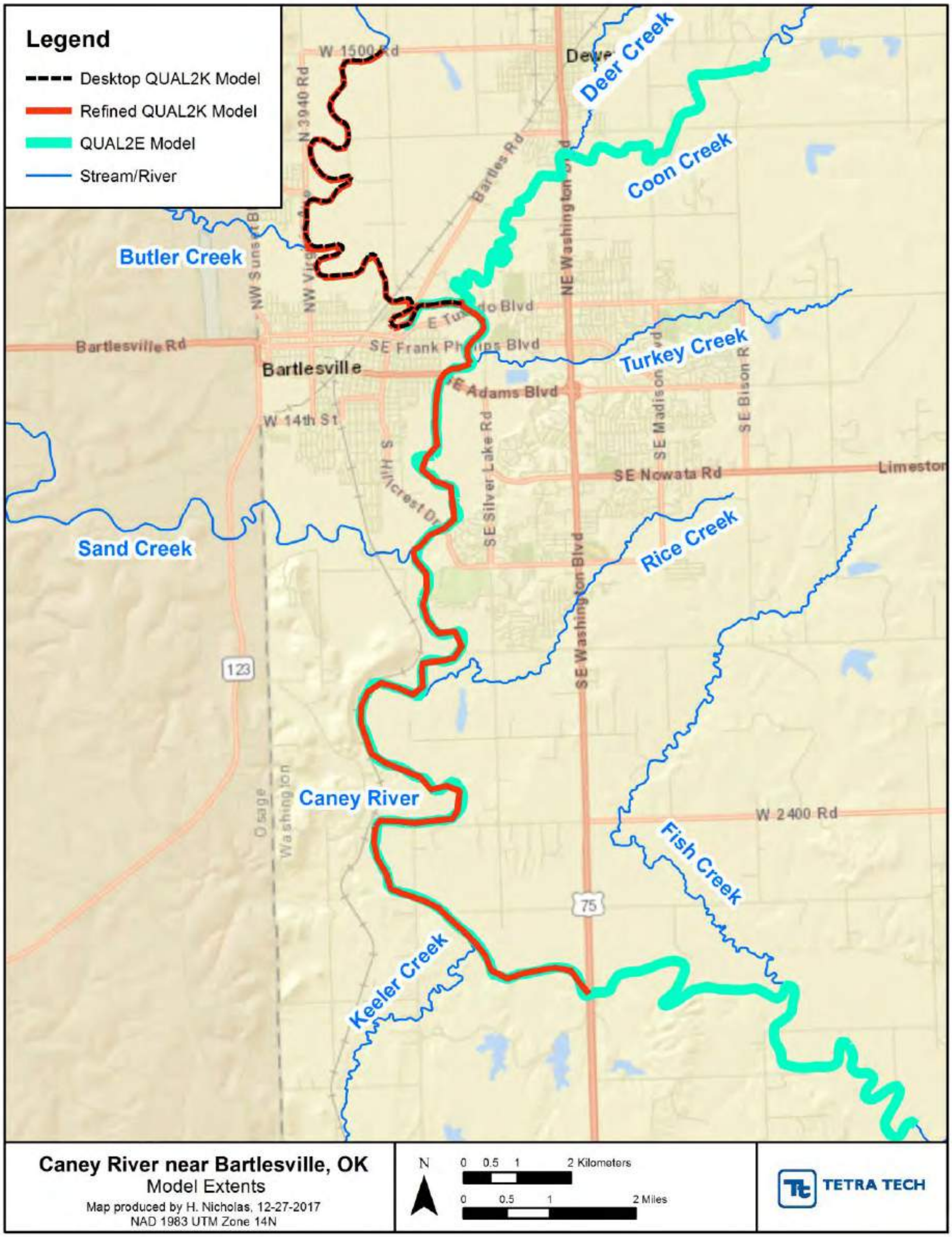

Figure 3-1. Caney River QUAL2K model extent

3.2 REACH SEGMENTATION

QUAL2K models are subdivided into segments or “reaches” which have consistent hydraulic characteristics. The refined 21.5 mile (34.7 kilometer) QUAL2K model extent was subdivided into twelve reaches based on locations of key physical features and boundary conditions including: key points of interest (e.g., 5 and 7 miles upstream of the intake), key boundary conditions (e.g., inflows of tributaries throughout the system), and areas with uniquely different hydraulic properties identified during the field investigation (e.g., the deepest area near Johnstone Park and the low-head dam) (Table 3-1 and Figure 3-2). Field observations and cross-sectional data obtained from the monitoring revealed that conditions upstream of the dam are significantly different than downstream of the dam.

The low-head dam located beneath Oklahoma Highway 123 (Cherokee Street Bridge) impacts the physical hydraulics of the Caney River by producing an upstream condition which is somewhat deeper and slow-moving. Water column profiling was conducted during the September monitoring period at seven locations (sites A through G in Figure 3-3) to improve understanding of the channel configuration upstream of the dam. Results demonstrated that maximum depth increases gradually over this segment from an average of approximately 10 feet deep to roughly 20 feet deep approaching the dam (with the red line section in Figure 3-3 showing the 20-feet deep extent), before decreasing again immediately before the dam likely due to historical sediment deposition. Downstream of the low-head dam, the Caney River is characterized by riffle-run-pool sequences that are seemingly more aerated with higher velocities than upstream of the dam. Field notes were made regarding details on riffle-run segment lengths, locations of frequent deep pools, and information on sediment properties (i.e., sand, gravel, and cobble) to further support model reach delineation.

Reaches vary in length from the shortest immediately upstream of the dam at 0.06 miles long to 5.75 miles long for Reach 11 in the lower portion of the model extent. Reach locations and lengths were determined using NHDPlusV2 flowlines, while model inputs for reach elevations were estimated using a 9.8-foot (3-meter) resolution digital elevation model (DEM). DEM-derived elevations reflect water surface rather than channel bed; however, these elevations are likely representative of changes in elevation between reaches. Channel slopes were calculated as the change in elevation divided by the length of the reach (Table 3-1). There is no channel slope associated with the dam segment as it is simulated as a weir. Reaeration instream is typically caused by a combination of channel slope, wind, streamflow, width, depth, and velocity.

Table 3-1. Caney River QUAL2K model segmentation

Reach	Description (Caney River)	Reach Length (mi)	Upstream Elevation (ft)	Downstream Elevation (ft)	Channel Slope (ft/mi)
1	7 miles to 5 miles US of intake	1.90	655.84	655.51	0.17
2	5 miles US of intake to Butler Creek	2.16	655.51	655.18	0.15
3	Butler Creek to deepest area behind dam	2.47	655.18	651.90	1.33
4	Deepest area behind low-head dam*	0.40	651.90	651.57	0.81
5	Dam segment	0.06	651.57	645.34	N/A
6	Dam to Bartlesville WWTF outfall	0.50	645.34	645.01	0.65
7	Bartlesville WWTF outfall to Coon Creek	0.44	645.01	644.69	0.74
8	Coon Creek to Turkey Creek	0.90	644.69	636.48	9.10
9	Turkey Creek to Sand Creek	2.92	636.48	636.15	0.11
10	Sand Creek to Rice Creek	2.32	636.15	623.81	5.33
11	Rice Creek to Keeler Creek	5.75	623.81	621.62	0.38
12	Keeler Creek to Highway 75	1.73	621.62	619.39	1.29

*Note that the end of Reach 4 is the location of the USGS flow gage and the City of Bartlesville water intake system.

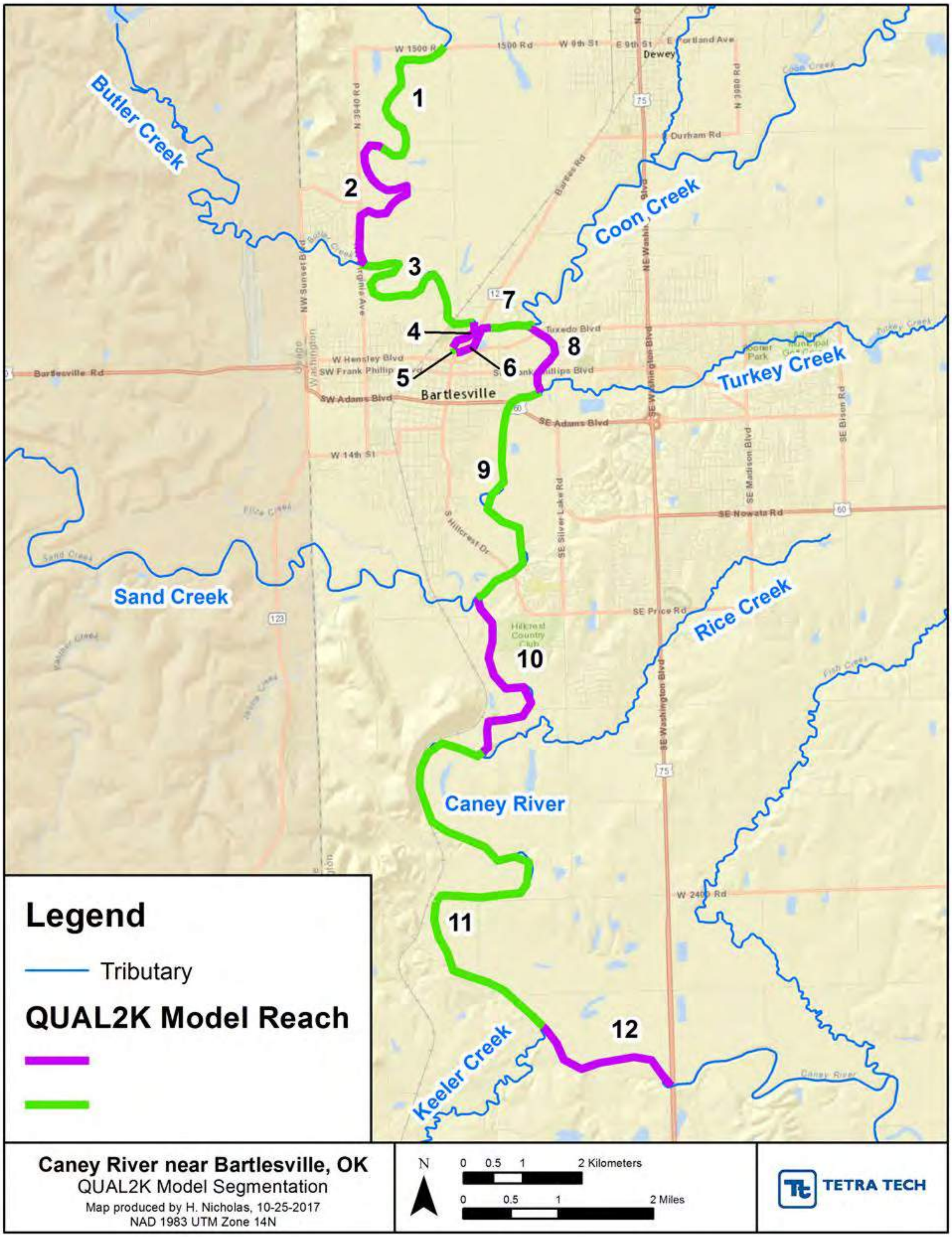

Figure 3-2. Caney River refined QUAL2K model segmentation

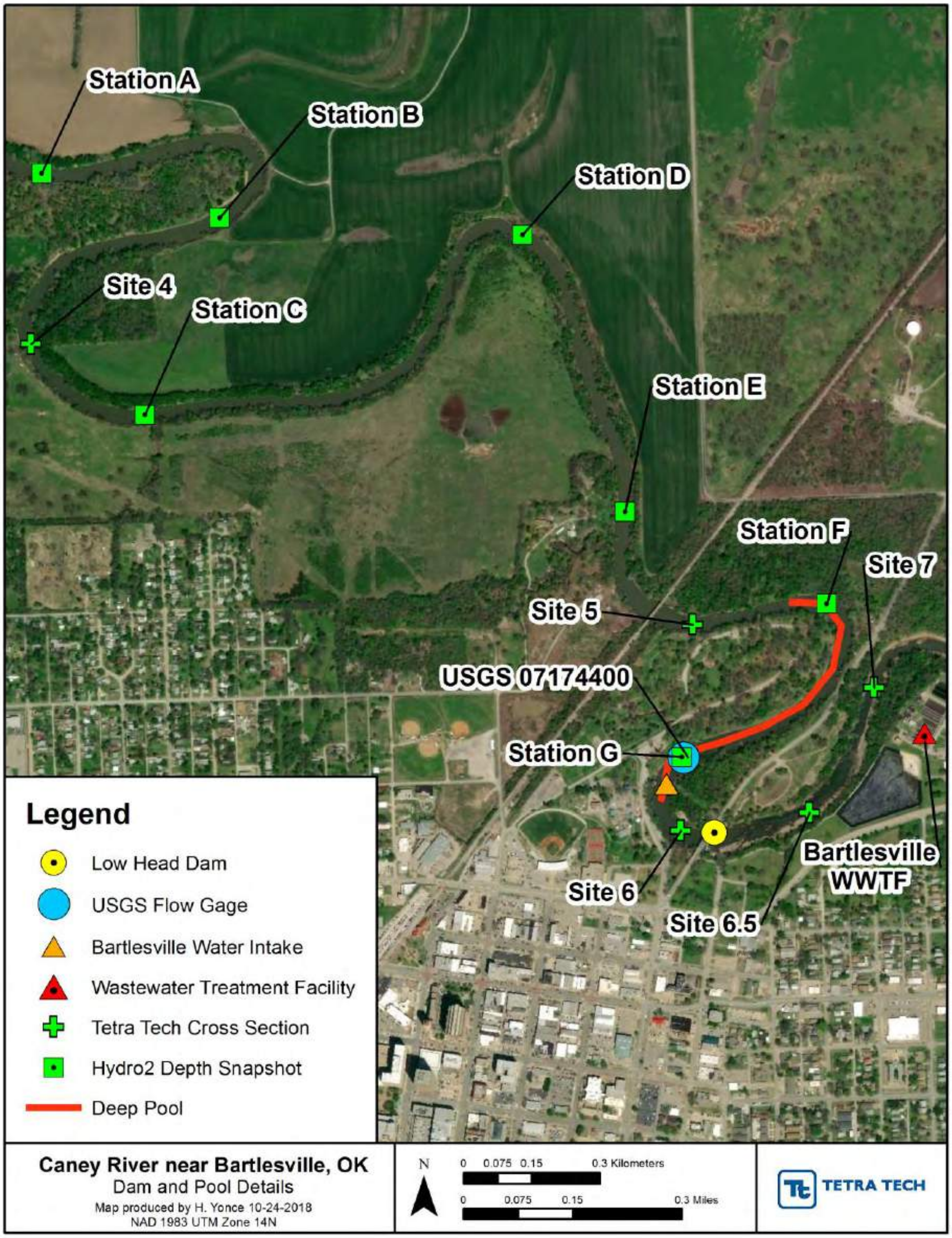

Figure 3-3. Detailed view of pool area and cross-section and depth locations

3.3 METEOROLOGICAL FORCING AND STREAM SHADING

Meteorological inputs for the QUAL2K model are based on observed weather data from the Bartlesville Municipal Airport station KBVO (www.wunderground.com). For the calibration and corroboration dates (9/8/2017 and 10/3/2017), model inputs for hourly air and dew point temperature are based on hourly observations from KBVO. Average daily wind speed was observed during September field sampling to be approximately 2 miles per hour at a height of 4 feet over the river, and inputs for the corroboration model are based on average daily wind speed observed at KBVO to be 7 miles per hour at a height of 32.8 feet over the land surface. These average daily wind speeds are applied to the model for all hours of the day and were altered during calibration to capture observed reaeration rates. These wind speeds were converted to a height of 23.0 feet (7 meters) for model input using the wind profile power law, which converts the observed wind speed (u_o) at the observed height (z_o) to the wind speed (u) at the model input height (z) in units of meters (Touma, 1977):

$$\frac{u}{u_o} = \left(\frac{z}{z_o}\right)^{(1/7)}$$

Inputs for wind speed at all hours of the day for the calibration and corroboration models are 3.53 mph (1.58 m/s) and 6.64 mph (2.97 m/s) respectively. Average daily cloud cover is estimated from KBVO as 0 percent for the calibration date (based on descriptive sky condition data as “clear” all day), and 100 percent for the corroboration date (based on sky conditions described all day as “overcast”). Model inputs for stream shade are approximated as zero percent for both simulation dates because stream-level photography and aerial imagery suggest limited ability of riparian vegetation and topography to provide impactful shade. Limited shade provided by the incised streambanks does not have a large impact on the heat balance of the river throughout the day due to the large channel width. A sensitivity test was conducted for which the final calibrated model was simulated with stream shading at 25 percent for all hours of the day (rather than the assumption of zero percent) and the water temperatures fell far below the observed range from the simulation period supporting the zero assumption.

Table 3-2. Caney River QUAL2K meteorological inputs for calibration and corroboration models

Hour	Air Temperature (°C)		Dew Point Temperature (°C)	
	Calibration	Corroboration	Calibration	Corroboration
1	16.72	21.11	11.72	17.22
2	14.39	21.11	11.72	17.78
3	12.78	22.22	11.11	18.28
4	12.78	22.78	11.11	18.89
5	12.22	22.22	10.61	18.89
6	11.11	21.11	10.00	18.89
7	10.61	21.11	10.00	19.39
8	11.72	22.22	10.61	20.00
9	16.72	23.28	13.28	20.61
10	20.61	23.89	14.39	20.61
11	26.11	24.39	12.78	20.61
12	27.78	24.39	12.78	21.11

Hour	Air Temperature (°C)		Dew Point Temperature (°C)	
	Calibration	Corroboration	Calibration	Corroboration
13	28.89	26.11	11.72	21.11
14	30.00	23.89	12.22	21.11
15	28.28	24.44	11.72	21.67
16	29.39	24.44	10.61	21.67
17	28.89	25.00	10.61	22.22
18	28.28	24.39	11.11	22.22
19	26.72	23.89	11.72	22.22
20	20.61	23.28	13.28	22.22
21	18.28	23.28	13.28	21.72
22	16.11	23.28	12.78	21.72
23	15.00	23.28	13.28	21.72
24	14.39	23.28	12.78	21.72

3.4 REACH HYDRAULICS

Reach hydraulics were developed for the Caney River QUAL2K model using depth, width, flow, and time-of-travel (TOT) dye study data collected during the two intensive field monitoring trips with some supplemental cross-sectional measurements during other periods. Average width, average depth, and maximum depth were measured at specific cross-section locations using survey equipment on 8/23/17, 9/6/17, 9/11/17, 10/5/17, and 10/6/17 at a number of different locations (Figure 3-4 and Table 3-3). Although cross-sections were observed on several different dates and flow conditions, variable flow conditions did not result in noticeable changes in channel width, so the suite of field data were all used to parameterize reach hydraulics. TOT measurements were collected along the Caney River using rhodamine dye deployed at several locations with overlapping extents. By observing the TOT of the dye at specific sonde sites throughout the system, velocities were calculated on the order of 0.10 – 0.26 ft/s during the September low-flow sampling period, and 0.13 – 0.39 ft/s during the October median flow sampling period (Table 3-4).

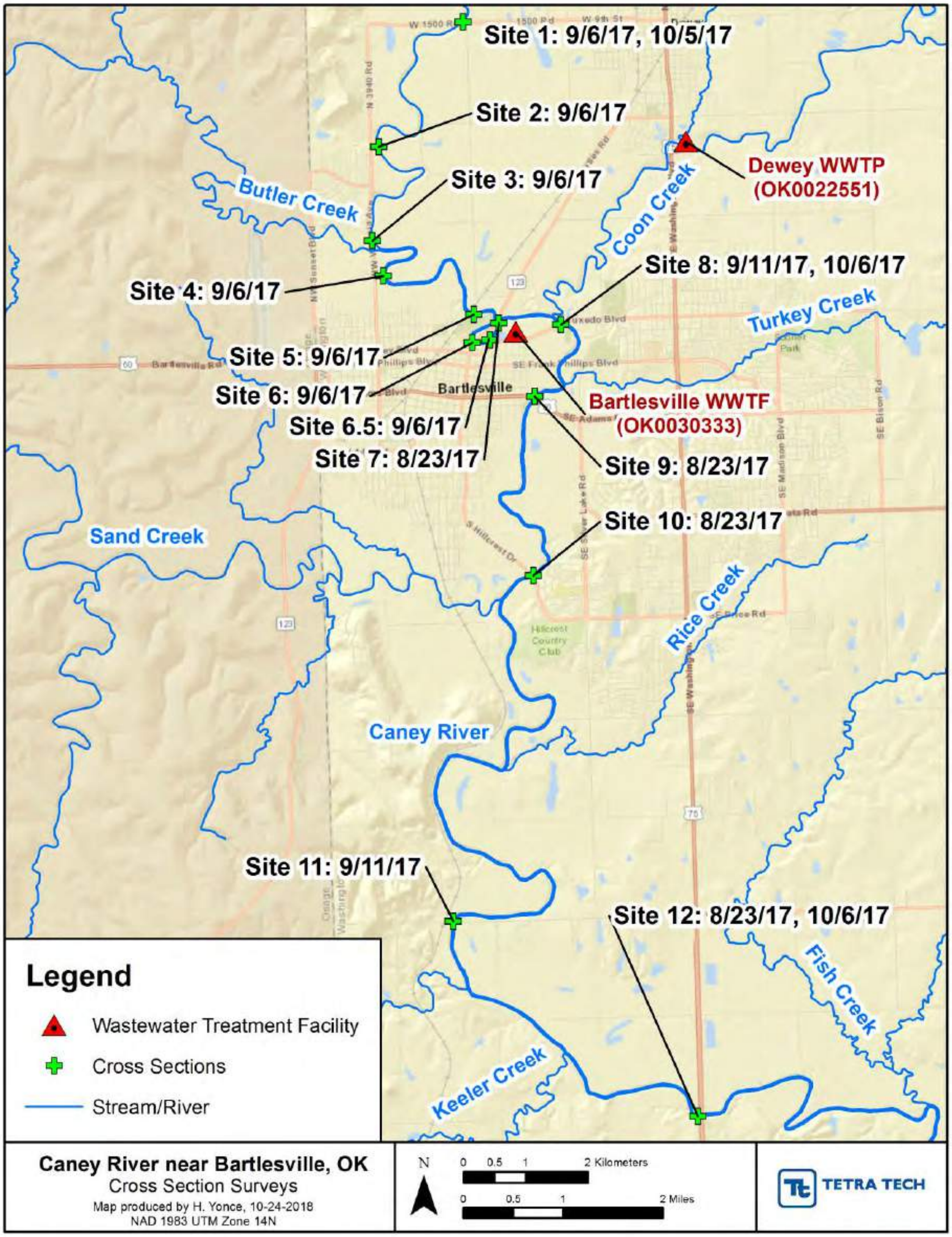

Figure 3-4. Cross section survey locations and dates

Table 3-3. Cross section locations, dates, and channel geometry

Site	Approximate Location	Date	Width (ft)	Mean Depth (ft)	Max Depth (ft)
1	1500 Rd	9/6/2017	74	5.9	9.8
1	1500 Rd	10/5/2017	76	6.1	8.2
2	5 miles up from intake	9/6/2017	90	2.0	3.0
3	Upstream of Butler	9/6/2017	90	6.3	10.3
4	Downstream of Butler	9/6/2017	90	8.3	12.0
5	Near RR crossing	9/6/2017	124	8.3	14.4
6	Upstream of dam	9/6/2017	117	6.7	10.7
6.5	Downstream of dam	9/6/2017	76	2.2	3.6
7	Upstream of WWTF	8/23/2017	40	0.7	1.1
8	Downstream of Coon	9/11/2017	95	5.2	7.1
8	Downstream of Coon	10/6/2017	89	1.1	1.9
9	SE Adams Blvd	8/23/2017	80	1.3	2.2
10	Hillcrest Drive	8/23/2017	70	0.9	1.3
11	2400 Road	9/11/2017	97	4.8	7.1
12	Highway 75	8/23/2017	90	1.0	1.6
12	Highway 75	10/6/2017	58	0.9	1.3

Table 3-4. Dye releases studies for developing velocity-based rating curves from trips 1 and 2

Dye Release Location	Downstream Location	Distance (mi)	TOT (days)		Avg. Velocity (ft/s)	
			Sept 2017	Oct 2017	Sept 2017	Oct 2017
1500 Rd	5 mi. US of intake	2.00	0.66	0.63	0.18	0.19
	Low-Head Dam	6.94	3.76	3.06	0.10	0.14*
	Bartlesville WWTF	7.40	N/A	3.10	N/A	0.15*
5 mi. US of intake	Low-Head Dam	5.02	3.09	N/A	0.11	N/A
	Bartlesville WWTF	5.49	N/A	2.46	N/A	0.14*
DS of dam	Hillcrest Drive	4.50	N/A	0.76	N/A	0.36
	Adams Blvd	2.14	N/A	0.33	N/A	0.40
	2400 Rd	11.31	N/A	2.06	N/A	0.34*
	Hwy 75	15.04	N/A	2.39	N/A	0.38*
Adams Blvd	Hillcrest Drive	2.36	1.04	0.43	0.14	0.33
Hillcrest Rd	2400 Rd	6.81	1.51	1.34	0.28	0.31
	Hwy 75	10.54	2.75	2.08	0.24	0.31*
2400 Rd	Hwy 75	3.73	1.11	N/A	0.21	N/A
Bartlesville WWTF	Adams Blvd	1.67	0.65	N/A	0.16	N/A
	E 0240 Rd	10.84	3.20	N/A	0.21	N/A
	Hwy 75	14.57	4.30	N/A	0.21	N/A

*These velocity measurements were affected by a small precipitation event and may reflect a higher flow condition than was observed prior to the storm.

N/A: Not measured.

Reach 5 represents the low-head dam and is simulated differently than the other reaches with a weir equation. Weir reaches in QUAL2K are simulated as single length with specified weir type, height, and width. There are also coefficients that describe reaeration by the weir, representing water cleanliness and steepness. For Reach 5, these parameters were set to: broad-crested weir, 9.8 feet high (3 meters, reflective of the difference in water elevation upstream and downstream of the day), 98.4 feet (30 meters) wide, with reaeration coefficients of 1.8 and 0.8 representative of “clean water” and a “flat broad-crested irregular step” dam.

Using the average observations of discharge, velocity, and depth, it was possible to develop hydraulic inputs for the model based on both low and median flow conditions for all other reaches. QUAL2K rating curve hydraulics are simulated with Leopold-Maddox power equations based on velocity (U), depth (H), streamflow (Q), and empirical coefficients (a, b, A, and B):

$$U = aQ^b$$

$$H = AQ^B$$

Coefficients and exponents for these model inputs were developed as a function of the TOT study velocity estimates and depth observations at the cross-sections under the low and median flow conditions (Table 3-5). Coefficient a is calculated as a power function of observed flow and TOT-estimated velocity from trips 1 and 2 aggregated by model reach. Both depth and velocity were observed to increase under higher flow conditions, although no significant changes in width were observed under the variable flow. Under conditions of a rectangular channel where width does not change in response to change in streamflow, the sum of exponents b and B may be equal to 1 (Chapra, et al., 2012). Once velocity power functions were developed by reach, exponent B was calculated as 1 minus exponent b for each reach. Coefficient A was calculated by reach as a function of B, the average flow condition between the two simulations, approximate observed width, and the velocity parameters.

Example calculation for Reach 1 hydraulic inputs (in metric units for model input):

- Sept 2017 and Oct 2017 TOT extents: 7 miles to 5 miles upstream of intake (Reach 1 extent)
- Sept 2017 and Oct 2017 flow along Reach 1: 0.67 cms (23.6 cfs) and 2.72 cms (96.2 cfs)
- Sept 2017 and Oct 2017 velocity along Reach 1: 0.05 m/s (0.18 ft/s) and 0.06 m/s (0.19 ft/s)

$$Power\ Equation\ for\ Velocity: U = (0.0525)Q^{0.0351}$$

$$B = 1 - b = 1 - 0.0351 = 0.9649$$

- Calculating coefficient A:
  - Approximated width (W) of 22.9 m (75.0 ft)
  - Average flow between 0.67 cms and 2.72 cms is 1.70 cms (59.86 cfs)
  - Average velocity between 0.05 m/s and 0.06 m/s is 0.055 m/s (0.18 ft/s)

$$Depth\ is\ a\ function\ of\ cross-sectional\ area\ (Ac) and\ width: H = \frac{A_C}{W}$$

$$Cross-sectional\ area\ is\ a\ function\ of\ flow\ and\ velocity: A_C = \frac{Q}{U}$$

$$Calculating\ Coefficient\ A: H = AQ^B \ \therefore A = \frac{H}{Q^B} = \frac{A_C/_W}{Q^B} = \frac{\left(Q/_U\right)/W}{Q^B}$$

$$A = \frac{A_C/W}{Q^B} = \frac{\left(59.86/0.055\right)/(22.9)}{(59.86)^{(0.9649)}} = 0.7973$$

Table 3-5. Caney River hydraulic rating curve inputs

Reach	Detail	Velocity*		Depth*	
		Coefficient	Exponent	Coefficient	Exponent
1	7 miles to 5 miles US of intake	0.0545	0.0351	0.7973	0.9649
2	5 miles US of intake to Butler Creek	0.0332	0.2233	1.2713	0.7767
3	Butler Creek to Johnstone Park	0.0317	0.1185	1.2671	0.8815
4	Deepest area behind low-head dam	0.0224	0.2911	1.4039	0.7089
5	Dam segment	Weir Formula (see text)			
6	Dam to Bartlesville WWTF outfall	0.0512	0.1576	1.0758	0.8424
7	Bartlesville WWTF to Coon Creek	0.0514	0.8493	0.6582	0.1507
8	Coon Creek to Turkey Creek	0.0501	0.9925	0.6885	0.0075
9	Turkey Creek to Sand Creek	0.0437	0.7612	0.8114	0.2388
10	Sand Creek to Rice Creek	0.0792	0.1113	0.5846	0.8887
11	Rice Creek to Keeler Creek	0.0703	0.2162	0.4710	0.7838
12	Keeler Creek to Highway 75	0.0685	0.4681	0.5155	0.5319

*These coefficients and exponents support prediction of velocity and depth in units of meters per second and meters respectively for model input.

4.0 MODEL CALIBRATION AND CORROBORATION

QUAL2K simulates riverine systems by calibrating reach hydraulics, thermal balance, and water chemistry relative to available observed data. Although DO is the key parameter of-interest for the Caney River, the model simulations must adequately represent existing flow, water temperature, and other water quality parameters as well to decrease uncertainty associated with the DO simulation. Some of the key parameters controlling DO kinetics in QUAL2K are sediment oxygen demand (SOD), reaeration of the water from the atmosphere and impacts of wind, decay rates of oxygen-demanding substances (carbonaceous biochemical oxygen demand [CBOD], for example), and photosynthesis from benthic algae and free-floating phytoplankton communities. Some of these parameters were measured directly at discrete locations along the stream (e.g., SOD, atmospheric reaeration, CBOD and chlorophyll-*a* concentrations), while other parameters were not directly measured but were parameterized and adjusted during calibration based on literature values, rates, and concentrations (e.g., reaeration due to wind, CBOD decay rates, phytoplankton and benthic algae growth, respiration, and death rates, kinetics associated with nutrients, detritus, and pH).

4.1 MODEL PARAMETERIZATION

4.1.1 CBOD Simulation

Oxygen depletion along the Caney River is due to a combination of SOD (discussed in Section 4.1.2), oxidation of organic carbon (decay of CBOD), and nitrification processes (decay of nitrogenous biochemical oxygen demand [NBOD]). Grab sampling was conducted during both monitoring trips to measure a number of water quality parameters including several oxygen-depleting substances (Figure 4-2). Ten field sites were sampled between two and four times per day to capture variability in water chemistry through space and time along the system, both instream and from key tributaries and point sources. The full suite of water chemistry data is provided in Appendix A.

4.1.1.1 Effluent CBOD Decay

CBOD decay rates inform the rate at which DO concentrations decrease due to consumption of oxygen from the decay of organic matter suspended in the water column and nitrification of ammonia both instream and within the effluent discharge from Bartlesville WWTF. According to the TMDL Technical Guidance Manual, effluent that has received secondary treatment like the Bartlesville WWTF is likely to have a CBOD decay rate of approximately 0.2 /day (EPA, 1997). CBOD decay was estimated using the grab samples taken at the discharge pipe for the Bartlesville WWTF, for which total (unfiltered) 5-day and 20-day CBOD concentrations were measured between two and four times per day (grab sample Site 5). CBOD decays under first-order reaction kinetics according to the following equation, where k is the decay rate (day^{-1}) between 5- and 20-day concentrations of CBOD:

$$\frac{CBOD_5}{CBOD_{20}} = 1 - e^{-5k}$$

Of the grab sampling conducted of the effluent in September, one of the two samples did not provide a difference between total $CBOD_5$ and total $CBOD_{20}$ with which to estimate the decay rate, however the other sample did and was estimated to have a decay rate of 0.26 /day. In October, the CBOD decay rate was calculated from the two effluent grab samples to be 0.14 /d and 0.04 /d respectively. An average of the three calculated CBOD decay rates between the two sampling periods was 0.15 /day, which aligns well with the aforementioned Technical Guidance for secondary treatment WWTF discharge of 0.2 /d.

4.1.1.2 Instream CBOD Decay

CBOD decay rates instream were estimated as a function of total $CBOD_5$ and $CBOD_{20}$ concentrations measured at all grab sample sites in the system aside from Site 5. During the September monitoring period, nearly all $CBOD_5$ and $CBOD_{20}$ samples were equal to one another, so decay rates could not properly be determined. Total BOD_5 and BOD_{20} concentrations observed during that period resulted in a BOD decay rate of 0.23 /day which may be informative to CBOD decay rates. During the October monitoring period, there were a number of total $CBOD_5$ and $CBOD_{20}$ concentrations below the detection limit, however the number of sites and samples with both measurements available revealed a CBOD decay rate of 0.16 /day. As both instream and effluent CBOD decay rates were roughly equal, a single rate constant was applied to the model globally as the decay rate for the single pool of labile or "fast" CBOD (fastCBOD) of 0.15 /d.

4.1.1.3 fastCBOD Boundary Conditions

Boundary conditions for the QUAL2K model include prescribed concentrations of ultimate dissolved (filtered) fastCBOD for tributaries, headwaters, and point sources. Ultimate biochemical oxygen demand (BOD_{ult}) is the sum of $CBOD_{ult}$ (fastCBOD for boundary conditions) and the nitrogenous biochemical oxygen demand (NBOD). In most cases, 20-day CBOD measurements approximate the total demand after 20-days which incorporates the presence of nitrogenous oxygen demanding substances. Based on a decay rate of 0.15 /d, and the relationship detailed in the QUAL2K Manual between CBOD test incubation periods and $CBOD_{ult}$, observed dissolved $CBOD_{20}$ is likely representative of 90% of $CBOD_{ult}$ in this system (Figure 4-1).

Boundary condition fastCBOD concentrations were parameterized using grab samples from Site 1 for the headwaters, Sites 2, 7, and 9 for all tributaries, and Site 5 for the effluent. Below are example calculations for fastCBOD concentration inputs for effluent and for the headwaters. By extrapolating for a decay rate of 0.15/d, observed filtered $CBOD_{20}$ concentrations were divided by 0.9 to reflect filtered $CBOD_{ult}$ concentrations using the relationships shown in Figure 4-1.

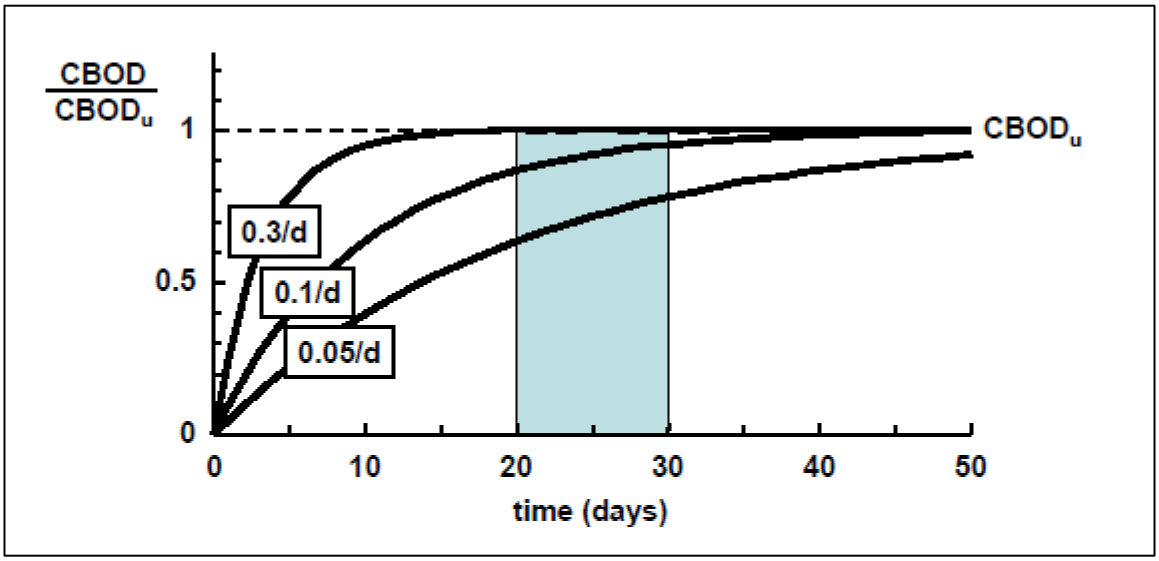

Figure 4-1. Progression of CBOD test for various levels of bottle decomposition rate (Chapra, 2012)

<u>fastCBOD in Effluent for Calibration and Corroboration Model:</u>

- Grab samples at Site 5 were used to estimate the fastCBOD concentration for the effluent during the calibration and corroboration periods. Non-detects were set to half the detection limit for calculations. Four dissolved $CBOD_{20}$ samples during each model period yielded an average of 1.55 and 2.13 mg/l dissolved $CBOD_{20}$ respectively. These concentrations were divided by 0.9 to reflect model input concentrations of filtered $CBOD_{ult}$ of 1.72 and 2.36 mg/l respectively.

<u>fastCBOD in Headwaters and Tributaries for Calibration and Corroboration Model:</u>

- Grab samples from Site 1 of dissolved $CBOD_{20}$ were used to estimate model inputs for the headwaters.
  - For the calibration model headwaters, filtered $CBOD_{20}$ observations were 1 mg/l (half detection limit was used for the sample below detection limit) and 2.7 mg/l, the average of

which is 1.85 mg/l. Dividing this value by 0.9 to reflect the relationship between $CBOD_{20}$ and $CBOD_{ult}$ results in a model input of 2.06 mg/l.

- For the corroboration model headwaters, filtered $CBOD_{20}$ observations were 1 mg/l (half detection limit was used for the sample below detection limit) and 5.2 mg/l, the average of which 3.1 mg/l. Dividing this value by 0.9 to reflect the relationship between $CBOD_{20}$ and $CBOD_{ult}$ results in a model input of 3.44 mg/l.

- Grab samples from Sites 2, 7, and 9 of dissolved $CBOD_{20}$ were used to estimate model inputs for the tributaries. For Site 2 (Butler Creek) the average observed dissolved $CBOD_{20}$ was 1.00 mg/l, which divided by 0.9 is 1.11 mg/l for model input. The average dissolved $CBOD_{20}$ for Site 7 on Coon Creek was 2.28 mg/l, which translates to a model input of 2.53 mg/l. The average dissolved CBOD20 for Site 9 on Sand Creek was 1.00 mg/l, corresponding to a model input of 1.11 mg/l. Unmonitored tributaries (Turkey, Rice, and Keeler Creeks) were set to the inputs for Sand Creek as it is the closest tributary and most likely to have similar characteristics of the unmonitored tributaries. Estimated fastCBOD concentrations for model input for all boundary conditions are shown in Table 4-1.

Table 4-1. CBOD concentration model inputs for boundary conditions

Boundary	Model Input fastCBOD (mg/l)	
	Calibration Model	Corroboration Model
Headwater	2.06	3.44
Bartlesville WWTF	1.72	2.94
Butler Creek	1.11	2.78
Coon Creek	2.53	2.33
Turkey Creek	1.11	3.44
Sand Creek	1.11	3.44
Rice Creek	1.11	3.44
Keeler Creek	1.11	3.44

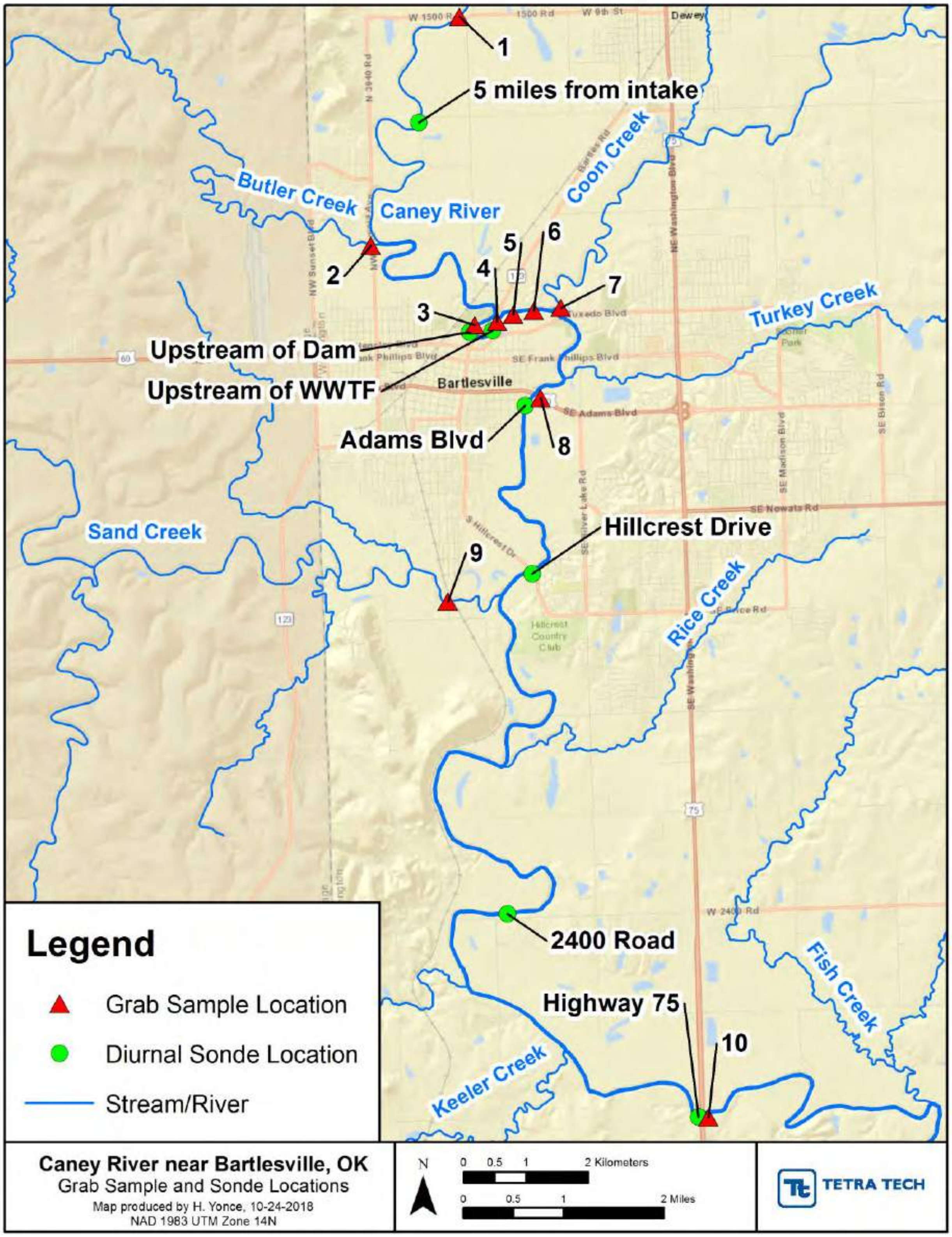

Figure 4-2. Grab sampling locations and sonde sites

4.1.2 SOD and Reaeration

Two parameters which control oxygen kinetics along Caney River are linked explicitly to the physical characteristics of the channel and bed sediment. Sediment oxygen demand (SOD) is the rate at which DO is removed from the water column due to the decomposition of organic matter in the channel bed. Reaeration is the rate at which oxygen is absorbed from the atmosphere into the stream which is typically a function of flow dynamics, channel slope, bed irregularity, and wind. Both SOD and reaeration were two of the parameters with the most uncertainty during the desktop modeling effort, therefore it was integral to refine these inputs based on field monitoring.

SOD was sampled by Gantzer Water Resources Engineering (GWRE) during the September field trip using *in-situ* SOD chamber deployment from September 6 – 9, 2017 (Appendix D). SOD chambers were deployed in triplicate with a fourth chamber measuring water column oxygen demand (WOD) at a total of four sites along Caney River (Figure 4-3). SOD was estimated at any given site by taking the average observed SOD at that site, then subtracting the observed WOD. SOD ranged along Caney River from 0.452 – 1.533 g/m^2/d when normalized to 20°C (Table 4-2). SOD was assigned for groups of reaches based on the observed data. Reaches 1 and 2 were assigned 0.61 g/m^2/d as an average of the observations from Virginia Avenue below Butler and the shallow Virginia Avenue site. Reaches 3, 4, and 5 were assigned 1.53 g/m^2/d from the deep Virginia Avenue site as these reaches are the deepest in the system. Reaches 6 through 12 below the dam were assigned 0.45 g/m^2/d which reflects the Tuxedo Boulevard sample results. Some of the most downstream reaches were initialized with a higher SOD of 1.07 g/m^2/d observed at Hillcrest Drive, however this was altered during the calibration period based on observed DO concentrations. The SOD coverage parameter was initialized at 50% for all reaches and was subject to change during the calibration process. The observed SOD rates generally fall within the normal range of SOD based on literature values which have been observed 0.6 - 13.0 g/m^2/d, with a central tendency on the order of 2.3 g/m^2/d (EPA, 1997).

Table 4-2. Sediment oxygen demand measurements along Caney River

Site	Location	Mean SOD (g/m^2/d) at Ambient Temperature	Ambient Temperature (°C)	Mean SOD (g/m^2/d) Normalized at 20°C
1	Hillcrest Drive	1.335	24.9	1.072
2	Tuxedo Blvd	0.527	23.3	0.452
3	Virginia Avenue (deep)	1.809	23.6	1.533
	Virginia Avenue (shallow)	0.882	23.0	0.726
4	Virginia Avenue below Butler	0.586	23.8	0.492

Reaeration rates were measured along the Caney River mainstem via floating diffusion dome technique on September 8 –9, 2017 (Appendix E). Three reaches upstream of the low-head dam on the Caney River were studied to measure reaeration rate coefficients (Figure 4-3). Flow velocities were observed to be quite sluggish during the reaeration studies, on the order of 0.20 ft/s (0.06 m/s). When normalized for temperature, the observed rate was 0.26 /day along the “Float 3” location where the water is deeper and sluggish. Upstream of the Butler Creek confluence, the rate was observed to be a similarly low 0.31 /day (Float 2). Near 5 miles upstream of the intake, the reaeration rate was observed to be 12.95 /day along a very shallow riffle (Float 1). The two segments with very low reaeration rates were more representative of conditions upstream of the low-head dam where depths range from 8-12 feet typically. The very high reaeration rate observed was along a shallow (2 feet deep) section of the river which showed that the impact of wind and low depth resulted in a well-mixed column which is unrepresentative of how Caney River behaves generally upstream of the dam but may provide insight into how the river may behave downstream of the dam under shallow depth conditions.

Reaeration due to wind is also relevant in the Caney River system given the observed winds moving the reaeration chambers across the stream surface and the low stream velocities. The effect of wind on river reaeration was incorporated into the model using the Wanninkhof formula which estimates reaeration as a function of wind velocity (U_{wind} in units of m/s) (Wanninkhof, 1991). This equation incorporates wind reaeration (k_{wind}) in addition to the natural reaeration occurring instream due to channel geometry which varies by reach:

$$k_{wind} = 0.0986 \times (U_{wind})^{1.64}$$

In order to identify the most appropriate reaeration formula to simulate the Caney River system, the observed reaeration rates were used to parameterize the model by selecting of the most appropriate formula (Table 4-3). The reaeration formula which most accurately captures the low upstream reaeration rates, higher downstream reaeration rates, and the significant boost in reaeration across the dam is Tsivoglou-Neal. The Tsivoglou-Neal formula calculates reaeration (k_a) as a function of stream velocity (U, in units of m/s) and channel bed slope (S, in units of m/m), both of which can vary by reach, and is appropriate for the low gradient of the Caney River channel (Tsivoglou and Neal, 1976):

$$For\ Q > 15\ cfs\colon\ k_a = 15308 \times U \times S$$

The results for the Churchill formula are reasonable in comparison to observed reaeration rates, however the QUAL2K manual suggests that the Churchill equation may be most applicable for velocities greater than 0.5 m/s which are far higher than those observed in Caney (Chapra et al., 2012).

Table 4-3. Reaeration rates estimated by reach for different formulae

Reach	Reaeration Notes	Simulated Reaeration by Formulae, with Wanninkhof Wind Formula Incorporated (/d)				
		Tsivoglou-Neal	Churchill	Owens-Gibbs	O'Connor-Dobbins	Thackston-Dawson
1	Similar to Reaches 2-3	0.47	1.26	3.00	2.91	1.67
2	Obs. reaeration ~0.31 /d	0.27	0.44	0.88	1.06	0.86
3	Obs. reaeration ~0.26 /d	0.37	0.43	0.87	1.05	1.98
4	Similar to Reaches 2-3	0.29	0.36	0.68	0.86	1.44
5	Dam Weir	1.67	0.08	0.09	0.13	3.05
6	Highly riffled reach	0.46	0.91	2.09	2.17	2.38
7	No data, shallower and higher velocity than upstream of the dam	0.45	0.80	1.81	1.94	2.37
8		1.48	0.74	1.63	1.77	6.84
9		0.30	0.59	1.24	1.41	0.88
10		1.57	0.97	2.00	2.04	5.44
11		0.47	1.29	2.87	2.76	2.11
12		0.67	1.29	2.83	2.72	3.52
Average		*0.70*	*0.76*	*1.67*	*1.73*	*2.71*

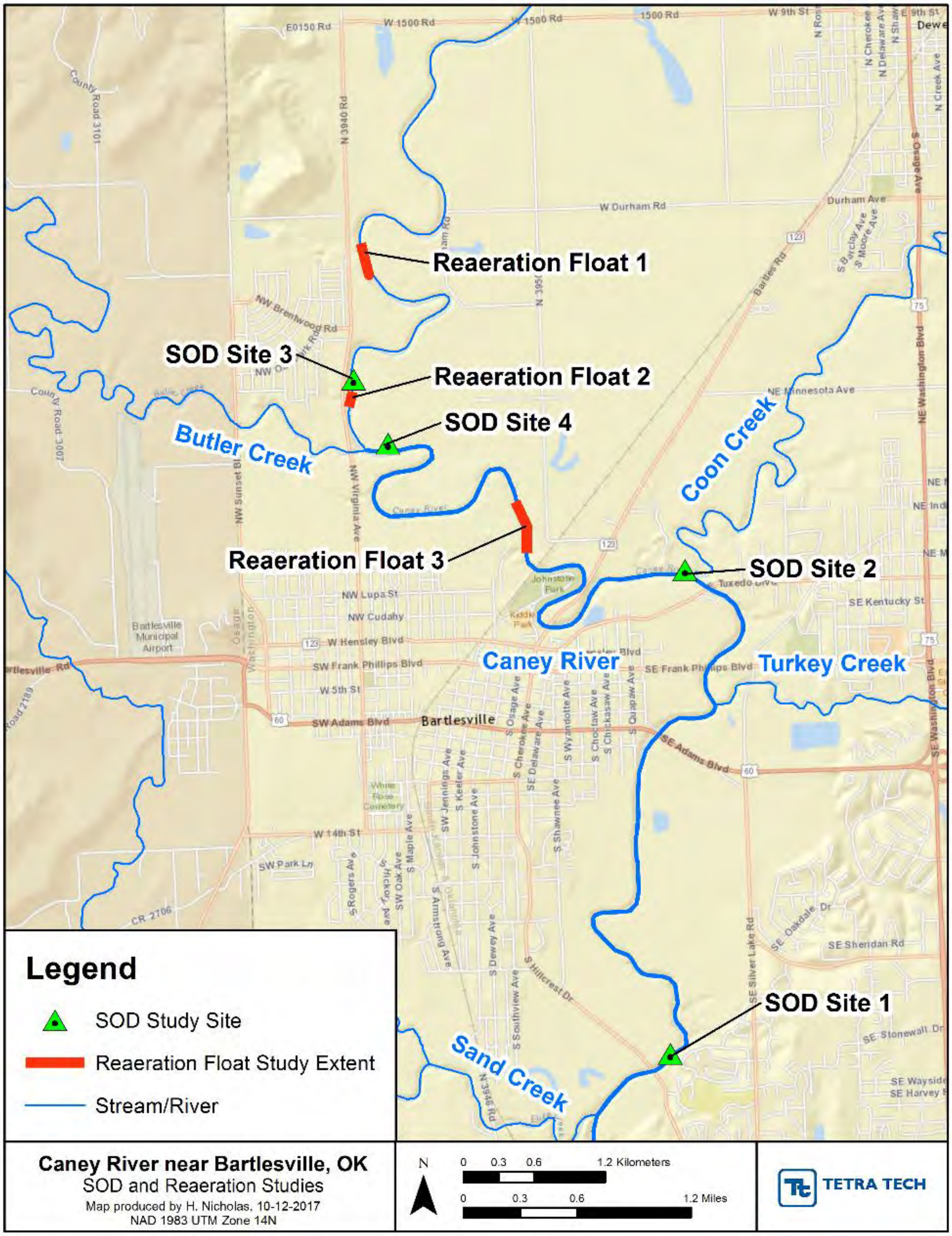

Figure 4-3. Sediment oxygen demand and reaeration study locations along Caney River

4.1.3 Flow Boundary Conditions

Boundary conditions for flow and water quality inputs to the Caney River model were identified as the headwaters and six key tributaries: Butler, Coon, Turkey, Sand, Rice, and Keeler Creeks. There are several active continuous streamflow and gage height gages available in the vicinity of the modeling extent which were used to parameterize the boundary conditions (Table 4-4 and Figure 4-4). The USGS flow gage upstream of the low-head dam north of Bartlesville was used to develop headwater flow conditions (07174400 Caney River above Coon Creek at Bartlesville, Oklahoma; co-located with USACE gage BVLO2), and the USACE gage located on the major tributary Sand Creek was used to develop tributary flow conditions (OKEO2: Sand Creek near Okesa).

As mentioned previously, flow conditions were maintained during the two sampling events by the USACE by controlling outflow from Hulah and Copan Reservoirs. Actual average flow conditions for the two events were approximately 24.5 cfs and 96.7 cfs as measured at USGS gage 07174400. There was a precipitation event that occurred during the second field trip on 10/4/2017 resulting in 1.2 inches of rainfall, increasing streamflow by about 60 cfs at the peak; however, flows returned to approximately 100 cfs the following day. The presence of the passing storm was taken into consideration when analyzing data measured during and after the event.

Table 4-4. Flow gages in and around the Caney River QUAL2K modeling extent

Gage Name	Agency	Gage ID	Data Type
Caney River above Coon Creek at Bartlesville, OK	USGS	07174400	Flow
	USACE	BVLO2	Flow
Hulah Reservoir Outfall	USACE	HULO2	Flow
Copan Reservoir Outfall	USACE	CPLO2	Flow
Caney River at Tuxedo Blvd at Bartlesville, OK	USGS	07174470	Gage Height
Sand Creek Below Little Rock Creek near Okesa, OK	USGS	07174618	Gage Height
	USACE	OKEO2	Flow

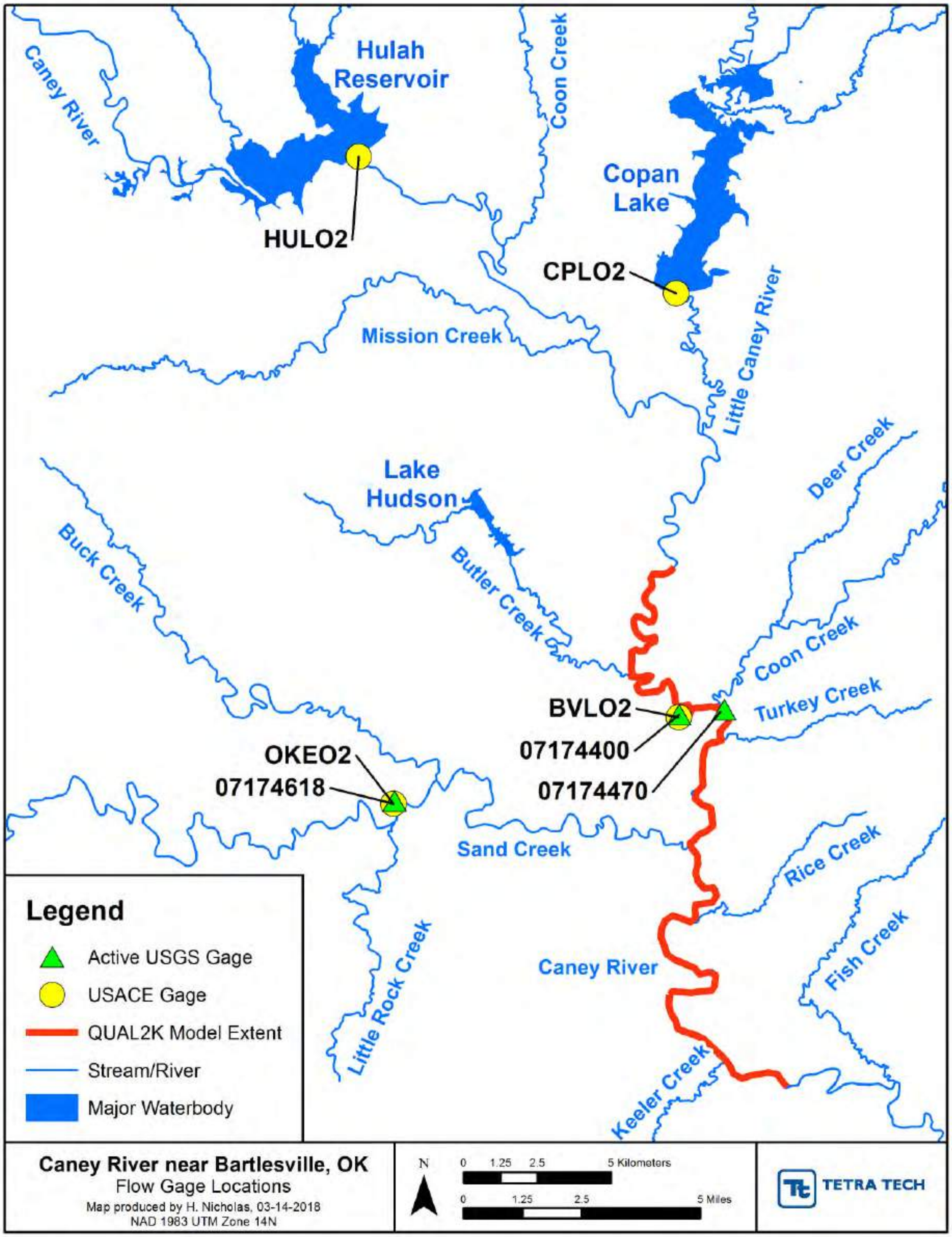

Figure 4-4. Flow gages in and around the Caney River QUAL2K modeling extent

The USACE/USGS Sand Creek gage (OKEO2/07174618) was used to estimate flow for tributaries in the model based on drainage area. Note that the USGS mainstem gage cannot be used in the same manner because flows are artificially controlled by the upstream reservoir releases. Because the majority of the Caney River drainage area in the model extent is attributed to tributaries, no diffuse or groundwater flows were simulated, but rather flow differences were accounted for as additions from tributaries and point sources alone.

Unfortunately, discharge reported by the USACE for Sand Creek appears to be unreliable as reported flows are far greater than total streamflow measured downstream along the Caney River at Ramona. Although a rating curve is not publicly available for the Sand Creek gage, field measurements of gage height and discharge reported by USGS were used to build a relationship between gage height and discharge (Figure 4-5).

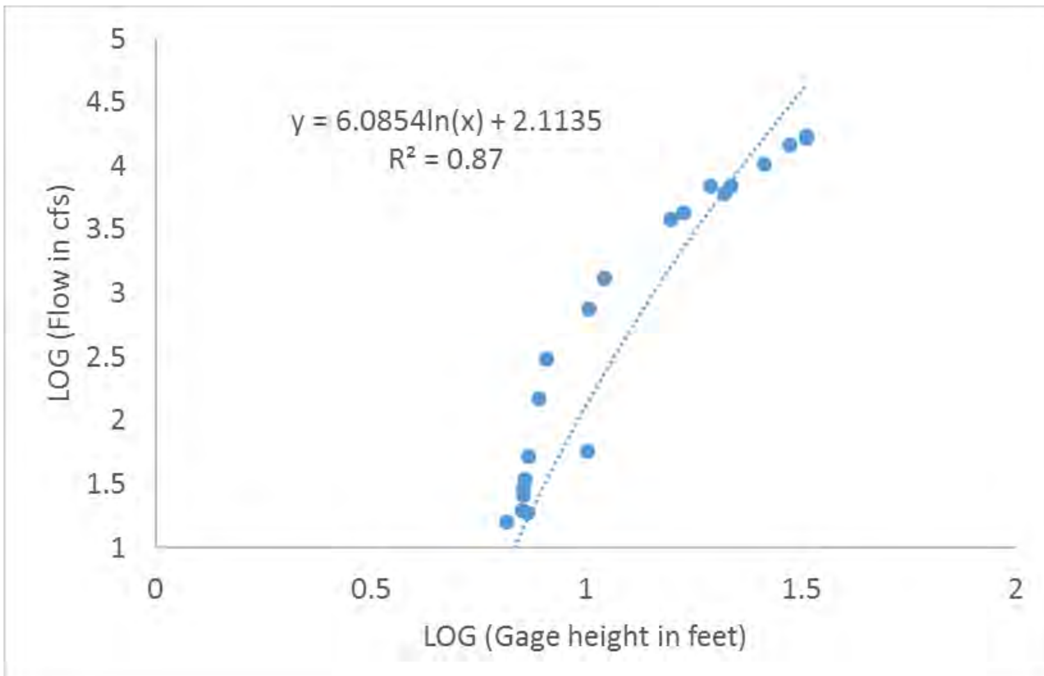

Figure 4-5. Log-transformed discharge and gage height relationship observed at USGS #07174618

Using observed gage heights at the USACE Sand Creek gage and the relationship between log-transformed gage height and streamflow observed at the USGS gage on Sand Creek, streamflow at the USACE gage can be estimated using the following regression:

$$Streamflow\ (cfs) = 6.0854\ \times \ln(Gage\ height\ in\ feet) + 2.1135 \qquad R^2 = 0.87$$

The equation yields a relationship of approximately 0.0089 cfs of flow per square mile of drainage area (0.02 cfs/km^2) for both calibration and corroboration flow conditions since tributary flows were not managed by the USACE as the mainstem flows were. Using this relationship and the drainage areas at the outlet of each tributary, flows were estimated for Butler, Coon, Turkey, Sand, Rice, and Keeler Creeks (Table 4-5).

Flows at the headwaters were estimated as the difference between observed flows at USGS gage 07174400 and the Butler Creek tributary flow which enters the system between the headwaters and the gage location. While flow conditions were quite different along the mainstem for the two model application periods, tributary flows did not vary much since their respective flows were not impacted by the manually controlled releases from the upstream reservoirs.

Table 4-5. Flow boundary conditions for calibration and corroboration models of Caney River

Boundary	Drainage Area (mi^2)	Calibration Model		Corroboration Model	
		Flow (cfs)	Flow (cms)	Flow (cfs)	Flow (cms)
Headwater	1,338.13	22.4	0.6355	95.1	2.6938
Butler Creek	25.64	1.56	0.0441	1.52	0.0430
Coon Creek	66.98	4.07	0.1153	3.97	0.1124
Turkey Creek	7.39	0.45	0.0127	0.44	0.0124
Sand Creek	255.73	15.55	0.4402	15.15	0.4291
Rice Creek	7.00	0.43	0.0120	0.41	0.0117
Keeler Creek	10.91	0.66	0.0188	0.65	0.0183

4.1.4 Water Quality Boundary Conditions

Water quality inputs for the boundary flows were developed based on laboratory-tested grab samples, probe-sampled field data, or available data from adjacent tributaries. Grab samples (shown in full detail in Appendix A) were collected between two and four times per day at ten discrete locations on each sampling trip, on dates 9/17/2017 and 10/6/2017. Grab sample analytes included: total dissolved and suspended solids (TDS, TSS), total organic carbon (TOC), chlorophyll-*a* (CHL-A), phosphate (PO_4), total phosphorus (TP), nitrate and nitrite (NOx), ammonia (NH_3), total Kjeldahl nitrogen (TKN), and BOD parameters described in Section 4.1.1. QUAL2K model inputs for organic phosphorus (OrgP) and organic nitrogen (OrgN) were calculated as the difference between TP and PO_4 or TKN and NH_3 respectively. Inputs for CBOD are detailed in Section 4.1.1. Grab samples relevant to boundary conditions were Site 1 (headwaters), Site 2 (Butler Creek), Site 7 (Coon Creek), and Site 9 (Sand Creek). Constituents sampled by probe in the field included: DO, DO saturation, pH, temperature (Temp), conductivity (Cond). Tributaries not sampled were assigned inputs based on Sand Creek data (Table 4-6). Alkalinity (Alk) inputs were set to 100 mg/l in the absence of other data. TSS data was split between inorganic suspended solids (ISS) and detritus or particulate organic matter (POM) at a ratio of 80% and 20% to differentiate between the two pools.

Table 4-6. Caney River tributary water quality QUAL2K inputs

Boundary	Temp (°C)	Cond (µmhos)	ISS (mg/l)	DO (mg/l)	Fast CBOD (mg/l)	OrgN (µg/l)	NH_3 (µg/l)	NOx (µg/l)	OrgP (µg/l)	PO_4 (µg/l)	CHL-A (µg/l)	intN (µg/l)	intP (µg/l)	pH	POM (mg/l)	Alk (mg/l)
September Model																
Headwater	24.3	402	10.6	6.2	2.06	100	90	30	80	20	12.8	2.5	0.1	7.74	2.7	100
Butler Creek	22.2	352	10.6	6.1	1.11	10	120	30	80	10	8.8	2.5	0.1	7.77	2.7	100
Coon Creek	23.4	411	10.6	6.2	2.53	50	80	50	50	10	8.8	2.5	0.1	7.78	2.7	100
Turkey Creek	24.7	411	10.6	9.6	1.11	50	80	30	40	10	8.8	2.5	0.1	7.54	2.7	100
Sand Creek	24.7	411	10.6	9.6	1.11	50	80	30	40	10	8.8	2.5	0.1	7.54	2.7	100
Rice Creek	24.7	411	10.6	9.6	1.11	50	80	30	40	10	8.8	2.5	0.1	7.54	2.7	100
Keeler Creek	24.7	411	10.6	9.6	1.11	50	80	30	40	10	8.8	2.5	0.1	7.54	2.7	100
October Model																
Headwater	22.5	327	36.2	7.3	3.44	264	50	125	103	13	11.8	2.5	0.1	8.00	9.1	100
Butler Creek	21.6	448	36.2	5.8	2.78	344	50	47	81	13	10.2	2.5	0.1	7.69	9.1	100
Coon Creek	23.0	510	36.2	8.6	2.33	221	114	25	72	13	10.2	2.5	0.1	7.89	9.1	100
Turkey Creek	22.5	367	36.2	7.5	3.44	75	50	25	44	13	10.2	2.5	0.1	7.80	9.1	100
Sand Creek	22.9	492	36.2	7.4	3.44	75	50	25	44	13	10.2	2.5	0.1	7.79	9.1	100
Rice Creek	22.9	492	36.2	7.4	3.44	75	50	25	44	13	10.2	2.5	0.1	7.79	9.1	100
Keeler Creek	22.9	492	36.2	7.4	3.44	75	50	25	44	13	10.2	2.5	0.1	7.79	9.1	100

4.1.5 Point Source Inputs

There is a single major point source in the model extent, which is the Bartlesville WWTF located downstream of the low-head dam. Note that the Dewey wastewater treatment facility is located on Coon Creek; however, it is a minor point source and is incorporated into the Coon Creek inputs based on water quality at the outlet of the tributary.

Flow and water quality data associated with the Bartlesville WWTF discharge was provided via Oklahoma Discharge Monitoring Report (DMR). DMR data included flow, DO, TSS, BOD_5, NH_3, alkalinity, and pH. Field-measurements were conducted by Tetra Tech for other parameters such as conductivity, temperature, oxygen demanding substances, and other nitrogen and phosphorus species. Model inputs for DO, TSS, NH_3, and pH used for calculations were all developed as the 5-day or 3-day averages prior to the calibration and corroboration simulation dates, respectively because the TOT data suggests that it takes about 5 days for water released at the outfall to reach the end of the model extent during low flows, and approximately 3 days during median flows. TSS data was split between inorganic and organic matter at a ratio of 50/50 to account of organic matter in the effluent. All model inputs for the WWTF are summarized in Table 4-7. FastCBOD inputs were tabulated as a function of grab sample $CBOD_{20}$ concentrations$_0$ as described in Section 4.1.1.

DMR data for the Bartlesville WWTF report DO concentrations on the order of 2.0–5.0 mg/l. These measurements are recorded as the water leaves the plant rather than at the point where the water has flowed to the river and gone down a cascade. Tetra Tech field measurements at the discharge location ranged from 7.1 – 8.0 mg/l on the days reported by DMR to be ~3 mg/l, so it is apparent that DMR-report DO concentrations are far lower than actual DO post-cascade aeration. A model input of 7.8 mg/l for DO was used for model input for calibration and 7.4 mg/l for corroboration as these represent the average of four measurements recorded by probe by Tetra Tech over the course of a day while sampling.

Table 4-7. Bartlesville WWTF point source parameterization

Parameter	Calibration Model	Corroboration Model	Data Source/ Processing Note
Flow (cms)	0.248	0.255	DMR data (5-day average)
Cond (µmhos)	427	461	Sampled at outfall (average of 4 field measurements)
Temp (°C)	25.6	23.8	Sampled at outfall (average of 4 field measurements)
DO (mg/l)	7.8	7.4	Calculated as daily average based on field-observed DO measured at the pipe outfall to the river (4 observations per sampling period).
ISS (mg/l)	5.09	2.35	DMR data (5-day average) of TSS split 50/50 as ISS/ POM
POM (mg/l)	5.09	2.35	DMR data (5-day average) of TSS split 50/50 as ISS/POM
fastCBOD (mg/l)	1.72	2.94	Calculated from 5-day average DMR data and grab samples (see text)
OrgN (µg/l)	218	461	Calculated from sampled TKN and NH_3 at outfall (average of 4 grab samples)
NH_3 (µg/l)	82	130	DMR data (5-day average)
NOx (µg/l)	7863	9038	Sampled at outfall (average of 4 grab samples)

Parameter	Calibration Model	Corroboration Model	Data Source/ Processing Note
OrgP (µg/l)	460	455	Calculated from sampled TP and PO_4 at outfall (average of 2 grab samples)
PO_4 (µg/l)	3350	1380	Sampled at outfall (average of 2 grab samples)
pH	6.91	6.93	DMR data (5-day average)
Alkalinity (mg/l)	86	82	DMR data (5-day average)

4.1.6 Additional Parameters

Several additional parameters and formulae were set up to apply to the entire QUAL2K model to simulate Caney River internal processes and kinetics. These parameters were held constant for both calibration and corroboration simulations, as well as application scenarios. Key parameter choices are identified below:

- Atmospheric attenuation model for solar radiation: Bras (QUAL2K model default)
- Atmospheric longwave emissivity model: Koberg (method which incorporates cloud cover into emissivity calculation which is a key factor in Caney River water temperature in the absence of vegetative shade)
- Wind speed function for evaporation and air convection/conduction: Adams 2 (method which incorporates wind as function of the virtual temperature difference between water and air)
- Sediment thermal properties are selected to be manual-suggested defaults
- Bottom algae are initialized as 25 percent coverage to account for photosynthetic benthic flora which was held constant during the calibration process. Bottom algae growth parameters are: zero-order growth model, max growth rate of 200 /d, respiration, excretion, and death rates of 0.05, 0.01, and 0.01 /d respectively. These parameters were held constant for both calibration and corroboration models.
- Phytoplankton growth kinetics were a major part of model calibration for DO. Model inputs were initialized based on QUAL2K manual default values and were subject to alteration during calibration. During both field trips, there was a noticeably green coloration to the water suggesting high phytoplankton density in the stream, which is also suggested by super-saturation of DO during the first sampling event. Phytoplankton parameterization in the model was initialized as follows: max growth rate of 1 /d, respiration and excretion rates of 0.01 /d, and death rate of 0.01 /d. During the calibration process, max growth rate was increased to 2 /d upstream of the dam, and respiration rates were set to 0.3 /d and 0.6 /d for Reaches 1-2 and Reaches 3-5 respectively. These modified parameters upstream of the dam reflect the different conditions present due to deep ponded water, and the apparent DO sag upstream of the dam.
- Light and heat parameters were generally held to model defaults with the exception of Photosynthetically Available Radiation (PAR) which was decreased from 0.47 to 0.2 as a function of how opaque and phytoplankton-rich the water was. This term was altered during the calibration process, particularly related to the calibration of DO and pH.

These parameters are maintained for both calibration and corroboration models, as well as for the application model scenarios. The sensitivity of the model to some of these parameters was assessed in Section 5.0. All model parameterization for rates and kinetics are included in Appendix F.

4.2 MODEL CALIBRATION RESULTS

Data used to calibrate and corroborate the Caney River QUAL2K model include physical and hydraulic geometry, grab sample water chemistry data, as well as diel water temperature, pH, and DO (see Appendix A, Appendix B, and Appendix C). Observations of width, depth, and velocity were used to constrain the basic physical conditions of the model relative to the water balance. Observations of water temperature were used to calibrate the heat balance, ensuring that water chemistry is simulated under reasonable thermal kinetics. Water chemistry observations of CBOD, nutrients, pH, and DO were used to evaluate model accuracy compared with observed conditions. Relative error statistics were calculated for long-term averages of water temperature, pH, and DO using data from the sondes.

4.2.1 Flow Balance

Model results for width, depth, velocity, and TOT from the QUAL2K calibration model reveal that the simulation reasonably approximates the observed data from the September field monitoring period. Dye-study TOT results were compared to simulated TOT, and the results were similar (Table 4-8). Dye-study TOT results and velocity observations are the most accurate to observed conditions as they were the highest resolution flow data collected and were also used to parameterize reach hydraulics. Observed ranges and point measurements of width, depth, and velocity were compared to simulation results of channel geometry and hydraulics, which appear to represent the system reasonably given the high variability of width and depth throughout the system due to pool-riffle-run hydrology (Table 4-9).

Table 4-8. Calibration model results compared to TOT studies over the same extents

Upstream Location	Downstream Location	Observed TOT (d)	Calibration Model TOT (d)
1500 Rd	5 mi. US of intake	0.7	0.7
	Low-Head Dam	3.8	3.9
5 mi. US of intake	Low-Head Dam	3.1	3.2
Adams Blvd	Hillcrest Drive	1.0	0.9
Hillcrest Rd	2400 Rd	1.5	1.7
	Hwy 75	2.8	2.6
2400 Rd	Hwy 75	1.1	0.9
Bartlesville WWTF	Adams Blvd	0.7	0.7
	E 0240 Rd	3.2	3.3
	Hwy 75	4.3	4.3

Table 4-9. Calibration model results compared to observed channel geometry and velocity

Reach	September Field-Observed Range			Calibration Model Results		
	Width (ft)	Depth (ft)	Velocity (ft/s)	Width (ft)	Depth (ft)	Velocity (ft/s)
1	74	5.9	0.18	76	1.7	0.18
2	90	2.0 - 6.3	0.10	78	2.9	0.10
3	90	8.3	0.10	82	3.0	0.10
4	117 - 124	6.7 - 8.3	0.07	104	3.1	0.06
5	30	N/A	N/A	98	10.0	0.02
6	40 - 76	0.7 - 2.2	0.16	60	2.0	0.15
7	N/A	N/A	0.16	97	2.1	0.13
8	95	5.2	0.16	95	2.3	0.14
9	90	1.3	0.14	93	2.6	0.13
10	70	0.9	0.26	71	2.5	0.27
11	97	4.8	0.24	99	1.9	0.25
12	90	1.0	0.24	93	2.0	0.26

4.2.2 Water Temperature

Water temperature is an integral part of DO simulation since DO concentration and saturation are linked directly to water temperature kinetics. Seven YSI sonde instruments were deployed for approximately one week during field monitoring periods at discrete locations along the river, and the daily average, minimum, and maximum temperature were used for comparison with QUAL2K simulation results (Figure 4-6). The QUAL2K model was parameterized to simulate average daily conditions along the Caney River on the calibration date, which the model does well. Water temperature on 9/6/2017 varied at each sonde by 1 - 4 °C, and the simulated temperature provided a reasonable approximation of the average daily condition along the entire model extent which was observed to be approximately 24.4°C. The full suite of water temperature data observed longitudinally and at the sondes may be referenced in Appendix B and Appendix C.

The relative error of simulated water temperature relative to observed average water temperature for the model extent is 3%.

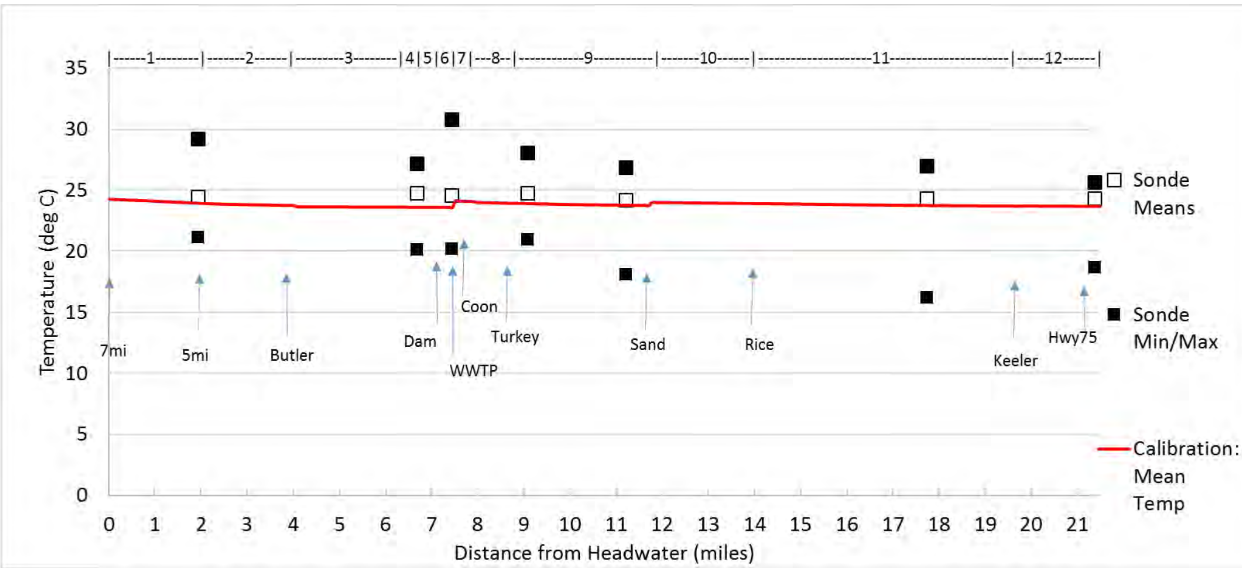

Figure 4-6. Calibration: simulation results and observed water temperature from September 2017

4.2.3 CBOD and Nitrogen

Instream concentration of fastCBOD is controlled by boundary condition inputs of fastCBOD and the instream CBOD decay rate. Instream $CBOD_{ult}$ is controlled by concentrations of fastCBOD and NBOD as a function of decaying organic matter (detritus, phytoplankton, and benthic algae) nitrification. Simulated fastCBOD range from about 2.1 mg/l at the headwaters, decaying to about 1.3 mg/l near the model outlet, which tracks well with the grab sample concentrations of filtered $CBOD_{20}$ which were converted to $CBOD_{ult}$ by dividing by 0.9 (Figure 4-7). Also seen in Figure 4-7 are the simulated results of $CBOD_{ult}$ which have a simulated range which tracks well relative to observed concentrations of unfiltered $CBOD_{20}$ converted to unfiltered $CBOD_{ult}$.

The model simulates instream nitrogen species concentrations well. Nitrogen species concentrations were low aside from the spike in NOx downstream of the WWTF which is well-simulated by the model. Observed data points shown in Figure 4-6 and Figure 4-7 represent grab samples measured multiple times per day on 9/7/2017 (Full suite of grab sample data available in Appendix A).

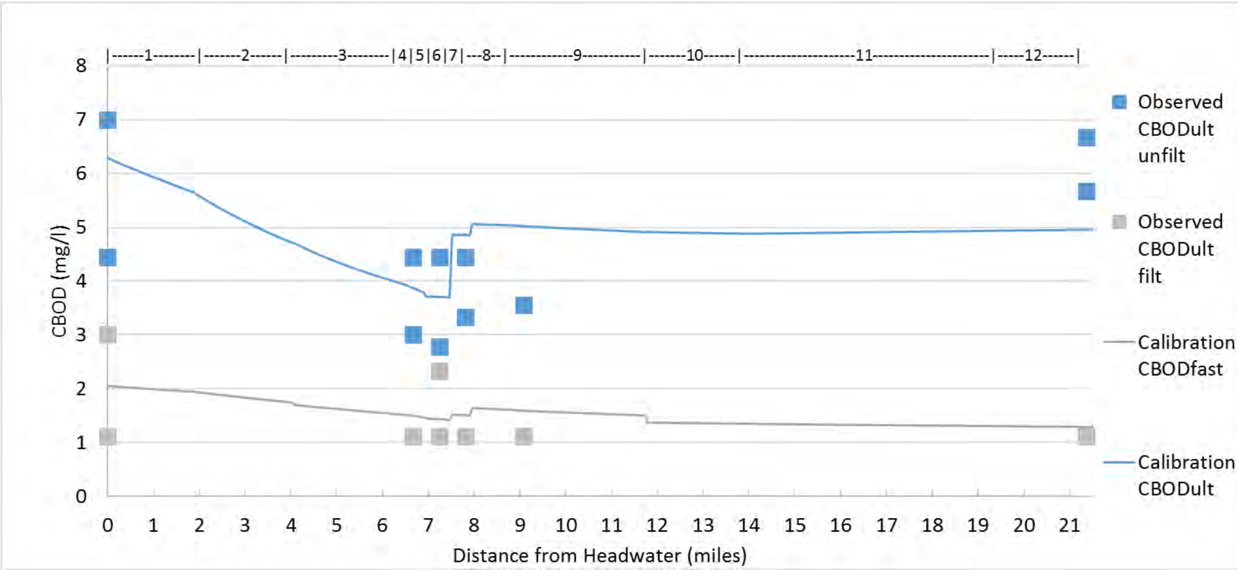

Figure 4-7. Calibration: simulation results and observed oxygen-demanding species data

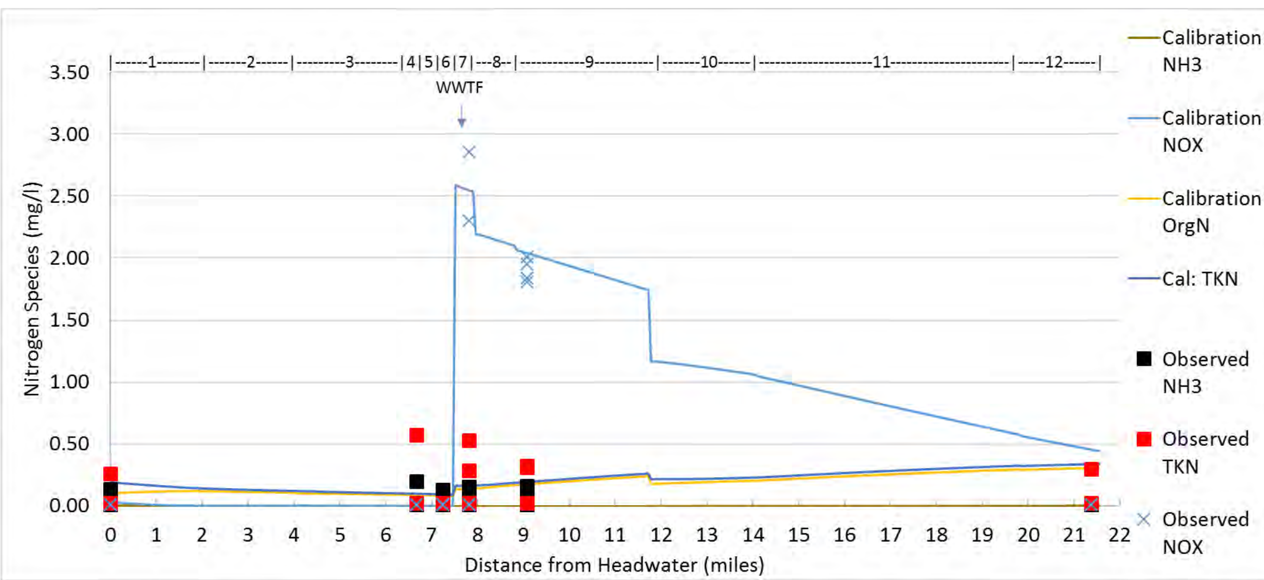

Figure 4-8. Calibration: simulation results and observed nitrogen species data

4.2.4 pH

The simulation of pH in QUAL2K is a function of both alkalinity and total inorganic carbon (TIC). Diurnal pH was observed at the sonde locations similarly to water temperature and DO, therefore pH calibration of the model was focused at those key locations to best predict the average pH. There was limited data observed related to alkalinity and TIC during the 2017 monitoring period. The model response of pH along the river is highly dependent on phytoplankton growth and die-off which are also large controls on DO. The calibration effort associated with pH occurred in tandem with DO to produce a model result with optimizes the model fits and provides reasonable agreement of both terms simultaneously. pH was observed to reach super-saturation around mile marker 18 which is well-simulated by the model.

The relative error of simulated pH relative to observed average pH for the extent model is 3%.

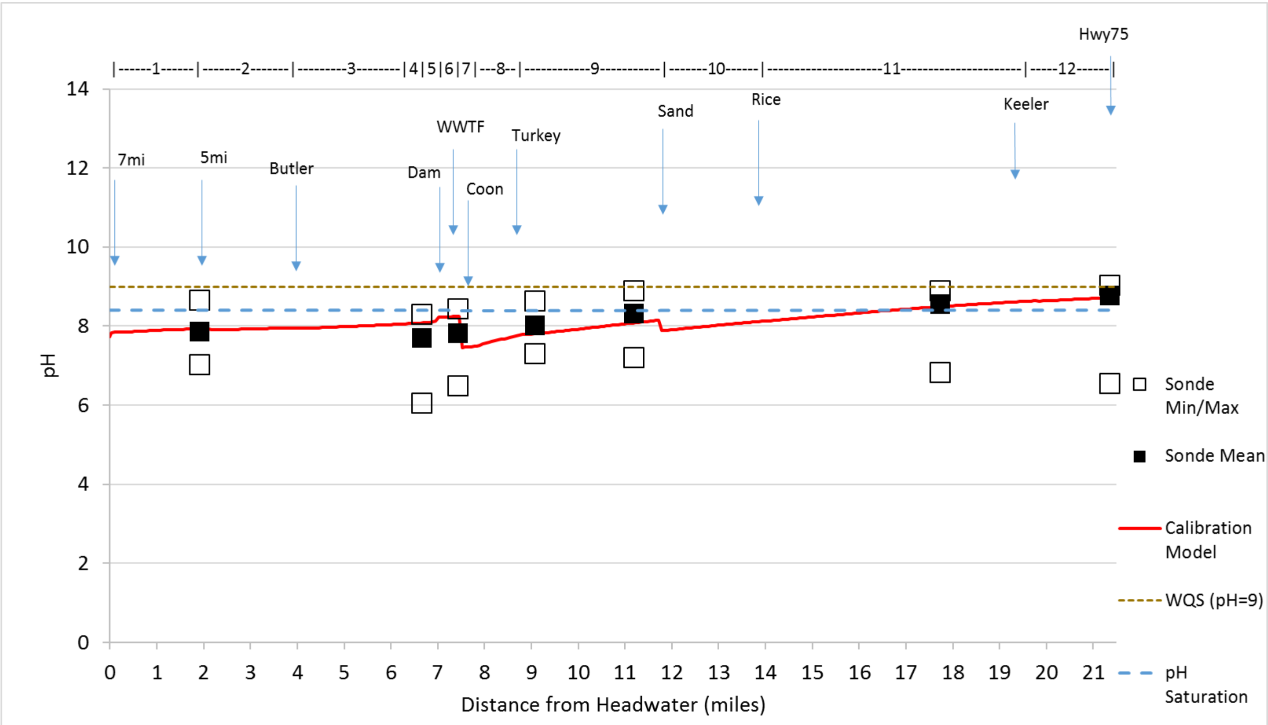

Figure 4-9. Calibration: simulation results and observed pH statistics from September 2017

4.2.5 Dissolved Oxygen

DO simulation results for the calibrated model were compared to YSI sonde data at the same seven locations used for water temperature and pH comparison where DO was also sampled (Appendix C). At these sonde locations, the average, minimum, and maximum DO concentrations observed on the simulation date are shown alongside the QUAL2K daily average simulation result for DO (Figure 4-10). Synoptic sampling of DO along the entire mainstem was conducted to validate the sonde data, which it did (Appendix C). The simulation provides a reasonable representation of the central tendency of DO concentrations along the entire model extent for the simulation date. The simulated fit captures the distinct trends and central tendencies across the system well. Average observed DO concentrations along the mainstem from the sondes ranged from 3.5 to 11.7 mg/l during the calibration period, while simulation results ranged from 5.9 to 9.5 mg/l. The model captures the shape of the DO concentrations longitudinally, although it over-predicts DO upstream of the dam, and under-predicts the extent of super-saturation of DO downstream of Turkey Creek. The observed super-saturation in the downstream portions of the Caney River modeled area are evidence of substantial existing organic photosynthetic activity.

To achieve this level of fit, the key calibration parameters that were altered from initialized values were those related to oxygen controls and kinetics. The daily average DO concentration is controlled primarily by the ratio of SOD to reaeration, and the combined impact of phytoplankton growth and bottom algae provides the super-saturation of DO observed downstream of the dam. SOD was held at the initialized observed flux rates for all reaches, with SOD coverage altered to 75% upstream of the dam where the sediment is thicker and less rocky, and to 20% downstream of the dam where the streambed is rockier with less silty deposits. Maximum growth rate for phytoplankton was determined through calibration to be 2/day for reaches upstream of the dam (reaches 1-5). Respiration rates were set to 0.3/day for reaches 1 and 2 and enhanced to 0.6 /day for reaches 3 through 5 in the most sluggish reaches where phytoplankton activity proliferates and the DO sag was evident. Model parameterization, particularly relative to phytoplankton growth and activity was a balance between what captures the shape of the DO longitudinally in the system during the calibration period but also predicts DO during the corroboration period within the range of observed concentrations.

The relative error of simulated DO relative to observed average DO for the extent model is 13%.

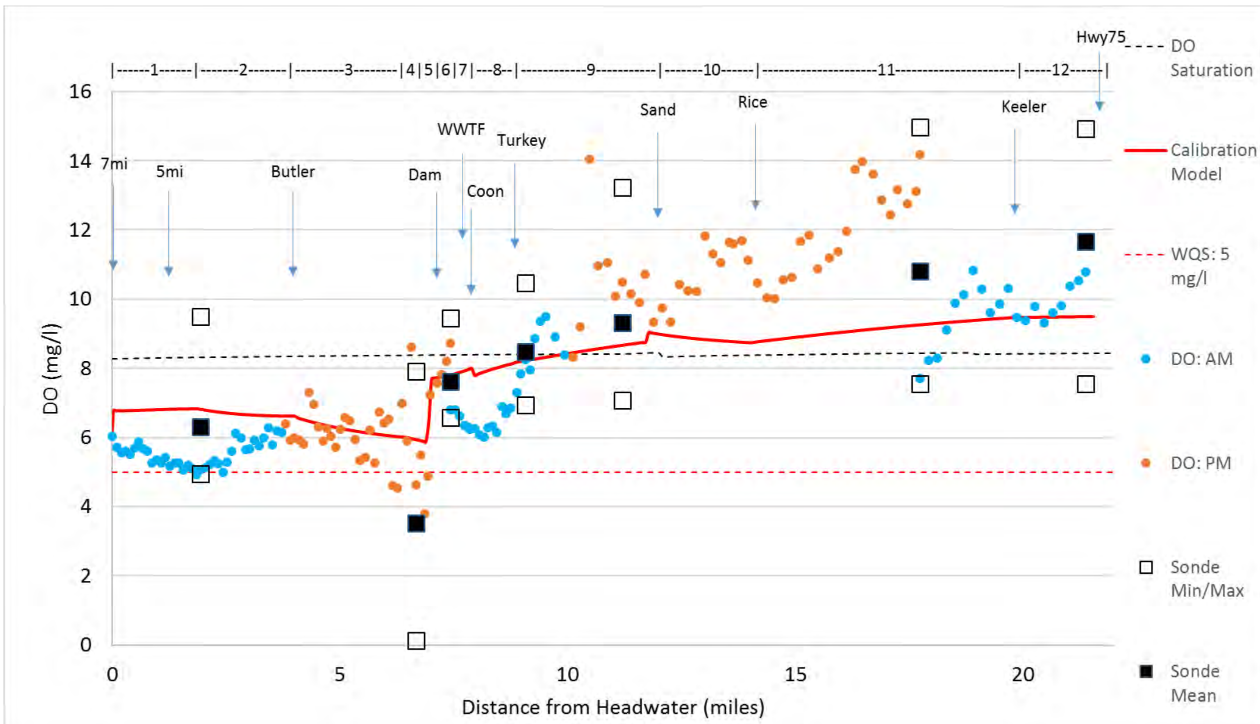

Figure 4-10. Calibration: simulation results and observed DO statistics from September 2017

As seen in Figure 4-10, the observed field data from September 2017 reveals that DO concentrations periodically dropped below the water quality standard of 5.0 mg/l DO upstream of the dam. The sluggish, deep, and partially stratified impounded water result in a system for which DO ranged along the surface from approximately 0 (anoxic) to 8 mg/l over the course of several days, with a daily mean concentration below 4 mg/l at the location of the sonde. It is important to note that although the average DO concentration observed at the dam sonde was 3.5 mg/l, synoptic sampling of DO at many points along that reach at the warmest time of day (around 3:00 PM on 9/7/2017, typically the time of day associated with the highest DO concentrations due to photosynthetic response to solar radiation) ranged from 3.8 – 8.6 mg/l, therefore the average observed by the sonde may not be representative of the entire reach. Based on the synoptic DO data across the reach, the DO sag was simulated above the dam to be greater than the DO standard, which is likely a better representation of the entire reach as opposed to the 3.5 mg/l mean DO observed at the sonde immediately upstream of the dam.

4.3 MODEL CORROBORATION RESULTS

The corroboration of the Caney River QUAL2K model was used to verify that model inputs developed and parameterized for the calibrated model would hold true while simulating median streamflow conditions on the order of 100 cfs (relative to the approximately 20 cfs flow conditions of the calibration model).

4.3.1 Flow Balance

As seen with the calibrated model, the corroborated model results provide reasonable approximation of TOT dye studies along simulated reaches (Table 4-10). Results for observed width, depth, and velocity are reasonably approximated by the corroboration model as well (Table 4-11).

Table 4-10. Corroboration model results compared to TOT studies over the same extents

Upstream Location	Downstream Location	Observed TOT (d)	Corroboration Model TOT (d)
1500 Rd	5 mi. US of intake	0.6	0.6
	Low-Head Dam	3.1	3.1
	Bartlesville WWTF	3.1	3.3
5 mi. US of intake	Bartlesville WWTF	3.1	2.7
DS of dam	Hillcrest Drive	0.8	0.8
	Adams Blvd	0.3	0.4
	2400 Rd	2.1	2.2
	Hwy 75	2.4	2.7
Adams Blvd	Hillcrest Drive	0.4	0.4
Hillcrest Rd	2400 Rd	1.3	1.4
	Hwy 75	2.1	2.0

Table 4-11. Corroboration model results compared to observed channel geometry and velocity

Reach	October Field-Observed Range			Corroboration Model Results		
	Width (ft)	Depth (ft)	Velocity (ft/s)	Width (ft)	Depth (ft)	Velocity (ft/s)
1	76	6.1	0.19	76	6.8	0.19
2	50 - 70	5 - 8	0.14	78	9.0	0.14
3	60 - 70	8	0.12	82	10.1	0.12
4	>70	N/A	0.10	104	9.1	0.10
5	30	N/A	N/A	98	10.4	0.09
6	50 - 60	1 - >8	0.20	60	7.8	0.19
7	50 - 75	2 - >8	0.43	97	2.5	0.41
8	50 - 80	1 - 8	0.49	95	2.3	0.48
9	60 - 90	1.5 - 9	0.33	93	3.4	0.33
10	60 - 80	2.5 - 8	0.30	71	5.6	0.30
11	40 - 90	1.5 - 8	0.30	99	4.0	0.30
12	58 - 100	0.9 - 9	0.38	93	3.2	0.40

4.3.2 Water Temperature

The observed water temperature during the October sampling period was cooler than the September period, and there was less of an observed range in daily water temperature due to the presence of clouds which dampened the diel cycle during the corroboration period. The simulation of water temperature during the corroboration model provided an excellent fit with the observed data, capturing the central tendency of the observed water temperature at the sondes around approximately 22.7°C (Figure 4-11). The full suite of water temperature data observed longitudinally and at the sondes may be referenced in Appendix B and Appendix C.

The relative error of simulated water temperature relative to observed average water temperature for the model extent is 2%.

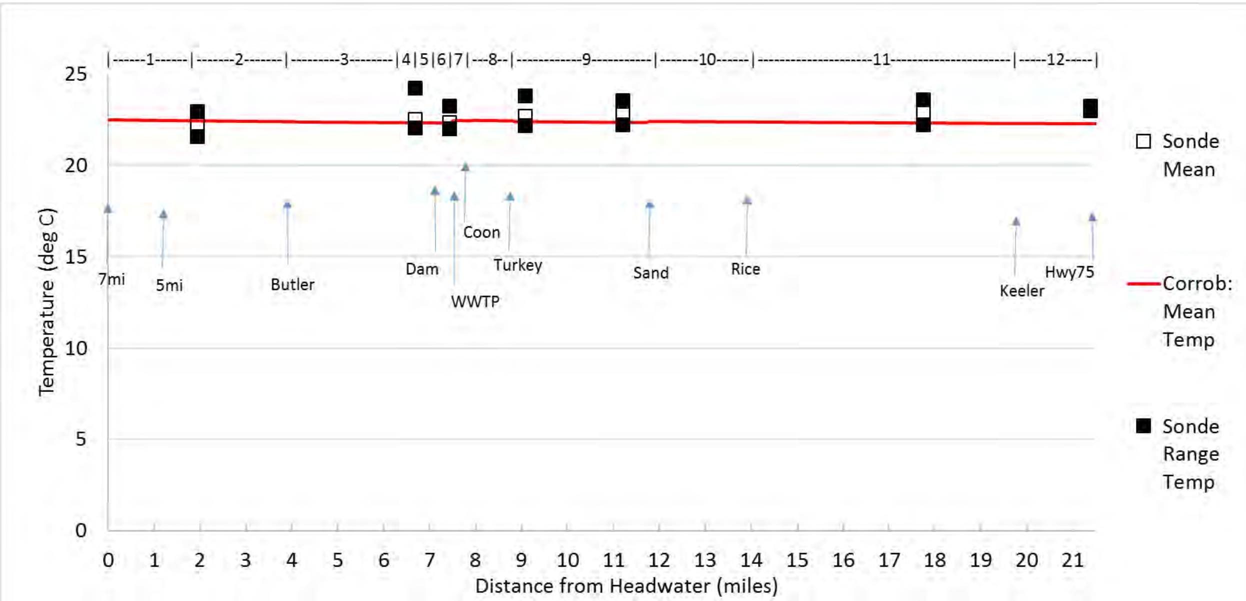

Figure 4-11. Corroboration: simulation results and observed water temperature from October 2017.

4.3.3 CBOD and Nitrogen

Water quality grab samples were sampled for corroborating field data on October 6, 2017. As mentioned previously, a precipitation event occurred on October 4, 2017. Streamflow had returned to pre-storm conditions by October 6, but there is reason to believe that the storm caused a flush of nutrients in the system, such that the water quality sampled on October 6 may not be representative of pre-storm conditions for some parameters. For example, September sampling revealed chlorophyll-*a* concentrations in the pooled area upstream of the dam to be 3.2 – 8.0 mg/m^3 during low flow conditions, and during October sampling, chlorophyll-*a* concentrations in the same location were observed as 29.9 – 494.0 mg/m^3 which suggest a significant algal response to the storm-induced nutrient pulse through the system. Overall results, however, supported QUAL2K simulation for corroboration purposes as shown below (Figure 4-12 and Figure 4-13). Full suite of grab sample data available in Appendix A, Section A.2.

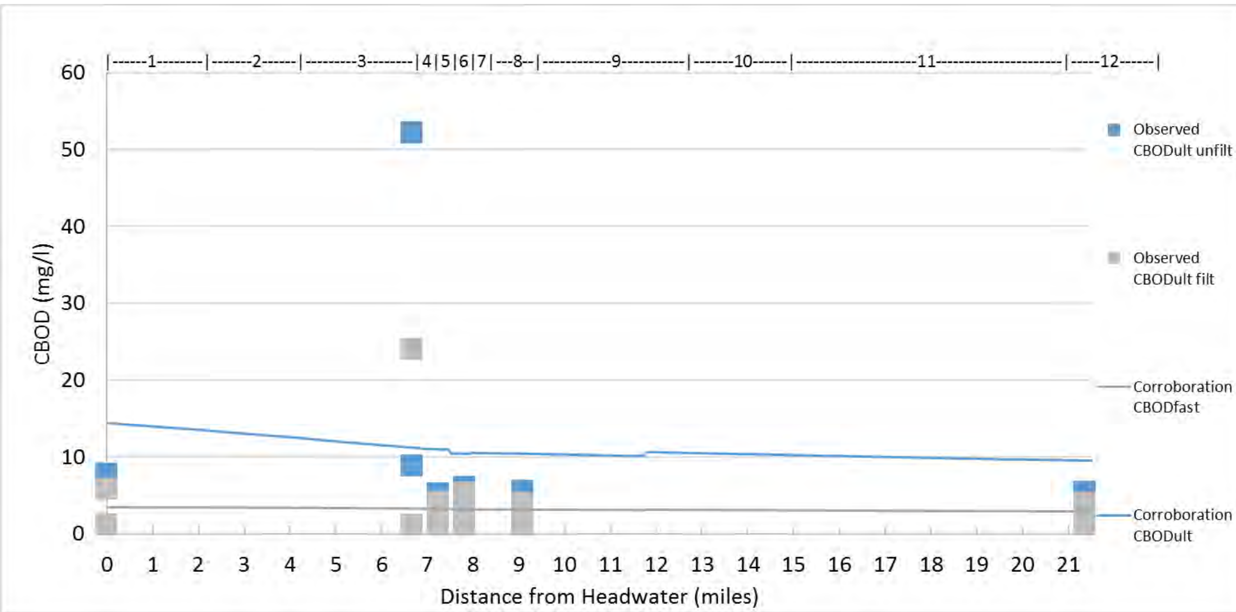

Figure 4-12. Corroboration: simulation results and observed oxygen-demanding species data

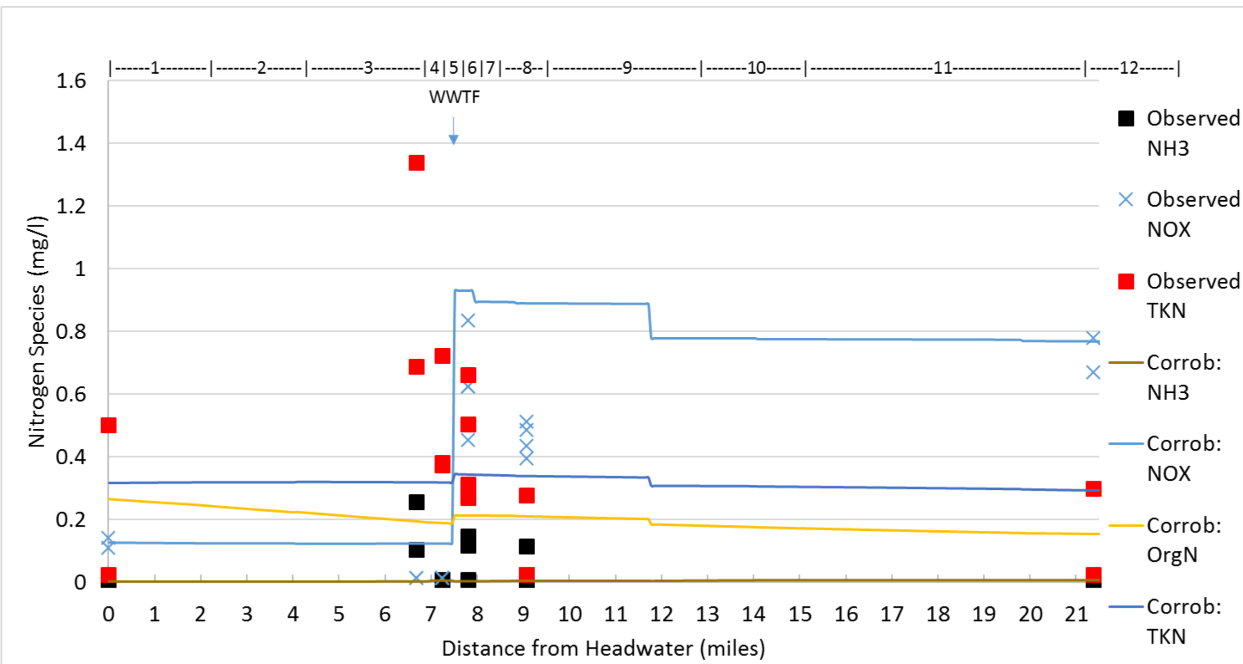

Figure 4-13. Corroboration: simulation results and observed nitrogen species data

4.3.4 pH

The corroboration model period does a great job in capturing the pH along the Caney River. As seen in the calibration model, the shape of the pH curve along the river tracks similarly with the DO curve due to the relationship between oxygen-producing and oxygen-consuming biota which impact both alkalinity and total inorganic carbon (the controls of pH). pH was observed and simulated to not reach super-saturation during the corroboration period and the simulated pH stayed roughly around an average of 7.9.

The relative error of simulated pH relative to observed average pH for the extent model is 5%.

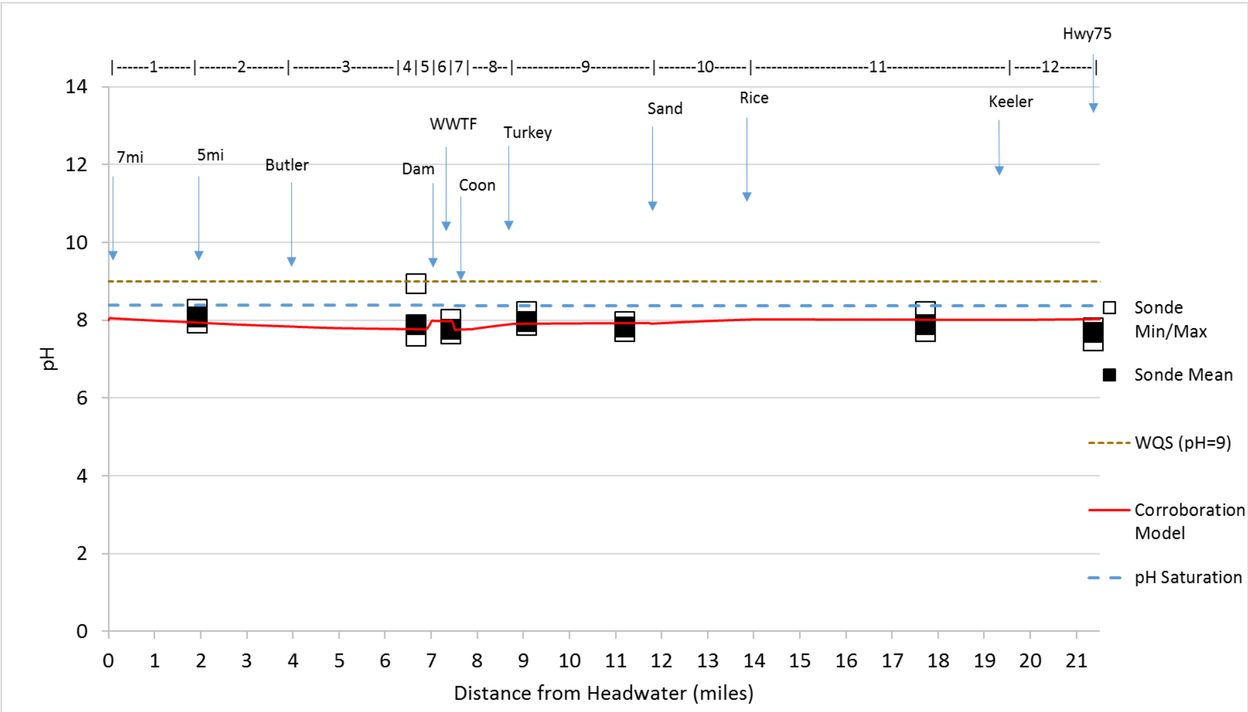

Figure 4-14. Corroboration: simulation results and observed pH statistics from October 2017

4.3.5 Dissolved Oxygen

Using the same model parameterization as the calibrated QUAL2K model, the corroboration model produces a reasonably accurate simulation of DO relative to the observed central tendency at the discrete sonde locations (Figure 4-15). The observed range in DO concentrations is very different during the October corroboration period due to a combination of higher flow and velocity conditions under which phytoplankton communities are less likely to establish and overcast conditions with less solar radiation available for photosynthesis. Average observed DO concentrations along the mainstem sondes ranged from 7.4 to 7.9 mg/l during the corroboration monitoring period, while simulation results ranged from 6.0 to 8.0 mg/l. All parameterization was held consistent with the calibration model for the corroboration model. Note that it is likely that phytoplankton growth parameters may have been different during the corroboration period and will vary between seasons, however all terms were held to the same values for consistency. Where the calibration simulation overestimated DO upstream of the dam relative to the sonde, the corroboration model underestimates DO upstream of the dam. The model parameterization reflects a reasonable optimization and compromise between the two very different observed DO conditions.

The relative error of simulated DO relative to observed average DO for the extent model is 7%.

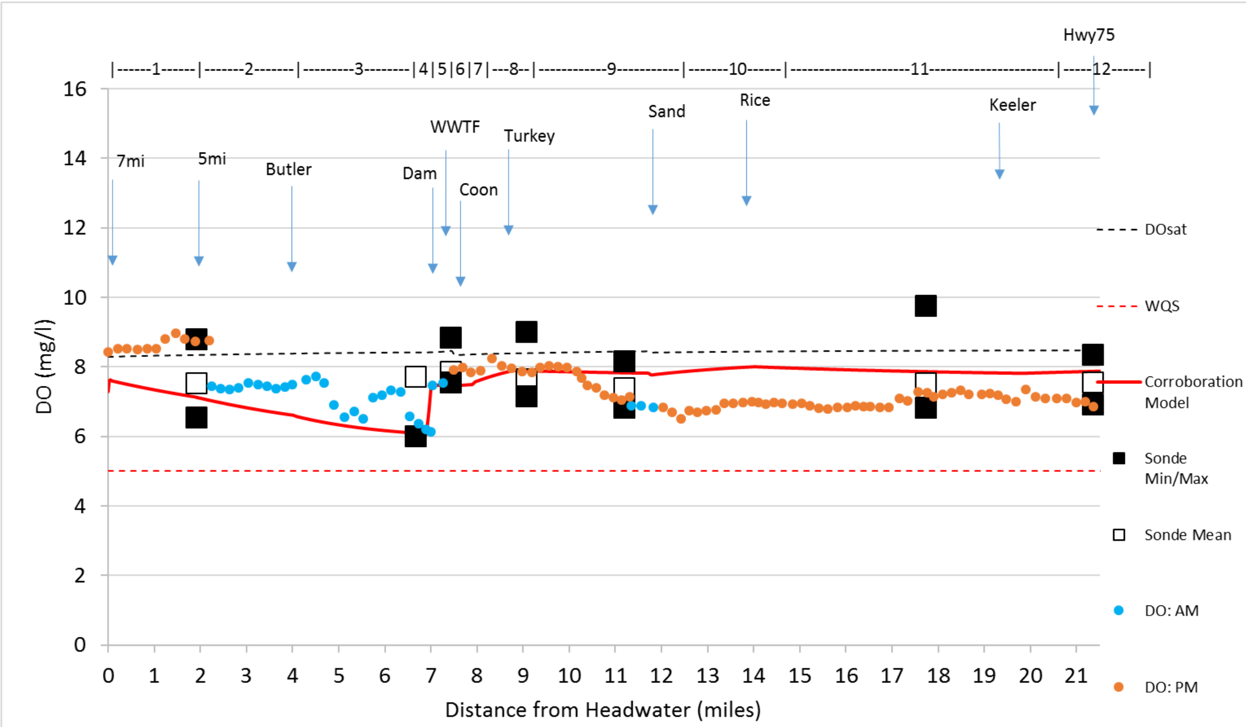

Figure 4-15. Corroboration: simulation results and observed DO statistics from October 2017.

5.0 QUAL2K MODEL SENSITIVITY

A series of sensitivity analyses were conducted to analyze the various impact of key model parameters on the simulation of DO. The model parameters tested were: wind speed, SOD rate, boundary conditions phytoplankton concentrations, boundary condition flows, and boundary condition DO concentrations (Table 5-1). Each parameter was adjusted by +25 percent and -25 percent relative to the calibration model for each sensitivity run, with all other parameters held to baseline conditions. Note that when a boundary condition was changed, the percent change to that parameter was applied to both headwaters and tributaries.

The sensitivity tests were used to compare the baseline calibrated model with each parameter input change individually to explore the impact on mean DO concentration along the mainstem. For each parameter change, the average DO concentration for the mainstem was summarized in Table 5-1 and as a tornado diagram to show the relative sensitivity of each parameter (Figure 5-1).

Of the parameters tested here, the average mainstem DO concentration was most sensitive to boundary flows, boundary DO concentrations, and SOD. Mainstem DO was least sensitive to boundary Chl-*a* concentrations and wind speed comparatively. Increasing flow resulted in a decrease in mean DO due to the increase of CBOD load to the stream. Increasing SOD resulted in a decrease in mean DO because the excursion of oxygen demand depletes DO in the water column. Alternatively, increasing boundary DO concentrations increases the DO in the system in response. Note that these parameters tested do not necessarily represent the breadth of possible parameters which may impact DO concentrations, but they provide insight into the level of sensitivity of the model to a cross-section of relevant parameter inputs.

Table 5-1. Caney River QUAL2K model sensitivity test model runs

Model Run	Details	Results: Mean DO (mg/l)
Calibration	Observed summer conditions	8.06
Sensitivity 1	Wind Speed + 25%	8.15
	Wind Speed - 25%	7.99
Sensitivity 2	SOD Rate + 25%	7.95
	SOD Rate - 25%	8.17
Sensitivity 3	Boundary Chl-*a* + 25%	8.03
	Boundary Chl-*a* - 25	8.09
Sensitivity 4	Boundary flows + 25%	7.91
	Boundary flows - 25%	8.28
Sensitivity 5	Boundary DO + 25%	8.43
	Boundary DO - 25%	7.69

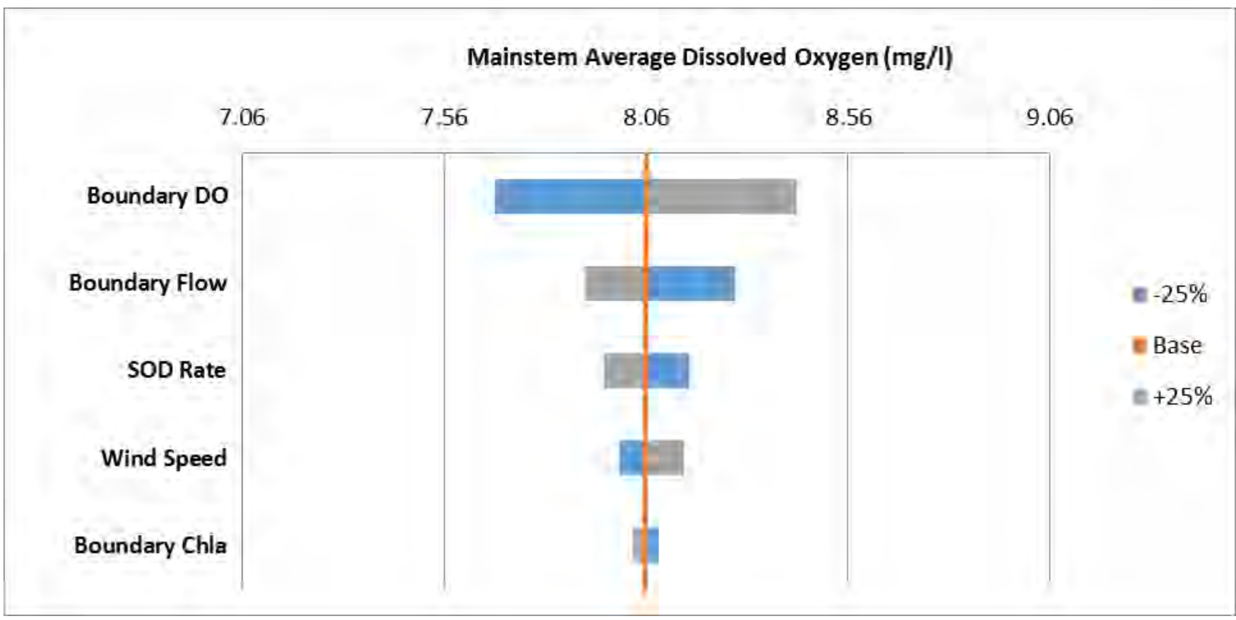

Figure 5-1. Tornado diagram of mean DO results from sensitivity relative to baseline

6.0 QUAL2K MODEL APPLICATION

6.1 CRITICAL CONDITIONS

According to ODEQ regulation, waste load allocation (WLA) scenario applications for DO are simulated under conditions of critical low flow (7Q2), and critical water temperatures which vary by season for WWAC (ODEQ, 2012). The ODEQ criteria for these seasonal simulations are:

- Summer (June 16 to October 15)
  - DO minimum WQS is 5.0 mg/l
  - Water temperature of 30°C (temperature determined as 90th percentile of observations from previous studies, Tetra Tech 2011)
- Winter (October 16 to March 31)
  - DO minimum WQS is 5.0 mg/l
  - Water temperature of 18°C
- Spring (April 1 to June 15)
  - DO minimum WQS is 6.0 mg/l
  - Water temperature of 25°C

These conservative conditions of warm waters and low flows aim to protect aquatic communities under even extreme environmental circumstances. To run WLA scenarios to assess the assimilative capacity of Caney River, the calibration model was adjusted to represent these conditions. Lowest flows tend to occur during the month of September while the highest air temperatures tend to occur during the month of July. Although these conditions are likely not to coincide, they represent the most critical conditions for the WLA scenarios.

The calibration model was setup during a period that is meant to mimic summer 7Q2 critically low flow conditions. With coordination with the USACE to control reservoir releases, flows were observed to be averaging 24.0 cfs for the calibration model, which is near the 7Q2 flow of 20.2 cfs. Flows were adjusted within the QUAL2K model environment for the headwaters and tributaries to match 7Q2 flow conditions. By dividing the 7Q2 flow by the drainage area of the USGS gage, flows were adjusted accordingly for all boundary conditions (Table 6-1). 7Q2 flows for spring and winter conditions based on the applicable dates for the flow period of record, and the results were 58.6 cfs for spring, and 17.2 cfs for winter.

Water temperatures of each boundary condition flow was modified to the assumed seasonal temperatures to ensure instream conditions would be approximately equal to the ODEQ critical water temperature. Meteorological forces were modified from the calibration period to ensure that water temperatures for each critical condition scenario were maintained at the critical seasonal water temperatures. Boundary conditions for DO concentrations for the headwaters and the tributaries were developed based on percent saturation relative to the seasonal temperatures. During the QUAL2E analyses, WLA scenarios assumed 6.50 mg/l DO at 30°C for headwater conditions, which is representative of 86% of DO saturation (7.56 mg/l) at that temperature (Tetra Tech, 2011; Rounds, et al., 2013). DO saturation for spring and winter critical conditions are 8.26 and 9.46 mg/l respectively under standard pressure, therefore 86% saturation yields a boundary condition DO concentration assumption of 7.1 mg/l for spring, and 8.1 mg/l for winter (Rounds, et al., 2013).

Field sampling conducted during the previous QUAL2E modeling analysis occurred during conditions in which the Caney River was naturally flowing near 7Q2 flows, as opposed to the 2017 sampling which occurred under artificial discharge conditions thanks to reservoir releases conducted by USACE. Field measurements of BOD constituents from that period along Caney River above the Coon Creek confluence were measured below detection limit at all times (Tetra Tech, 2003; 2004; 2011). When concentrations are below detection, modeling assumptions generally call for model input of half the detection limit of 2 mg/l. The boundary condition concentrations for fastCBOD in QUAL2K were set to 2.0 mg/l for all critical seasonal simulations which is a more conservative assumption. Note that this assumption is not very different from the model inputs for the calibration and corroboration period for which fastCBOD inputs were 2.06 and 3.44 mg/l respectively.

For the baseline critical scenarios, the Bartlesville WWTF outfall is removed as a point source to the system such that the baseline reference scenario represents the most critical natural condition. Note that the summer critical simulations were run for the simulation date of the calibration model (9/8/2017), while the spring and winter models were run for representative seasonal dates of 5/1/2017 and 1/1/2017 respectively.

Table 6-1. Critical condition boundary conditions for WLA scenarios

Boundary	Summer Critical Conditions			Spring Critical Conditions			Winter Critical Conditions		
	Flow (cfs)	Temp (°C)	DO (mg/l)	Flow (cfs)	Temp (°C)	DO (mg/l)	Flow (cfs)	Temp (°C)	DO (mg/l)
Headwater	19.8	30	6.5	57.5	25	7.1	16.9	18	8.1
Butler Creek	0.4	30	6.5	1.1	25	7.1	0.3	18	8.1
Coon Creek	1.0	30	6.5	2.9	25	7.1	0.8	18	8.1
Turkey Creek	0.1	30	6.5	0.3	25	7.1	0.1	18	8.1
Sand Creek	3.8	30	6.5	10.9	25	7.1	3.2	18	8.1
Rice Creek	0.1	30	6.5	0.3	25	7.1	0.1	18	8.1
Keeler Creek	0.2	30	6.5	0.5	25	7.1	0.1	18	8.1

6.2 WASTELOAD ALLOCATION SCENARIOS

To examine the assimilative capacity of the Caney River to handle increased wasteflow from the Bartlesville WWTF, a number of simulations were conducted with variable flows, outfall locations, and permitted nutrient concentrations. The existing Bartlesville WWTF NPDES permit limits effluent flow to 7 MGD, with maximum effluent concentrations of BOD_5 at 10 mg/l, and NH_3 at 2 mg/l. There is no permitted limit associated with minimum DO concentration in the effluent currently. Year-round permit limits for the upgraded Bartlesville WWTF are anticipated to be a maximum flow of 8.206 MGD based on updated population projections, BOD_5 of 10 mg/l, and NH_3 of 1 mg/l based on upgraded plant processes. There is likely to be a new DO permit limit for the plant of at least 6 mg/l, a standard which the plant likely already meets, which also is applicable year-round[1]. WLA scenarios consider the possibility of a portion of future effluent being discharged seven miles upstream of the existing water intake on the Caney River, as well as maintaining the existing outfall location at the WWTF, all under the anticipated permit limits of BOD_5/NH_3/DO at concentrations of 10, 1, and 6 mg/l respectively year-round. The location 7 miles upstream of the existing water intake is defined as immediately downstream of the 1500 Road crossing (also known as 9th Street) on the Caney River (Figure 6-1.).

[1] Discharge Monitoring Report (DMR) data for Bartlesville WWTF report DO concentrations on the order of 2–5 mg/l. These measurements are taken at the plant and not at the discharge point. Tetra Tech field measurements at the discharge location were ~7 mg/l on the days reported by DMR to be ~3 mg/l, so it is apparent that DMR-report DO concentrations are far lower than actual DO post-cascade aeration.

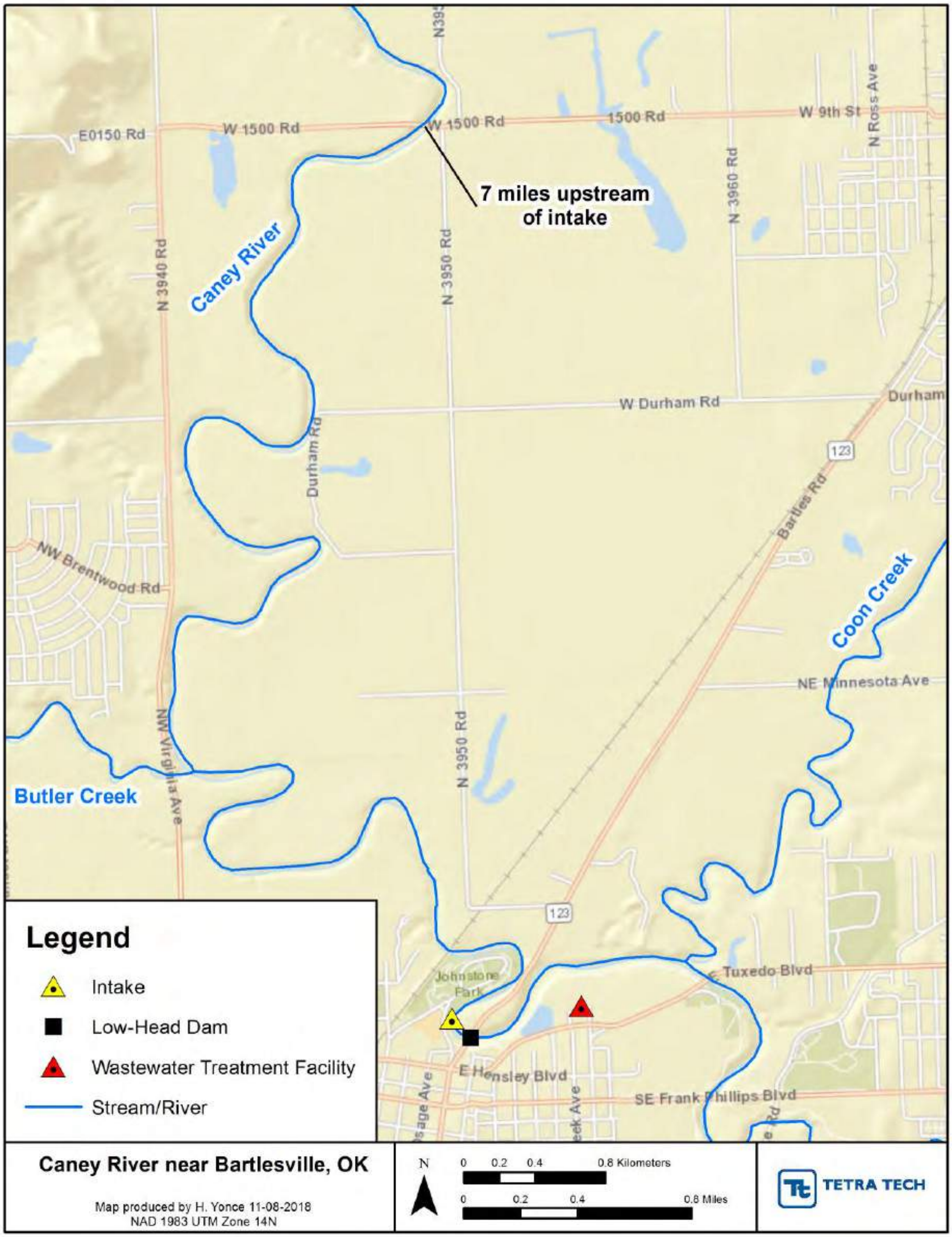

Figure 6-1. Location seven miles upstream of the existing water intake on Caney River

The fast$CBOD_{ult}$ model input for WLA scenarios must be estimated as a function of the permit limit as measured and reported as BOD_5. Grab samples from Site 5 during both sampling periods were used to develop a translator from BOD_5 to dissolved $CBOD_{20}$, which can then be divided by 0.9 as previously documented to estimate a model input of $CBOD_{ult}$. Samples below detection were set to half the detection limit and were also weighted far less than samples which were above detection. Sample data, calculated ratios, and weighting factors are presented in Table 6-2. The translator ratio was calculated as 2.02, therefore the existing NPDES permit limit of 10 mg/l BOD_5 is analogous to 4.94 mg/l $CBOD_{20}$ and a model input of 5.49 mg/l $CBOD_{ult}$.

Table 6-2. Site 5 grab sample water quality data for dissolved $CBOD_{20}$ and BOD_5

Sample Date and Time	Dissolved $CBOD_{20}$ (mg/l)	BOD_5 (mg/l)	BOD_5:dslv$CBOD_{20}$ Ratio	Weight Factor
9/7/17 8:56	2.1	5.5	2.62	0.20
9/7/17 10:48	Below Detection	3.3	3.30	0.05
9/7/17 14:31	2.1	3.6	1.71	0.20
9/7/17 15:41	Below Detection	3.0	3.00	0.05
10/6/17 8:26	Below Detection	3.0	3.10	0.05
10/6/17 10:01	Below Detection	4.4	4.40	0.05
10/6/17 14:52	3.8	3.8	1.00	0.20
10/6/17 16:26	4.8	6.4	1.33	0.20
Weighted Average BOD_5:dslv$CBOD_{20}$ Translator Ratio: 2.0233				

The summer WLA scenario is based on conditions from the calibration period, while the spring and winter WLA scenarios are based on conditions from the corroboration period. Aside from the WLA seasonally-specific changes to meteorological inputs, boundary flow, water temperature, DO concentrations, and fastCBOD concentrations that reflect seasonal critical conditions, all other model parameterization was held consistent with the calibration and corroboration model setups respectively, particularly as it applies to headwater and tributary water quality conditions. It is likely that some parameters related to phytoplankton kinetics in particular would vary by season, but at this time all terms are held consistent with those employed during the calibration and corroboration periods.

The suite of scenarios conducted for WLA consideration are shown in Table 6-3. Scenarios include discharging the entire projected effluent volume at the existing location and splitting the effluent 50/50 between the existing location and a point seven miles upstream of the water intake site. These two scenarios were duplicated for critical conditions during summer, spring, and winter.

Table 6-3. Caney River model application descriptions summarized

Run	Description of Scenario
Summer Critical Condition	Baseline calibration model modified to summer critical low flows and warm water temperatures
Summer Scenario 1	8.206 MGD effluent released at existing discharge outfall
Summer Scenario 2	4.103 MGD effluent released at both existing outfall and 7 miles upstream of intake

Run	Description of Scenario
Spring Critical Condition	Baseline corroboration model modified to spring critical low flows and warm water temperatures
Spring Scenario 1	8.206 MGD effluent released at existing discharge outfall
Spring Scenario 2	4.103 MGD effluent released at both existing outfall and 7 miles upstream of intake
Winter Critical Condition	Baseline corroboration model modified to winter critical low flows and warm water temperatures
Winter Scenario 1	8.206 MGD effluent released at existing discharge outfall
Winter Scenario 2	4.103 MGD effluent released at both existing outfall and 7 miles upstream of intake

To examine the impact of each WLA scenario, the minimum daily average DO concentration (sag) upstream of the dam and downstream of the existing outfall location is reported in Table 6-4. Note that the seasonal WQS for summer, spring, and winter are 5.0, 6.0, and 5.0 mg/l respectively. A margin of safety (MOS) of 5 percent applied to each concentration results in 5.25, 6.30, and 5.25 mg/l DO respectively.

Table 6-4. Caney River WLA scenario inputs and results

Run	Outfall Location	Outfall Flow (MGD)	Minimum DO (mg/l)	Maximum pH
Calibration Model	Existing	Observed (5.66)	5.9	8.7
Corroboration Model	Existing	Observed (5.83)	6.0	8.1
Summer Critical	None	N/A	5.4	8.8
Summer Scenario 1	Existing	8.206	5.4	8.9
Summer Scenario 2	7 miles / Existing	4.103 / 4.103	5.4	8.9
Spring Critical	None	N/A	5.2	8.2
Spring Scenario 1	Existing	8.206	5.2	8.1
Spring Scenario 2	7 miles / Existing	4.103 / 4.103	5.1	8.0
Winter Critical	None	N/A	6.1	8.6
Winter Scenario 1	Existing	8.206	6.1	8.3
Winter Scenario 2	7 miles / Existing	4.103 / 4.103	5.8	8.5

As shown in Table 6-4, although observed conditions from the summer 2017 calibration model were below the WQS of 5.0 mg/l DO upstream of the low-head dam, it is anticipated that more naturally-occurring summer critical conditions (i.e., extended natural periods of low flow rather than shorter periods induced by USACE upstream dam flow reduction) produce a state in which the Caney River does meet the WQS. The WQS is met under all seasonal critical condition simulations along the entire modeled extent. The longitudinal results for each WLA run are shown by season for summer (Figure 6-2), winter (Figure 6-3), and spring (Figure 6-4) relative to the respective seasonal WQS with applied MOS.

As shown Figure 6-2, the critical summer baseline condition with no WWTF discharge present meets the WQS + MOS along the entire extent. The addition of the 8.206 MGD discharge at the existing outfall location produces a brief sag in DO which recovers quickly due in part to phytoplankton DO production. The addition of 4.103 MGD of effluent at seven miles upstream of the dam along with 4.103 MGD of effluent at the existing location results in a sag upstream of the dam which is not worse than the summer critical condition, and the WQS + MOS is still met along the entire model extent.

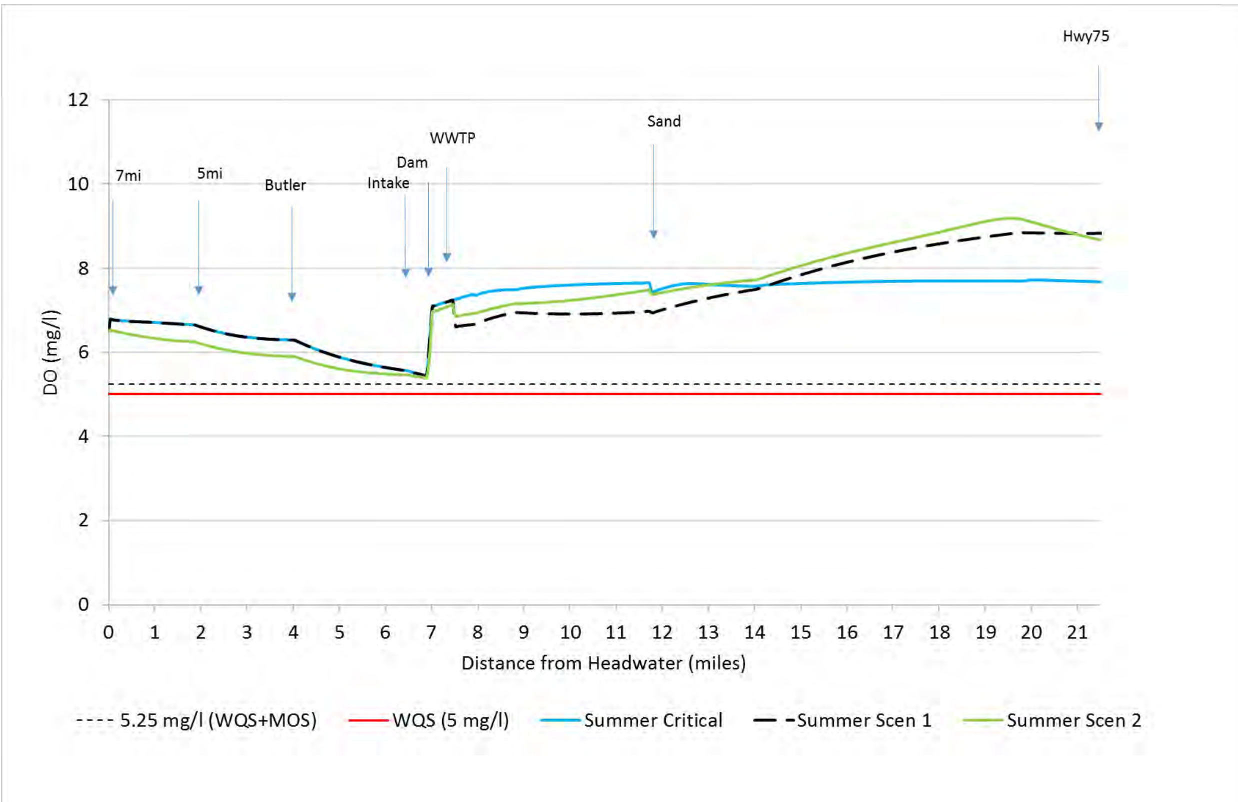

Figure 6-2. Summer WLA DO results: summer scenarios 1 and 2 relative to summer critical conditions

As shown in Figure 6-3, the critical winter baseline condition with no WWTF discharge present meets the WQS + MOS along the entire extent. Similar to the summer conditions, all scenarios meet the WQS + MOS for winter.

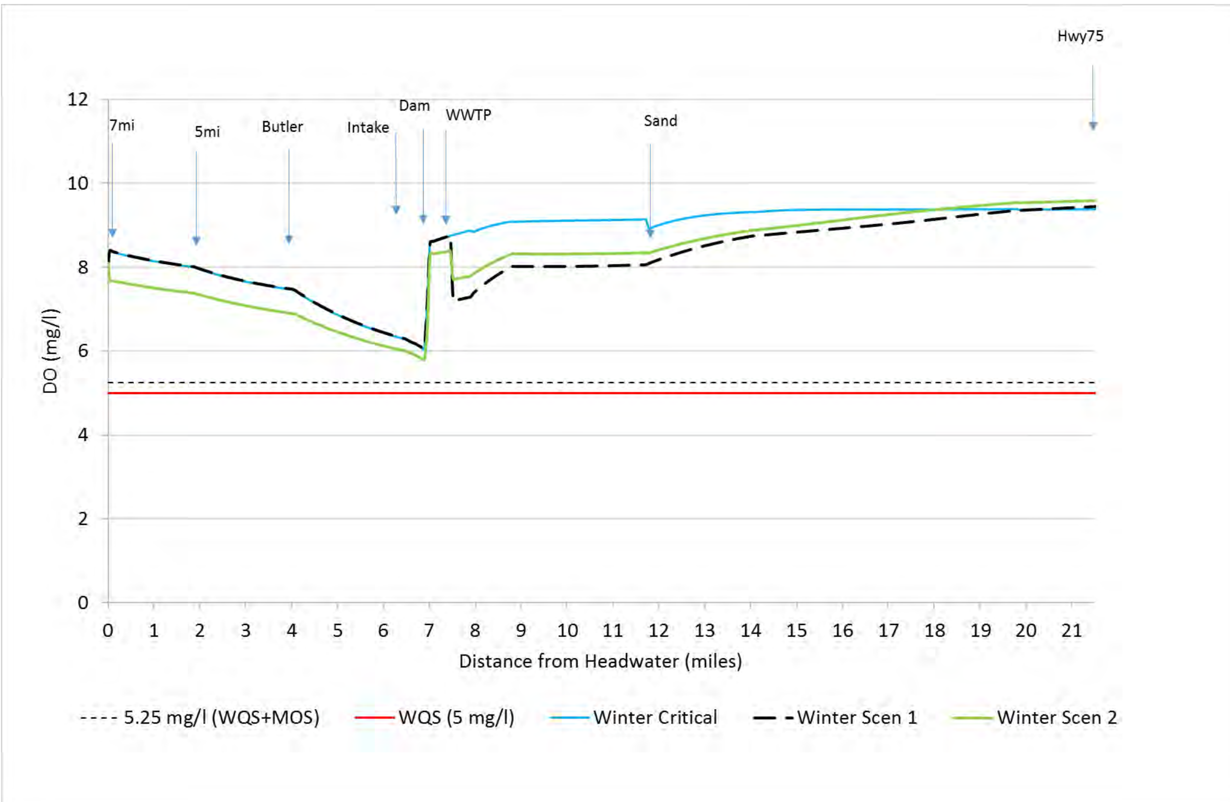

Figure 6-3. Winter WLA DO results: winter scenarios 1 and 2 relative to winter critical conditions

As shown Figure 6-4, the critical spring baseline condition with no WWTF discharge present does not meet the WQS + MOS upstream of the dam. This is due in part to the algal and photosynthesis parameterization which were developed based on observations during the summer and may not be reflective of existing critical spring conditions. Due to the more stringent WQS in the spring, the critical conditions do not allow for assimilative capacity of effluent to be released upstream of the dam (Scenario 2), however there is assimilative capacity for the full effluent flow rate to be released from the existing outfall location (Scenario 1).

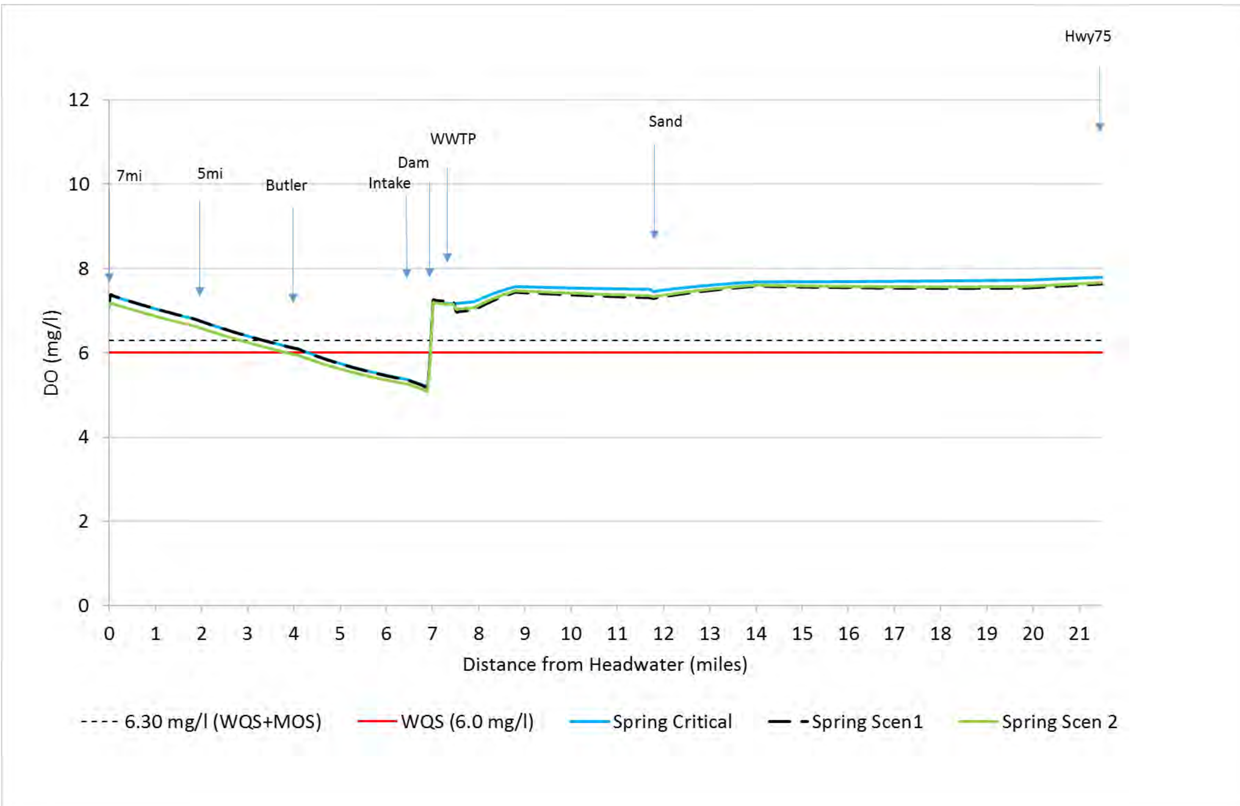

Figure 6-4. Spring WLA DO results: spring scenarios 1 and 2 relative to spring critical conditions

7.0 CONCLUSIONS

In conclusion, the calibrated and corroborated QUAL2K model demonstrates that there is assimilative capacity along the Caney River model extent to support expanded discharge at the existing outfall during all seasons. The model predicts that assimilative capacity along the Caney River can support the split discharge 50-50 between the existing outfall and a new outfall located seven miles above the existing water supply intake location on the Caney River meeting the WQS during the summer and winter periods, but not during the spring period when the WQS is more stringent. Expanding discharge at the existing outfall site to 8.2 MGD does not appear to negatively impact mean daily DO concentrations downstream with respect to the mean daily minimum DO during all seasons. There is no observed DO data available for the spring period along this section of the Caney River, so there is some uncertainly associated with the spring prediction in the absence of corroborating data. The simulation may be overly conservative during the spring period. Additional monitoring can therefore be considered for further assessing whether an upstream discharge during the spring would also be assimilated.

8.0 REFERENCES

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APPENDIX A: GRAB SAMPLE FIELD DATA

All grab sample and field probe results at the 10 measured field sites are included within this appendix (Figure 4-2). Sampling frequency varied by site, as seen in Table A-1. Sample counts per day by parameter and site for both field trips 1 and 2, per the approved monitoring plan (Tetra Tech, 2017).

Table A-1. Sample counts per day by parameter and site for both field trips 1 and 2

Site	Location	Temperature, Conductivity, DO Sat, DO, pH	NOx, NH_3, TKN	PO_4, TP	BOD_5, BOD_{20}, $CBOD_{20}$-dslv, $CBOD_5$-dslv	$CBOD_5$, $CBOD_{20}$	TOC, TDS, TSS, CHL-A
1	7 miles US of intake	2	2	2	2	2	2
2	Butler Creek	2	2	2	2	2	0
3	Dam Pool	2	2	2	2	2	2
4	DS of Dam	4	4	2	4	2	0
5	WWTF effluent	4	4	2	4	2	0
6	DS of outfall	4	4	2	4	2	2
7	Coon Creek	4	4	2	4	2	2
8	DS of Turkey Creek	4	4	2	4	2	2
9	Sand Creek	2	2	2	2	2	0
10	HWY75	2	2	2	2	2	0

A.1 Field Sample Results

Basic water quality parameters of water temperature (TEMP), conductivity (COND), pH (pH), and dissolved oxygen concentration and saturation (DO, DOSAT) were sampled in the field at the collection time of the grab samples (Figure 4-2). Results from these field measurements are included below (Table A-2. Field basic water quality sample results, 2017). Sample IDs reported in Table A-2. Field basic water quality sample results, 2017 reflect two pieces of information: the first digit represents the grab sample site ID corresponding to locations in Figure 4-2, while the second number reflects which sampling period per day the measurement was taken (i.e. Sample ID “3-1” is the first sample of the day recorded at grab sample site 3).

Table A-2. Field basic water quality sample results, 2017

Sample ID	Time	Date	Temp (°C)	Cond (mS/cm^3)	DO SAT (%)	DO (mg/l)	pH
8-1	8:36 AM	9/7/2017	22.55	0.362	84.7	7.28	7.68
8-1	8:40 AM	9/7/2017	22.64	0.361	82.8	7.14	7.55
2-1	9:06 AM	9/7/2017	22.03	0.345	53.6	4.66	7.58
3-1	9:50 AM	9/7/2017	24.21	0.314	36.9	3.08	7.63
4-2	10:17 AM	9/7/2017	24.23	0.312	92.9	7.76	7.86
5-2	10:48 AM	9/7/2017	25.03	0.307	108.5	8.95	7.88
6-2	10:56 AM	9/7/2017	24.73	0.309	80	6.64	7.92
7-1	11:36 AM	9/7/2017	21.22	0.49	58.8	5.20	7.95

Sample ID	Time	Date	Temp (°C)	Cond (mS/cm^3)	DO SAT (%)	DO (mg/l)	pH
7-3	1:43 PM	9/7/2017	22.38	0.507	70.5	6.09	8.06
8-3	2:12 PM	9/7/2017	25.33	0.361	114	9.37	8.27
2-3	2:36 PM	9/7/2017	22.01	0.352	48.7	4.22	7.98
3-3	3:04 PM	9/7/2017	26.7	0.322	86	6.86	7.67
4-4	3:33 PM	9/7/2017	26.46	0.31	107.8	8.67	8.06
5-4	3:41 AM	9/7/2017	26.36	0.31	107.2	8.62	8.01
6-4	3:47 AM	9/7/2017	26.59	0.311	102	8.2	8.09
1-1	7:51 AM	9/7/2017	23.7	0.395	72.7	6.15	7.58
4-1	8:29 AM	9/7/2017	23.73	0.312	88.3	7.47	7.69
5-1	9:00 AM	9/7/2017	24.58	0.48	90.8	7.54	7.58
6-1	9:20 AM	9/7/2017	23.4	0.363	81.5	6.93	7.17
7-2	10:15 AM	9/7/2017	20.87	0.879	68.7	6.1	7.47
8-2	10:42 AM	9/7/2017	23.97	0.36	94.5	7.95	8.04
9-1	11:18 AM	9/7/2017	21.82	0.348	66.5	5.8	7.41
10-1	11:58 AM	9/7/2017	23.13	0.387	119.1	10.18	8.75
1-3	1:47 PM	9/7/2017	24.75	0.401	86.9	7.2	7.81
4-3	2:20 PM	9/7/2017	26.54	0.31	106.5	8.54	8.21
5-3	2:36 PM	9/7/2017	26.44	0.611	97.9	7.87	7.35
6-3	2:52 PM	9/7/2017	26.87	0.372	114.2	9.08	8.09
7-4	3:47 PM	9/7/2017	22.84	0.887	87.1	7.51	7.64
8-4	4:12 PM	9/7/2017	25.62	0.363	122.1	9.96	8.3
9-3	4:53 PM	9/7/2017	22.88	0.351	74.3	6.21	7.31
10-3	5:20 PM	9/7/2017	24.54	0.386	163.8	13.62	8.9
7-1	8:00 AM	10/6/2017	21.93	0.595	65	5.71	7.42
8-1	8:35 AM	10/6/2017	22.38	0.371	78.2	6.78	7.51
2-1	8:57 AM	10/6/2017	22.52	0.343	60.6	5.23	7.58
3-1	9:15 AM	10/6/2017	22.49	0.329	81.7	7.07	7.63
4-2	9:53 AM	10/6/2017	22.45	0.331	91.9	7.96	7.75
5-2	10:01 AM	10/6/2017	22.46	0.334	91	7.88	7.79
6-2	10:10 AM	10/6/2017	24.78	0.585	82.8	6.84	7.46
7-3	1:38 PM	10/6/2017	23.97	0.589	100.8	8.46	7.6
8-3	2:04 PM	10/6/2017	23.04	0.361	92.9	7.95	7.62
2-3	2:27 PM	10/6/2017	23.55	0.334	87.8	7.6	7.64
3-3	3:20 PM	10/6/2017	23.97	0.317	176	14.65	8.43
4-4	4:15 PM	10/6/2017	22.68	0.329	90	7.76	7.59
5-4	4:26 PM	10/6/2017	22.74	0.33	89	7.67	6.99
6-4	4:38 PM	10/6/2017	25.17	0.619	80.9	6.62	6.77
1-1	7:40 AM	10/6/2017	22.62	0.318	80.4	6.93	7.93
4-1	8:16 AM	10/6/2017	22.26	0.328	97.6	8.49	7.76
5-1	8:26 AM	10/6/2017	24.7	0.581	82.6	7.06	7.38
6-1	8:51 AM	10/6/2017	22.38	0.348	92.6	8	7.76

Sample ID	Time	Date	Temp (°C)	Cond (mS/cm^3)	DO SAT (%)	DO (mg/l)	pH
7-2	9:42 AM	10/6/2017	22.42	0.678	66.9	5.8	7.63
8-2	10:04 AM	10/6/2017	22.34	0.361	92.9	8.06	7.72
9-1	10:39 AM	10/6/2017	22.42	0.533	73.5	6.37	7.5
10-1	11:09 AM	10/6/2017	22.64	0.348	86.2	7.43	7.53
10-1	11:13 AM	10/6/2017	22.63	0.343	84.5	7.29	7.49
10-1	11:15 AM	10/6/2017	22.66	0.353	83.1	7.17	7.49
1-3	1:31 PM	10/6/2017	22.89	0.318	82.8	7.11	7.24
4-3	2:48 PM	10/6/2017	22.92	0.326	109.3	9.35	7.82
5-3	2:52 PM	10/6/2017	25.3	0.598	87.4	7.17	7.56
6-3	3:06 PM	10/6/2017	23.44	0.355	109.6	9.27	7.94
7-4	3:47 PM	10/6/2017	23.65	0.678	101.7	8.54	7.86
8-4	4:08 PM	10/6/2017	23.31	0.358	115.2	9.8	7.98
9-3	4:35 PM	10/6/2017	23.49	0.54	94.2	7.99	7.69
10-3	5:03 PM	10/6/2017	23.6	0.363	96.3	8.16	7.82
10-3	5:05 PM	10/6/2017	23.58	0.352	97.6	8.26	7.71
10-3	5:07 PM	10/6/2017	23.57	0.376	96	8.13	7.69

A.2 Grab Sample Results

The results for all grab samples from the September and October sampling trips as-reported by Accurate Labs (Table A-3. Grab sample results from September sampling trip and Table A-4. Grab sample results from October sampling trip, and see Figure 4-2). Parameter abbreviations included in these tables are: total organic carbon (TOC), nitrate and nitrite (NOx), ammonia (NH_3), total Kjeldahl nitrogen (TKN), phosphate (PO_4), total phosphorus (TP), total dissolved solids (TDS), total suspended solids (TSS), chlorophyll-*a* (CHL-A), pH (pH), water temperature (TEMP), 5-day biochemical oxygen demand (BOD_5), 20-day biochemical oxygen demand (BOD_{20}), total and dissolved 5-day carbonaceous biochemical oxygen demand ($CBOD_5$, $CBOD_5$-dslv), and total and dissolved 20-day carbonaceous biochemical oxygen demand ($CBOD_{20}$, $CBOD_{20}$-dslv). Each "Sample ID" is comprised of the grab sample location (first value) and identifying which sample of the day as each parameter was sampled at each site between two and four times per day (second value).

Note that for the October sampling period, samples 5-2 and 5-4 were incidentally mislabeled as 6-2 and 6-4 respectively (error identified by matching water temperatures with field notes). The information in the table reflects the corrected site assignments.

Table A-3. Grab sample results from September sampling trip

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
1-1	09/07/2017 07:41:00	TOC	6.05	mg/L	0.082	0.25
1-1	09/07/2017 07:41:00	NOx	<0.050	mg/L	0.025	0.05
1-1	09/07/2017 07:41:00	NH_3	0.134	mg/L	0.017	0.1
1-1	09/07/2017 07:41:00	TKN	0.26	mg/L	0.05	0.25
1-1	09/07/2017 07:41:00	PO_4	0.029	mg/L	0.009	0.025
1-1	09/07/2017 07:41:00	TP	0.116	mg/L	0.005	0.025

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
1-1	09/07/2017 07:41:00	TDS	244	mg/L	10	25
1-1	09/07/2017 07:41:00	TSS	14	mg/L	0.5	6.25
1-1	09/07/2017 07:41:00	pH	7.58	pH Units	0	0.01
1-1	09/07/2017 07:41:00	BOD_{20}	13.3	mg/L	0.6	2
1-1	09/07/2017 07:41:00	$CBOD_{20}$	6.3	mg/L	0.6	2
1-1	09/07/2017 07:41:00	$CBOD_{20}$-dslv	2.7	mg/L	0.6	2
1-1	09/07/2017 07:41:00	$CBOD_5$	6.2	mg/L	0.6	2
1-1	09/07/2017 07:41:00	$CBOD_5$-dslv	2.7	mg/L	0.6	2
1-1	09/07/2017 07:41:00	CHL-A	12.3	mg/m^3	0.25	0.5
1-1	09/07/2017 07:41:00	TEMP	23.7	°C	-40	-30
1-1	09/07/2017 07:41:00	BOD_5	8.1	mg/L	0.6	2
2-1	09/07/2017 09:06:00	NOx	<0.050	mg/L	0.025	0.05
2-1	09/07/2017 09:06:00	NH_3	0.128	mg/L	0.017	0.1
2-1	09/07/2017 09:06:00	TKN	<0.250	mg/L	0.05	0.25
2-1	09/07/2017 09:06:00	PO_4	<0.025	mg/L	0.009	0.025
2-1	09/07/2017 09:06:00	TP	0.094	mg/L	0.005	0.025
2-1	09/07/2017 09:06:00	pH	7.58	pH Units	0	0.01
2-1	09/07/2017 09:06:00	BOD_{20}	5.5	mg/L	0.6	2
2-1	09/07/2017 09:06:00	$CBOD_{20}$	2.8	mg/L	0.6	2
2-1	09/07/2017 09:06:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
2-1	09/07/2017 09:06:00	$CBOD_5$	2.8	mg/L	0.6	2
2-1	09/07/2017 09:06:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
2-1	09/07/2017 09:06:00	CHL-A	22	°C	-40	-30
2-1	09/07/2017 09:06:00	TEMP	3.6	mg/L	0.6	2
3-1	09/07/2017 09:50:00	TOC	0.797	mg/L	0.082	0.25
3-1	09/07/2017 09:50:00	NOx	<0.050	mg/L	0.025	0.05
3-1	09/07/2017 09:50:00	NH_3	<0.100	mg/L	0.017	0.1
3-1	09/07/2017 09:50:00	TKN	<0.250	mg/L	0.05	0.25
3-1	09/07/2017 09:50:00	PO_4	<0.025	mg/L	0.009	0.025
3-1	09/07/2017 09:50:00	TP	0.105	mg/L	0.005	0.025
3-1	09/07/2017 09:50:00	TDS	199	mg/L	10	25
3-1	09/07/2017 09:50:00	TSS	148	mg/L	1	12.5
3-1	09/07/2017 09:50:00	pH	7.63	pH Units	0	0.01
3-1	09/07/2017 09:50:00	BOD_{20}	5.4	mg/L	0.6	2
3-1	09/07/2017 09:50:00	$CBOD_{20}$	2.7	mg/L	0.6	2
3-1	09/07/2017 09:50:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
3-1	09/07/2017 09:50:00	$CBOD_5$	2.7	mg/L	0.6	2
3-1	09/07/2017 09:50:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
3-1	09/07/2017 09:50:00	CHL-A	3.2	mg/m^3	0.25	0.5
3-1	09/07/2017 09:50:00	TEMP	24.2	°C	-40	-30
3-1	09/07/2017 09:50:00	BOD_5	3.8	mg/L	0.6	2

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
4-1	09/07/2017 08:20:00	NOx	<0.050	mg/L	0.025	0.05
4-1	09/07/2017 08:20:00	NH_3	<0.100	mg/L	0.017	0.1
4-1	09/07/2017 08:20:00	TKN	<0.250	mg/L	0.05	0.25
4-1	09/07/2017 08:20:00	PO_4	<0.025	mg/L	0.009	0.025
4-1	09/07/2017 08:20:00	TP	0.062	mg/L	0.005	0.025
4-1	09/07/2017 08:20:00	pH	7.69	pH Units	0	0.01
4-1	09/07/2017 08:20:00	BOD_{20}	4.4	mg/L	0.6	2
4-1	09/07/2017 08:20:00	$CBOD_{20}$	2.5	mg/L	0.6	2
4-1	09/07/2017 08:20:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
4-1	09/07/2017 08:20:00	$CBOD_5$	2.5	mg/L	0.6	2
4-1	09/07/2017 08:20:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
4-1	09/07/2017 08:20:00	TEMP	23.7	°C	-40	-30
4-1	09/07/2017 08:20:00	BOD_5	3.4	mg/L	0.6	2
4-2	09/07/2017 10:17:00	NOx	<0.050	mg/L	0.025	0.05
4-2	09/07/2017 10:17:00	NH_3	<0.100	mg/L	0.017	0.1
4-2	09/07/2017 10:17:00	TKN	<0.250	mg/L	0.05	0.25
4-2	09/07/2017 10:17:00	pH	7.86	pH Units	0	0.01
4-2	09/07/2017 10:17:00	BOD_{20}	3.9	mg/L	0.6	2
4-2	09/07/2017 10:17:00	$CBOD_{20}$-dslv	2.1	mg/L	0.6	2
4-2	09/07/2017 10:17:00	$CBOD_5$-dslv	2.1	mg/L	0.6	2
4-2	09/07/2017 10:17:00	TEMP	24.2	°C	-40	-30
4-2	09/07/2017 10:17:00	BOD_5	3.2	mg/L	0.6	2
5-1	09/07/2017 08:56:00	TOC	9.75	mg/L	0.082	0.25
5-1	09/07/2017 08:56:00	NOx	17.4	mg/L	0.25	0.5
5-1	09/07/2017 08:56:00	NH_3	0.526	mg/L	0.017	0.1
5-1	09/07/2017 08:56:00	TKN	0.677	mg/L	0.05	0.25
5-1	09/07/2017 08:56:00	PO_4	3.26	mg/L	0.09	0.25
5-1	09/07/2017 08:56:00	TP	4.05	mg/L	0.05	0.25
5-1	09/07/2017 08:56:00	TDS	386	mg/L	10	25
5-1	09/07/2017 08:56:00	TSS	4.75	mg/L	0.25	3.12
5-1	09/07/2017 08:56:00	pH	7.58	pH Units	0	0.01
5-1	09/07/2017 08:56:00	BOD_{20}	9.8	mg/L	0.6	2
5-1	09/07/2017 08:56:00	$CBOD_{20}$	3.5	mg/L	0.6	2
5-1	09/07/2017 08:56:00	$CBOD_{20}$-dslv	2.1	mg/L	0.6	2
5-1	09/07/2017 08:56:00	$CBOD_5$	3.5	mg/L	0.6	2
5-1	09/07/2017 08:56:00	$CBOD_5$-dslv	2.1	mg/L	0.6	2
5-1	09/07/2017 08:56:00	TEMP	24.5	°C	-40	-30
5-1	09/07/2017 08:56:00	BOD_5	5.5	mg/L	0.6	2
5-2	09/07/2017 10:48:00	NOx	<0.050	mg/L	0.025	0.05
5-2	09/07/2017 10:48:00	NH_3	<0.100	mg/L	0.017	0.1
5-2	09/07/2017 10:48:00	TKN	<0.250	mg/L	0.05	0.25

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
5-2	09/07/2017 10:48:00	pH	7.88	pH Units	0	0.01
5-2	09/07/2017 10:48:00	BOD_{20}	4.6	mg/L	0.6	2
5-2	09/07/2017 10:48:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
5-2	09/07/2017 10:48:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
5-2	09/07/2017 10:48:00	TEMP	25.8	°C	-40	-30
5-2	09/07/2017 10:48:00	BOD5	3.3	mg/L	0.6	2
6-1	09/07/2017 09:13:00	TOC	6.17	mg/L	0.082	0.25
6-1	09/07/2017 09:13:00	NOx	2.3	mg/L	0.25	0.5
6-1	09/07/2017 09:13:00	NH_3	0.156	mg/L	0.017	0.1
6-1	09/07/2017 09:13:00	TKN	0.289	mg/L	0.05	0.25
6-1	09/07/2017 09:13:00	PO_4	0.503	mg/L	0.009	0.025
6-1	09/07/2017 09:13:00	TP	0.579	mg/L	0.005	0.025
6-1	09/07/2017 09:13:00	TDS	228	mg/L	10	25
6-1	09/07/2017 09:13:00	TSS	23	mg/L	0.667	8.33
6-1	09/07/2017 09:13:00	pH	7.17	pH Units	0	0.01
6-1	09/07/2017 09:13:00	BOD_{20}	4.8	mg/L	0.6	2
6-1	09/07/2017 09:13:00	$CBOD_{20}$	3	mg/L	0.6	2
6-1	09/07/2017 09:13:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
6-1	09/07/2017 09:13:00	$CBOD_5$	3	mg/L	0.6	2
6-1	09/07/2017 09:13:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
6-1	09/07/2017 09:13:00	CHL-A	3.2	mg/m^3	0.25	0.5
6-1	09/07/2017 09:13:00	TEMP	23.4	°C	-40	-30
6-1	09/07/2017 09:13:00	BOD_5	3.2	mg/L	0.6	2
6-2	09/07/2017 10:56:00	NOx	<0.050	mg/L	0.025	0.05
6-2	09/07/2017 10:56:00	NH_3	<0.100	mg/L	0.017	0.1
6-2	09/07/2017 10:56:00	TKN	<0.250	mg/L	0.05	0.25
6-2	09/07/2017 10:56:00	pH	7.92	pH Units	0	0.01
6-2	09/07/2017 10:56:00	BOD_{20}	4.2	mg/L	0.6	2
6-2	09/07/2017 10:56:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
6-2	09/07/2017 10:56:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
6-2	09/07/2017 10:56:00	TEMP	24.7	°C	-40	-30
6-2	09/07/2017 10:56:00	BOD_5	3.9	mg/L	0.6	2
7-1	09/07/2017 11:36:00	TOC	5.85	mg/L	0.082	0.25
7-1	09/07/2017 11:36:00	NOx	<0.050	mg/L	0.025	0.05
7-1	09/07/2017 11:36:00	NH_3	<0.100	mg/L	0.017	0.1
7-1	09/07/2017 11:36:00	TKN	<0.250	mg/L	0.05	0.25
7-1	09/07/2017 11:36:00	PO_4	<0.025	mg/L	0.009	0.025
7-1	09/07/2017 11:36:00	TP	0.066	mg/L	0.005	0.025
7-1	09/07/2017 11:36:00	TDS	296	mg/L	10	25
7-1	09/07/2017 11:36:00	TSS	13.6	mg/L	0.4	5
7-1	09/07/2017 11:36:00	pH	7.95	pH Units	0	0.01

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
7-1	09/07/2017 11:36:00	BOD_{20}	5.2	mg/L	0.6	2
7-1	09/07/2017 11:36:00	$CBOD_{20}$	2.9	mg/L	0.6	2
7-1	09/07/2017 11:36:00	$CBOD_{20}$-dslv	2.5	mg/L	0.6	2
7-1	09/07/2017 11:36:00	$CBOD_5$	2.9	mg/L	0.6	2
7-1	09/07/2017 11:36:00	$CBOD_5$-dslv	2.5	mg/L	0.6	2
7-1	09/07/2017 11:36:00	CHL-A	12.3	mg/m^3	0.25	0.5
7-1	09/07/2017 11:36:00	TEMP	21.2	°C	-40	-30
7-1	09/07/2017 11:36:00	BOD_5	3.6	mg/L	0.6	2
7-2	09/07/2017 10:13:00	NOx	<0.050	mg/L	0.025	0.05
7-2	09/07/2017 10:13:00	NH_3	0.104	mg/L	0.017	0.1
7-2	09/07/2017 10:13:00	TKN	<0.250	mg/L	0.05	0.25
7-2	09/07/2017 10:13:00	pH	7.47	pH Units	0	0.01
7-2	09/07/2017 10:13:00	BOD_{20}	4.2	mg/L	0.6	2
7-2	09/07/2017 10:13:00	$CBOD_{20}$-dslv	2.3	mg/L	0.6	2
7-2	09/07/2017 10:13:00	$CBOD_5$-dslv	2.3	mg/L	0.6	2
7-2	09/07/2017 10:13:00	TEMP	20.8	°C	-40	-30
7-2	09/07/2017 10:13:00	BOD_5	3	mg/L	0.6	2
8-1	09/07/2017 08:40:00	TOC	6.18	mg/L	0.082	0.25
8-1	09/07/2017 08:40:00	NOx	2.01	mg/L	0.025	0.05
8-1	09/07/2017 08:40:00	NH_3	0.1	mg/L	0.017	0.1
8-1	09/07/2017 08:40:00	TKN	<0.250	mg/L	0.05	0.25
8-1	09/07/2017 08:40:00	PO_4	0.399	mg/L	0.009	0.025
8-1	09/07/2017 08:40:00	TP	0.47	mg/L	0.005	0.025
8-1	09/07/2017 08:40:00	TDS	224	mg/L	10	25
8-1	09/07/2017 08:40:00	TSS	18	mg/L	0.5	6.25
8-1	09/07/2017 08:40:00	pH	7.55	pH Units	0	0.01
8-1	09/07/2017 08:40:00	BOD_{20}	5.6	mg/L	0.6	2
8-1	09/07/2017 08:40:00	$CBOD_{20}$	3.2	mg/L	0.6	2
8-1	09/07/2017 08:40:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
8-1	09/07/2017 08:40:00	$CBOD_5$	3.2	mg/L	0.6	2
8-1	09/07/2017 08:40:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
8-1	09/07/2017 08:40:00	CHL-A	7.48	mg/m^3	0.25	0.5
8-1	09/07/2017 08:40:00	TEMP	22.6	°C	-40	-30
8-1	09/07/2017 08:40:00	BOD_5	3.5	mg/L	0.6	2
8-2	09/07/2017 10:39:00	NOx	1.95	mg/L	0.025	0.05
8-2	09/07/2017 10:39:00	NH_3	0.16	mg/L	0.017	0.1
8-2	09/07/2017 10:39:00	TKN	<0.250	mg/L	0.05	0.25
8-2	09/07/2017 10:39:00	pH	8.04	pH Units	0	0.01
8-2	09/07/2017 10:39:00	BOD_{20}	8.1	mg/L	0.6	2
8-2	09/07/2017 10:39:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
8-2	09/07/2017 10:39:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
8-2	09/07/2017 10:39:00	TEMP	23.9	°C	-40	-30
8-2	09/07/2017 10:39:00	BOD_5	4.7	mg/L	0.6	2
9-1	09/07/2017 11:12:00	NOx	<0.050	mg/L	0.025	0.05
9-1	09/07/2017 11:12:00	NH_3	0.101	mg/L	0.017	0.1
9-1	09/07/2017 11:12:00	TKN	<0.250	mg/L	0.05	0.25
9-1	09/07/2017 11:12:00	PO_4	<0.025	mg/L	0.009	0.025
9-1	09/07/2017 11:12:00	TP	0.054	mg/L	0.005	0.025
9-1	09/07/2017 11:12:00	pH	7.41	pH Units	0	0.01
9-1	09/07/2017 11:12:00	BOD_{20}	4	mg/L	0.6	2
9-1	09/07/2017 11:12:00	$CBOD_{20}$	2.7	mg/L	0.6	2
9-1	09/07/2017 11:12:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
9-1	09/07/2017 11:12:00	$CBOD_5$	2.7	mg/L	0.6	2
9-1	09/07/2017 11:12:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
9-1	09/07/2017 11:12:00	TEMP	21.8	°C	-40	-30
9-1	09/07/2017 11:12:00	BOD_5	2.9	mg/L	0.6	2
10-1	09/07/2017 11:55:00	NOx	<0.050	mg/L	0.025	0.05
10-1	09/07/2017 11:55:00	NH_3	<0.100	mg/L	0.017	0.1
10-1	09/07/2017 11:55:00	TKN	<0.250	mg/L	0.05	0.25
10-1	09/07/2017 11:55:00	PO_4	0.119	mg/L	0.009	0.025
10-1	09/07/2017 11:55:00	TP	0.157	mg/L	0.005	0.025
10-1	09/07/2017 11:55:00	pH	8.75	pH Units	0	0.01
10-1	09/07/2017 11:55:00	BOD_{20}	8.9	mg/L	0.6	2
10-1	09/07/2017 11:55:00	$CBOD_{20}$	5.1	mg/L	0.6	2
10-1	09/07/2017 11:55:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
10-1	09/07/2017 11:55:00	$CBOD_5$	5.1	mg/L	0.6	2
10-1	09/07/2017 11:55:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
10-1	09/07/2017 11:55:00	TEMP	23.1	°C	-40	-30
10-1	09/07/2017 11:55:00	BOD_5	5.8	mg/L	0.6	2
1-3	09/07/2017 13:46:00	TOC	5.83	mg/L	0.082	0.25
1-3	09/07/2017 13:46:00	NOx	<0.050	mg/L	0.025	0.05
1-3	09/07/2017 13:46:00	NH_3	<0.100	mg/L	0.017	0.1
1-3	09/07/2017 13:46:00	TKN	<0.250	mg/L	0.05	0.25
1-3	09/07/2017 13:46:00	PO_4	<0.025	mg/L	0.009	0.025
1-3	09/07/2017 13:46:00	TP	0.074	mg/L	0.005	0.025
1-3	09/07/2017 13:46:00	TDS	247	mg/L	10	25
1-3	09/07/2017 13:46:00	TSS	12.6	mg/L	0.571	7.14
1-3	09/07/2017 13:46:00	pH	7.81	pH Units	0	0.01
1-3	09/07/2017 13:46:00	BOD_{20}	6.2	mg/L	0.6	2
1-3	09/07/2017 13:46:00	$CBOD_{20}$	4	mg/L	0.6	2
1-3	09/07/2017 13:46:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
1-3	09/07/2017 13:46:00	$CBOD_5$	4	mg/L	0.6	2

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
1-3	09/07/2017 13:46:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
1-3	09/07/2017 13:46:00	CHL-A	13.4	mg/m³	0.25	0.5
1-3	09/07/2017 13:46:00	TEMP	24.8	°C	-40	-30
1-3	09/07/2017 13:46:00	BOD_5	4.1	mg/L	0.6	2
2-3	09/07/2017 14:36:00	NOx	<0.050	mg/L	0.025	0.05
2-3	09/07/2017 14:36:00	NH_3	0.116	mg/L	0.017	0.1
2-3	09/07/2017 14:36:00	TKN	<0.250	mg/L	0.05	0.25
2-3	09/07/2017 14:36:00	PO_4	<0.025	mg/L	0.009	0.025
2-3	09/07/2017 14:36:00	TP	0.078	mg/L	0.005	0.025
2-3	09/07/2017 14:36:00	pH	7.98	pH Units	0	0.01
2-3	09/07/2017 14:36:00	BOD_{20}	5.1	mg/L	0.6	2
2-3	09/07/2017 14:36:00	$CBOD_{20}$	3	mg/L	0.6	2
2-3	09/07/2017 14:36:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
2-3	09/07/2017 14:36:00	$CBOD_5$	3	mg/L	0.6	2
2-3	09/07/2017 14:36:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
2-3	09/07/2017 14:36:00	TEMP	22	°C	-40	-30
2-3	09/07/2017 14:36:00	BOD_5	3	mg/L	0.6	2
3-3	09/07/2017 15:04:00	TOC	6.26	mg/L	0.082	0.25
3-3	09/07/2017 15:04:00	NOx	<0.050	mg/L	0.025	0.05
3-3	09/07/2017 15:04:00	NH_3	0.199	mg/L	0.017	0.1
3-3	09/07/2017 15:04:00	TKN	0.571	mg/L	0.05	0.25
3-3	09/07/2017 15:04:00	PO_4	<0.025	mg/L	0.009	0.025
3-3	09/07/2017 15:04:00	TP	0.576	mg/L	0.005	0.025
3-3	09/07/2017 15:04:00	TDS	193	mg/L	10	25
3-3	09/07/2017 15:04:00	TSS	281	mg/L	1.33	16.7
3-3	09/07/2017 15:04:00	pH	7.67	pH Units	0	0.01
3-3	09/07/2017 15:04:00	BOD_{20}	6.8	mg/L	0.6	2
3-3	09/07/2017 15:04:00	$CBOD_{20}$	4	mg/L	0.6	2
3-3	09/07/2017 15:04:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
3-3	09/07/2017 15:04:00	$CBOD_5$	4	mg/L	0.6	2
3-3	09/07/2017 15:04:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
3-3	09/07/2017 15:04:00	CHL-A	8.01	mg/m³	0.25	0.5
3-3	09/07/2017 15:04:00	TEMP	26.7	°C	-40	-30
3-3	09/07/2017 15:04:00	BOD_5	4	mg/L	0.6	2
4-3	09/07/2017 14:17:00	NOx	<0.050	mg/L	0.025	0.05
4-3	09/07/2017 14:17:00	NH_3	0.128	mg/L	0.017	0.1
4-3	09/07/2017 14:17:00	TKN	<0.250	mg/L	0.05	0.25
4-3	09/07/2017 14:17:00	PO_4	<0.025	mg/L	0.009	0.025
4-3	09/07/2017 14:17:00	TP	0.078	mg/L	0.005	0.025
4-3	09/07/2017 14:17:00	pH	8.21	pH Units	0	0.01
4-3	09/07/2017 14:17:00	BOD_{20}	6.2	mg/L	0.6	2

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
4-3	09/07/2017 14:17:00	$CBOD_{20}$	4	mg/L	0.6	2
4-3	09/07/2017 14:17:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
4-3	09/07/2017 14:17:00	$CBOD_5$	4	mg/L	0.6	2
4-3	09/07/2017 14:17:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
4-3	09/07/2017 14:17:00	TEMP	26.5	°C	-40	-30
4-3	09/07/2017 14:17:00	BOD_5	4	mg/L	0.6	2
4-4	09/07/2017 15:33:00	NOx	<0.050	mg/L	0.025	0.05
4-4	09/07/2017 15:33:00	NH_3	0.107	mg/L	0.017	0.1
4-4	09/07/2017 15:33:00	TKN	<0.250	mg/L	0.05	0.25
4-4	09/07/2017 15:33:00	pH	8.06	pH Units	0	0.01
4-4	09/07/2017 15:33:00	BOD_{20}	4.9	mg/L	0.6	2
4-4	09/07/2017 15:33:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
4-4	09/07/2017 15:33:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
4-4	09/07/2017 15:33:00	TEMP	26.5	°C	-40	-30
4-4	09/07/2017 15:33:00	BOD_5	3.2	mg/L	0.6	2
5-3	09/07/2017 14:31:00	TOC	9.46	mg/L	0.082	0.25
5-3	09/07/2017 14:31:00	NOx	14	mg/L	0.25	0.5
5-3	09/07/2017 14:31:00	NH_3	0.335	mg/L	0.017	0.1
5-3	09/07/2017 14:31:00	TKN	0.906	mg/L	0.05	0.25
5-3	09/07/2017 14:31:00	PO_4	3.44	mg/L	0.09	0.25
5-3	09/07/2017 14:31:00	TP	3.57	mg/L	0.05	0.25
5-3	09/07/2017 14:31:00	TDS	405	mg/L	10	25
5-3	09/07/2017 14:31:00	TSS	4	mg/L	0.25	3.12
5-3	09/07/2017 14:31:00	pH	7.35	pH Units	0	0.01
5-3	09/07/2017 14:31:00	BOD_{20}	8.2	mg/L	0.6	2
5-3	09/07/2017 14:31:00	$CBOD_{20}$	4.4	mg/L	0.6	2
5-3	09/07/2017 14:31:00	$CBOD_{20}$-dslv	2.1	mg/L	0.6	2
5-3	09/07/2017 14:31:00	$CBOD_5$	3.2	mg/L	0.6	2
5-3	09/07/2017 14:31:00	$CBOD_5$-dslv	2.1	mg/L	0.6	2
5-3	09/07/2017 14:31:00	TEMP	26.4	°C	-40	-30
5-3	09/07/2017 14:31:00	BOD_5	3.6	mg/L	0.6	2
5-4	09/07/2017 15:41:00	NOx	<0.050	mg/L	0.025	0.05
5-4	09/07/2017 15:41:00	NH_3	<0.100	mg/L	0.017	0.1
5-4	09/07/2017 15:41:00	TKN	<0.250	mg/L	0.05	0.25
5-4	09/07/2017 15:41:00	pH	8.01	pH Units	0	0.01
5-4	09/07/2017 15:41:00	BOD_{20}	4.9	mg/L	0.6	2
5-4	09/07/2017 15:41:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
5-4	09/07/2017 15:41:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
5-4	09/07/2017 15:41:00	TEMP	26.4	°C	-40	-30
5-4	09/07/2017 15:41:00	BOD_5	3	mg/L	0.6	2
6-3	09/07/2017 14:53:00	TOC	6.19	mg/L	0.082	0.25

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
6-3	09/07/2017 14:53:00	NOx	2.86	mg/L	0.25	0.5
6-3	09/07/2017 14:53:00	NH_3	0.153	mg/L	0.017	0.1
6-3	09/07/2017 14:53:00	TKN	0.528	mg/L	0.05	0.25
6-3	09/07/2017 14:53:00	PO_4	0.611	mg/L	0.09	0.25
6-3	09/07/2017 14:53:00	TP	0.714	mg/L	0.005	0.025
6-3	09/07/2017 14:53:00	TDS	244	mg/L	10	25
6-3	09/07/2017 14:53:00	TSS	20.3	mg/L	0.571	7.14
6-3	09/07/2017 14:53:00	pH	8.09	pH Units	0	0.01
6-3	09/07/2017 14:53:00	BOD_{20}	6	mg/L	0.6	2
6-3	09/07/2017 14:53:00	$CBOD_{20}$	4	mg/L	0.6	2
6-3	09/07/2017 14:53:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
6-3	09/07/2017 14:53:00	$CBOD_5$	4	mg/L	0.6	2
6-3	09/07/2017 14:53:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
6-3	09/07/2017 14:53:00	CHL-A	5.34	mg/m³	0.25	0.5
6-3	09/07/2017 14:53:00	TEMP	26.9	°C	-40	-30
6-3	09/07/2017 14:53:00	BOD_5	4	mg/L	0.6	2
6-4	09/07/2017 15:47:00	NOx	<0.050	mg/L	0.025	0.05
6-4	09/07/2017 15:47:00	NH_3	0.119	mg/L	0.017	0.1
6-4	09/07/2017 15:47:00	TKN	<0.250	mg/L	0.05	0.25
6-4	09/07/2017 15:47:00	pH	8.09	pH Units	0	0.01
6-4	09/07/2017 15:47:00	BOD_{20}	5.9	mg/L	0.6	2
6-4	09/07/2017 15:47:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
6-4	09/07/2017 15:47:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
6-4	09/07/2017 15:47:00	TEMP	26.6	°C	-40	-30
6-4	09/07/2017 15:47:00	BOD_5	3.6	mg/L	0.6	2
7-3	09/07/2017 13:43:00	TOC	5.9	mg/L	0.082	0.25
7-3	09/07/2017 13:43:00	NOx	<0.050	mg/L	0.025	0.05
7-3	09/07/2017 13:43:00	NH_3	<0.100	mg/L	0.017	0.1
7-3	09/07/2017 13:43:00	TKN	<0.250	mg/L	0.05	0.25
7-3	09/07/2017 13:43:00	PO_4	<0.025	mg/L	0.009	0.025
7-3	09/07/2017 13:43:00	TP	0.061	mg/L	0.005	0.025
7-3	09/07/2017 13:43:00	TDS	297	mg/L	10	25
7-3	09/07/2017 13:43:00	TSS	11.4	mg/L	0.286	3.57
7-3	09/07/2017 13:43:00	pH	8.06	pH Units	0	0.01
7-3	09/07/2017 13:43:00	BOD_{20}	5.1	mg/L	0.6	2
7-3	09/07/2017 13:43:00	$CBOD_{20}$	4	mg/L	0.6	2
7-3	09/07/2017 13:43:00	$CBOD_{20}$-dslv	3.3	mg/L	0.6	2
7-3	09/07/2017 13:43:00	$CBOD_5$	4	mg/L	0.6	2
7-3	09/07/2017 13:43:00	$CBOD_5$-dslv	3.3	mg/L	0.6	2
7-3	09/07/2017 13:43:00	CHL-A	5.34	mg/m³	0.25	0.5
7-3	09/07/2017 13:43:00	TEMP	22.4	°C	-40	-30

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
7-3	09/07/2017 13:43:00	BOD_5	4	mg/L	0.6	2
7-4	09/07/2017 15:48:00	NOx	0.139	mg/L	0.025	0.05
7-4	09/07/2017 15:48:00	NH_3	0.118	mg/L	0.017	0.1
7-4	09/07/2017 15:48:00	TKN	<0.250	mg/L	0.05	0.25
7-4	09/07/2017 15:48:00	pH	7.64	pH Units	0	0.01
7-4	09/07/2017 15:48:00	BOD_{20}	5.6	mg/L	0.6	2
7-4	09/07/2017 15:48:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
7-4	09/07/2017 15:48:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
7-4	09/07/2017 15:48:00	TEMP	22.8	°C	-40	-30
7-4	09/07/2017 15:48:00	BOD_5	3.5	mg/L	0.6	2
8-3	09/07/2017 14:12:00	TOC	6.12	mg/L	0.082	0.25
8-3	09/07/2017 14:12:00	NOx	1.81	mg/L	0.025	0.05
8-3	09/07/2017 14:12:00	NH_3	<0.100	mg/L	0.017	0.1
8-3	09/07/2017 14:12:00	TKN	0.308	mg/L	0.05	0.25
8-3	09/07/2017 14:12:00	PO_4	0.375	mg/L	0.009	0.025
8-3	09/07/2017 14:12:00	TP	0.453	mg/L	0.005	0.025
8-3	09/07/2017 14:12:00	TDS	242	mg/L	10	25
8-3	09/07/2017 14:12:00	TSS	15.1	mg/L	0.444	5.56
8-3	09/07/2017 14:12:00	pH	8.27	pH Units	0	0.01
8-3	09/07/2017 14:12:00	BOD_{20}	6.1	mg/L	0.6	2
8-3	09/07/2017 14:12:00	$CBOD_{20}$	3.2	mg/L	0.6	2
8-3	09/07/2017 14:12:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
8-3	09/07/2017 14:12:00	$CBOD_5$	3.2	mg/L	0.6	2
8-3	09/07/2017 14:12:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
8-3	09/07/2017 14:12:00	CHL-A	6.41	mg/m^3	0.25	0.5
8-3	09/07/2017 14:12:00	TEMP	25.3	°C	-40	-30
8-3	09/07/2017 14:12:00	BOD_5	3.6	mg/L	0.6	2
8-4	09/07/2017 16:11:00	NOx	1.84	mg/L	0.025	0.05
8-4	09/07/2017 16:11:00	NH_3	<0.100	mg/L	0.017	0.1
8-4	09/07/2017 16:11:00	TKN	0.325	mg/L	0.05	0.25
8-4	09/07/2017 16:11:00	pH	8.3	pH Units	0	0.01
8-4	09/07/2017 16:11:00	BOD_{20}	13	mg/L	0.6	2
8-4	09/07/2017 16:11:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
8-4	09/07/2017 16:11:00	CBOD5-dslv	<2.0	mg/L	0.6	2
8-4	09/07/2017 16:11:00	TEMP	25.6	°C	-40	-30
8-4	09/07/2017 16:11:00	BOD_5	7.2	mg/L	0.6	2
9-3	09/07/2017 16:45:00	NOX	<0.050	mg/L	0.025	0.05
9-3	09/07/2017 16:45:00	NH_3	<0.100	mg/L	0.017	0.1
9-3	09/07/2017 16:45:00	TKN	<0.250	mg/L	0.05	0.25
9-3	09/07/2017 16:45:00	PO_4	<0.025	mg/L	0.009	0.025
9-3	09/07/2017 16:45:00	TP	0.048	mg/L	0.005	0.025

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
9-3	09/07/2017 16:45:00	pH	7.31	pH Units	0	0.01
9-3	09/07/2017 16:45:00	BOD_{20}	5.5	mg/L	0.6	2
9-3	09/07/2017 16:45:00	$CBOD_{20}$	3.7	mg/L	0.6	2
9-3	09/07/2017 16:45:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
9-3	09/07/2017 16:45:00	CBOD5	3.7	mg/L	0.6	2
9-3	09/07/2017 16:45:00	CBOD5-dslv	<2.0	mg/L	0.6	2
9-3	09/07/2017 16:45:00	TEMP	22.9	°C	-40	-30
9-3	09/07/2017 16:45:00	BOD_5	4	mg/L	0.6	2
10-3	09/07/2017 17:19:00	NOx	<0.050	mg/L	0.025	0.05
10-3	09/07/2017 17:19:00	NH_3	<0.100	mg/L	0.017	0.1
10-3	09/07/2017 17:19:00	TKN	0.297	mg/L	0.05	0.25
10-3	09/07/2017 17:19:00	PO_4	0.071	mg/L	0.009	0.025
10-3	09/07/2017 17:19:00	TP	0.148	mg/L	0.005	0.025
10-3	09/07/2017 17:19:00	pH	8.9	pH Units	0	0.01
10-3	09/07/2017 17:19:00	BOD_{20}	11.5	mg/L	0.6	2
10-3	09/07/2017 17:19:00	$CBOD_{20}$	6	mg/L	0.6	2
10-3	09/07/2017 17:19:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2
10-3	09/07/2017 17:19:00	$CBOD_5$	6	mg/L	0.6	2
10-3	09/07/2017 17:19:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2
10-3	09/07/2017 17:19:00	TEMP	24.5	°C	-40	-30
10-3	09/07/2017 17:19:00	BOD_5	6.2	mg/L	0.6	2

Table A-4. Grab sample results from October sampling trip

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
1-1	10/06/2017 07:40:00	TOC	4.05	mg/L	0.082	0.250
1-1	10/06/2017 07:40:00	pH	7.93	pH	0	0.01
1-1	10/06/2017 07:40:00	TEMP	22.6	C	-40.0	-30.0
1-1	10/06/2017 07:40:00	NOx	0.108	mg/L	0.025	0.050
1-1	10/06/2017 07:40:00	PO_4	<0.025	mg/L	0.009	0.025
1-1	10/06/2017 07:40:00	TKN	0.502	mg/L	0.050	0.250
1-1	10/06/2017 07:40:00	TP	0.113	mg/L	0.005	0.025
1-1	10/06/2017 07:40:00	TDS	150	mg/L	10.0	25.0
1-1	10/06/2017 07:40:00	TSS	46	mg/L	0.800	10.0
1-1	10/06/2017 07:40:00	BOD_{20}	4.4	mg/L	0.6	2.0
1-1	10/06/2017 07:40:00	CHL-A	10.6	mg/m^3	0.25	0.50
1-1	10/06/2017 07:40:00	BOD_5	2.1	mg/L	0.6	2.0
1-1	10/06/2017 07:40:00	NH_3	<0.100	mg/L	0.017	0.100
1-1	10/06/2017 07:40:00	$CBOD_{20}$	<2.0	mg/L	0.6	2.0
1-1	10/06/2017 07:40:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
1-1	10/06/2017 07:40:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
1-1	10/06/2017 07:40:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
1-3	10/06/2017 13:01:00	PO_4	<0.025	mg/L	0.009	0.025
1-3	10/06/2017 13:01:00	NH_3	<0.100	mg/L	0.017	0.100
1-3	10/06/2017 13:01:00	TKN	<0.250	mg/L	0.050	0.250
1-3	10/06/2017 13:01:00	TOC	4.77	mg/L	0.082	0.250
1-3	10/06/2017 13:01:00	pH	7.24	pH	0	0.01
1-3	10/06/2017 13:01:00	TEMP	22.9	C	-40.0	-30.0
1-3	10/06/2017 13:01:00	NOx	0.141	mg/L	0.025	0.050
1-3	10/06/2017 13:01:00	TP	0.117	mg/L	0.005	0.025
1-3	10/06/2017 13:01:00	TDS	183	mg/L	10.0	25.0
1-3	10/06/2017 13:01:00	TSS	44.5	mg/L	1.00	12.5
1-3	10/06/2017 13:01:00	BOD_{20}	7	mg/L	0.6	2.0
1-3	10/06/2017 13:01:00	$CBOD_{20}$	7	mg/L	0.6	2.0
1-3	10/06/2017 13:01:00	$CBOD_{20}$-dslv	5.2	mg/L	0.6	2.0
1-3	10/06/2017 13:01:00	$CBOD_5$	2.6	mg/L	0.6	2.0
1-3	10/06/2017 13:01:00	CHL-A	13	mg/m^3	0.25	0.50
1-3	10/06/2017 13:01:00	BOD_5	2.8	mg/L	0.6	2.0
1-3	10/06/2017 13:01:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
2-1	10/06/2017 08:57:00	PO4	<0.025	mg/L	0.009	0.025
2-1	10/06/2017 08:57:00	pH	7.58	pH	0	0.01
2-1	10/06/2017 08:57:00	TEMP	22.5	C	-40.0	-30.0
2-1	10/06/2017 08:57:00	NOx	0.068	mg/L	0.025	0.050
2-1	10/06/2017 08:57:00	TKN	0.662	mg/L	0.050	0.250
2-1	10/06/2017 08:57:00	TP	0.111	mg/L	0.005	0.025
2-1	10/06/2017 08:57:00	BOD_{20}	5	mg/L	0.6	2.0
2-1	10/06/2017 08:57:00	$CBOD_{20}$	3.4	mg/L	0.6	2.0
2-1	10/06/2017 08:57:00	$CBOD_5$	2	mg/L	0.6	2.0
2-1	10/06/2017 08:57:00	BOD_5	2	mg/L	0.6	2.0
2-1	10/06/2017 08:57:00	NH_3	<0.100	mg/L	0.017	0.100
2-1	10/06/2017 08:57:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
2-1	10/06/2017 08:57:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
2-3	10/06/2017 14:27:00	PO_4	<0.025	mg/L	0.009	0.025
2-3	10/06/2017 14:27:00	NOx	<0.050	mg/L	0.025	0.050
2-3	10/06/2017 14:27:00	NH_3	<0.100	mg/L	0.017	0.100

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
2-3	10/06/2017 14:27:00	TKN	<0.250	mg/L	0.050	0.250
2-3	10/06/2017 14:27:00	pH	7.64	pH	0	0.01
2-3	10/06/2017 14:27:00	TEMP	23.5	C	-40.0	-30.0
2-3	10/06/2017 14:27:00	TP	0.076	mg/L	0.005	0.025
2-3	10/06/2017 14:27:00	BOD_{20}	6.9	mg/L	0.6	2.0
2-3	10/06/2017 14:27:00	$CBOD_{20}$	6.5	mg/L	0.6	2.0
2-3	10/06/2017 14:27:00	$CBOD_{20}$-dslv	4	mg/L	0.6	2.0
2-3	10/06/2017 14:27:00	$CBOD_5$	2.2	mg/L	0.6	2.0
2-3	10/06/2017 14:27:00	BOD_5	2.7	mg/L	0.6	2.0
2-3	10/06/2017 14:27:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
3-1	10/06/2017 09:15:00	PO_4	<0.025	mg/L	0.009	0.025
3-1	10/06/2017 09:15:00	TOC	4.42	mg/L	0.082	0.250
3-1	10/06/2017 09:15:00	pH	7.63	pH	0	0.01
3-1	10/06/2017 09:15:00	TEMP	22.5	C	-40.0	-30.0
3-1	10/06/2017 09:15:00	NH_3	0.103	mg/L	0.017	0.100
3-1	10/06/2017 09:15:00	TKN	0.689	mg/L	0.050	0.250
3-1	10/06/2017 09:15:00	NOx	<0.050	mg/L	0.025	0.050
3-1	10/06/2017 09:15:00	TP	0.117	mg/L	0.005	0.025
3-1	10/06/2017 09:15:00	TDS	172	mg/L	10.0	25.0
3-1	10/06/2017 09:15:00	TSS	29.2	mg/L	0.800	10.0
3-1	10/06/2017 09:15:00	BOD_{20}	8.7	mg/L	0.6	2.0
3-1	10/06/2017 09:15:00	$CBOD_{20}$	7.9	mg/L	0.6	2.0
3-1	10/06/2017 09:15:00	$CBOD_5$	5	mg/L	0.6	2.0
3-1	10/06/2017 09:15:00	CHL-A	29.9	mg/m^3	0.25	0.50
3-1	10/06/2017 09:15:00	BOD_5	5.6	mg/L	0.6	2.0
3-1	10/06/2017 09:15:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
3-1	10/06/2017 09:15:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
3-3	10/06/2017 15:20:00	NOx	<0.050	mg/L	0.025	0.050
3-3	10/06/2017 15:20:00	TOC	7.54	mg/L	0.082	0.250
3-3	10/06/2017 15:20:00	pH	8.43	pH	0	0.01
3-3	10/06/2017 15:20:00	TEMP	24	C	-40.0	-30.0
3-3	10/06/2017 15:20:00	NH_3	0.256	mg/L	0.017	0.100
3-3	10/06/2017 15:20:00	TKN	1.34	mg/L	0.050	0.250
3-3	10/06/2017 15:20:00	PO_4	0.185	mg/L	0.009	0.025
3-3	10/06/2017 15:20:00	TP	0.626	mg/L	0.005	0.025
3-3	10/06/2017 15:20:00	TDS	192	mg/L	10.0	25.0
3-3	10/06/2017 15:20:00	TSS	59.3	mg/L	1.33	16.7
3-3	10/06/2017 15:20:00	BOD_{20}	47	mg/L	0.6	2.0
3-3	10/06/2017 15:20:00	$CBOD_{20}$	46.9	mg/L	0.6	2.0
3-3	10/06/2017 15:20:00	$CBOD_{20}$-dslv	21.6	mg/L	0.6	2.0
3-3	10/06/2017 15:20:00	$CBOD_5$	13.2	mg/L	0.6	2.0
3-3	10/06/2017 15:20:00	$CBOD_5$-dslv	10.2	mg/L	0.6	2.0
3-3	10/06/2017 15:20:00	CHL-A	494	mg/m^3	0.25	0.50
3-3	10/06/2017 15:20:00	BOD_5	13.5	mg/L	0.6	2.0
4-1	10/06/2017 08:16:00	PO_4	<0.025	mg/L	0.009	0.025
4-1	10/06/2017 08:16:00	NOx	<0.050	mg/L	0.025	0.050
4-1	10/06/2017 08:16:00	pH	7.76	pH	0	0.01
4-1	10/06/2017 08:16:00	TEMP	22.6	C	-40.0	-30.0
4-1	10/06/2017 08:16:00	TKN	0.372	mg/L	0.050	0.250
4-1	10/06/2017 08:16:00	TP	0.083	mg/L	0.005	0.025
4-1	10/06/2017 08:16:00	BOD_{20}	5.9	mg/L	0.6	2.0
4-1	10/06/2017 08:16:00	$CBOD_{20}$	4.3	mg/L	0.6	2.0

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
4-1	10/06/2017 08:16:00	$CBOD_5$	2.7	mg/L	0.6	2.0
4-1	10/06/2017 08:16:00	BOD_5	3	mg/L	0.6	2.0
4-1	10/06/2017 08:16:00	NH_3	<0.100	mg/L	0.017	0.100
4-1	10/06/2017 08:16:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
4-1	10/06/2017 08:16:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
4-2	10/06/2017 09:53:00	NOx	<0.050	mg/L	0.025	0.050
4-2	10/06/2017 09:53:00	pH	7.75	pH	0	0.01
4-2	10/06/2017 09:53:00	TEMP	22.4	C	-40.0	-30.0
4-2	10/06/2017 09:53:00	TKN	0.722	mg/L	0.050	0.250
4-2	10/06/2017 09:53:00	BOD_{20}	8.6	mg/L	0.6	2.0
4-2	10/06/2017 09:53:00	BOD_5	4.2	mg/L	0.6	2.0
4-2	10/06/2017 09:53:00	NH_3	<0.100	mg/L	0.017	0.100
4-2	10/06/2017 09:53:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
4-2	10/06/2017 09:53:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
4-3	10/06/2017 14:48:00	PO_4	<0.025	mg/L	0.009	0.025
4-3	10/06/2017 14:48:00	NOx	<0.050	mg/L	0.025	0.050
4-3	10/06/2017 14:48:00	NH_3	<0.100	mg/L	0.017	0.100
4-3	10/06/2017 14:48:00	pH	7.82	pH	0	0.01
4-3	10/06/2017 14:48:00	TEMP	22.9	C	-40.0	-30.0
4-3	10/06/2017 14:48:00	TKN	0.382	mg/L	0.050	0.250
4-3	10/06/2017 14:48:00	TP	0.076	mg/L	0.005	0.025
4-3	10/06/2017 14:48:00	BOD_{20}	6.3	mg/L	0.6	2.0
4-3	10/06/2017 14:48:00	$CBOD_{20}$	4.7	mg/L	0.6	2.0
4-3	10/06/2017 14:48:00	$CBOD_{20}$-dslv	3.7	mg/L	0.6	2.0
4-3	10/06/2017 14:48:00	BOD_5	2	mg/L	0.6	2.0
4-3	10/06/2017 14:48:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
4-3	10/06/2017 14:48:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
4-4	10/06/2017 16:15:00	NOx	<0.050	mg/L	0.025	0.050
4-4	10/06/2017 16:15:00	NH_3	<0.100	mg/L	0.017	0.100
4-4	10/06/2017 16:15:00	pH	7.59	pH	0	0.01
4-4	10/06/2017 16:15:00	TEMP	22.7	C	-40.0	-30.0
4-4	10/06/2017 16:15:00	TKN	0.379	mg/L	0.050	0.250
4-4	10/06/2017 16:15:00	BOD_{20}	5.3	mg/L	0.6	2.0
4-4	10/06/2017 16:15:00	$CBOD_{20}$-dslv	3.3	mg/L	0.6	2.0
4-4	10/06/2017 16:15:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
4-4	10/06/2017 16:15:00	BOD_5	<2.0	mg/L	0.6	2.0
5-1	10/06/2017 08:26:00	TOC	5.16	mg/L	0.082	0.250
5-1	10/06/2017 08:26:00	pH	7.38	pH	0	0.01
5-1	10/06/2017 08:26:00	TEMP	24.7	C	-40.0	-30.0
5-1	10/06/2017 08:26:00	NOx	7.8	mg/L	1.25	2.50
5-1	10/06/2017 08:26:00	NH_3	0.181	mg/L	0.017	0.100
5-1	10/06/2017 08:26:00	TKN	0.573	mg/L	0.050	0.250
5-1	10/06/2017 08:26:00	PO_4	1.31	mg/L	0.090	0.250
5-1	10/06/2017 08:26:00	TP	1.7	mg/L	0.050	0.250
5-1	10/06/2017 08:26:00	TDS	311	mg/L	10.0	25.0
5-1	10/06/2017 08:26:00	TSS	3.5	mg/L	0.250	3.12
5-1	10/06/2017 08:26:00	BOD_{20}	7	mg/L	0.6	2.0
5-1	10/06/2017 08:26:00	$CBOD_{20}$	4.2	mg/L	0.6	2.0
5-1	10/06/2017 08:26:00	$CBOD_5$	2.1	mg/L	0.6	2.0
5-1	10/06/2017 08:26:00	BOD_5	3.1	mg/L	0.6	2.0
5-1	10/06/2017 08:26:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
5-1	10/06/2017 08:26:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
5-2	10/06/2017 10:10:00	pH	7.46	pH	0	0.01
5-2	10/06/2017 10:10:00	TEMP	24.8	C	-40.0	-30.0
5-2	10/06/2017 10:10:00	NOx	7.55	mg/L	1.25	2.50
5-2	10/06/2017 10:10:00	NH_3	0.176	mg/L	0.017	0.100
5-2	10/06/2017 10:10:00	TKN	0.531	mg/L	0.050	0.250
5-2	10/06/2017 10:10:00	BOD_{20}	8.6	mg/L	0.6	2.0
5-2	10/06/2017 10:10:00	BOD_5	4.4	mg/L	0.6	2.0
5-2	10/06/2017 10:10:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
5-2	10/06/2017 10:10:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
5-3	10/06/2017 14:52:00	TOC	5.37	mg/L	0.082	0.250
5-3	10/06/2017 14:52:00	pH	7.56	pH	0	0.01
5-3	10/06/2017 14:52:00	TEMP	25.3	C	-40.0	-30.0
5-3	10/06/2017 14:52:00	NOx	9.7	mg/L	1.25	2.50
5-3	10/06/2017 14:52:00	NH_3	0.403	mg/L	0.017	0.100
5-3	10/06/2017 14:52:00	TKN	0.55	mg/L	0.050	0.250
5-3	10/06/2017 14:52:00	PO_4	1.45	mg/L	0.090	0.250
5-3	10/06/2017 14:52:00	TP	1.97	mg/L	0.050	0.250
5-3	10/06/2017 14:52:00	TDS	324	mg/L	10.0	25.0
5-3	10/06/2017 14:52:00	TSS	4.38	mg/L	0.250	3.12
5-3	10/06/2017 14:52:00	BOD_{20}	8.3	mg/L	0.6	2.0
5-3	10/06/2017 14:52:00	$CBOD_{20}$	6.2	mg/L	0.6	2.0
5-3	10/06/2017 14:52:00	$CBOD_{20}$-dslv	3.8	mg/L	0.6	2.0
5-3	10/06/2017 14:52:00	BOD_5	3.8	mg/L	0.6	2.0
5-3	10/06/2017 14:52:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
5-3	10/06/2017 14:52:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
5-4	10/06/2017 16:38:00	pH	6.77	pH	0	0.01
5-4	10/06/2017 16:38:00	TEMP	25.2	C	-40.0	-30.0
5-4	10/06/2017 16:38:00	NOx	11.1	mg/L	1.25	2.50
5-4	10/06/2017 16:38:00	NH_3	1.55	mg/L	0.017	0.100
5-4	10/06/2017 16:38:00	TKN	2.5	mg/L	0.050	0.250
5-4	10/06/2017 16:38:00	BOD_{20}	15.2	mg/L	0.6	2.0
5-4	10/06/2017 16:38:00	$CBOD_{20}$-dslv	4.8	mg/L	0.6	2.0
5-4	10/06/2017 16:38:00	BOD_5	6.4	mg/L	0.6	2.0
5-4	10/06/2017 16:38:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
6-1	10/06/2017 08:51:00	TOC	4.62	mg/L	0.082	0.250
6-1	10/06/2017 08:51:00	pH	7.76	pH	0	0.01
6-1	10/06/2017 08:51:00	TEMP	22.4	C	-40.0	-30.0
6-1	10/06/2017 08:51:00	NOx	0.452	mg/L	0.025	0.050
6-1	10/06/2017 08:51:00	NH_3	0.118	mg/L	0.017	0.100
6-1	10/06/2017 08:51:00	TKN	0.504	mg/L	0.050	0.250
6-1	10/06/2017 08:51:00	PO_4	0.087	mg/L	0.009	0.025
6-1	10/06/2017 08:51:00	TP	0.166	mg/L	0.005	0.025
6-1	10/06/2017 08:51:00	TDS	169	mg/L	10.0	25.0
6-1	10/06/2017 08:51:00	TSS	23	mg/L	0.500	6.25
6-1	10/06/2017 08:51:00	BOD_{20}	6.7	mg/L	0.6	2.0
6-1	10/06/2017 08:51:00	$CBOD_{20}$	4.4	mg/L	0.6	2.0
6-1	10/06/2017 08:51:00	$CBOD_5$	3.2	mg/L	0.6	2.0
6-1	10/06/2017 08:51:00	CHL-A	23	mg/m^3	0.25	0.50
6-1	10/06/2017 08:51:00	BOD_5	3.3	mg/L	0.6	2.0
6-1	10/06/2017 08:51:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
6-1	10/06/2017 08:51:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
6-2	10/06/2017 10:01:00	NOx	<0.050	mg/L	0.025	0.050

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
6-2	10/06/2017 10:01:00	pH	7.79	pH	0	0.01
6-2	10/06/2017 10:01:00	TEMP	22.5	C	-40.0	-30.0
6-2	10/06/2017 10:01:00	NH_3	0.146	mg/L	0.017	0.100
6-2	10/06/2017 10:01:00	TKN	0.661	mg/L	0.050	0.250
6-2	10/06/2017 10:01:00	BOD_{20}	9.7	mg/L	0.6	2.0
6-2	10/06/2017 10:01:00	BOD_5	4.6	mg/L	0.6	2.0
6-2	10/06/2017 10:01:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
6-2	10/06/2017 10:01:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
6-3	10/06/2017 15:06:00	TOC	4.4	mg/L	0.082	0.250
6-3	10/06/2017 15:06:00	pH	7.94	pH	0	0.01
6-3	10/06/2017 15:06:00	TEMP	23.4	C	-40.0	-30.0
6-3	10/06/2017 15:06:00	NOx	0.835	mg/L	0.025	0.050
6-3	10/06/2017 15:06:00	NH_3	0.123	mg/L	0.017	0.100
6-3	10/06/2017 15:06:00	TKN	0.268	mg/L	0.050	0.250
6-3	10/06/2017 15:06:00	PO_4	0.141	mg/L	0.009	0.025
6-3	10/06/2017 15:06:00	TP	0.234	mg/L	0.005	0.025
6-3	10/06/2017 15:06:00	TDS	198	mg/L	10.0	25.0
6-3	10/06/2017 15:06:00	TSS	31.3	mg/L	0.667	8.33
6-3	10/06/2017 15:06:00	BOD_{20}	7.2	mg/L	0.6	2.0
6-3	10/06/2017 15:06:00	$CBOD_{20}$	5.4	mg/L	0.6	2.0
6-3	10/06/2017 15:06:00	$CBOD_{20}$-dslv	2.9	mg/L	0.6	2.0
6-3	10/06/2017 15:06:00	CHL-A	26.7	mg/m^3	0.25	0.50
6-3	10/06/2017 15:06:00	BOD_5	3	mg/L	0.6	2.0
6-3	10/06/2017 15:06:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
6-3	10/06/2017 15:06:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
6-4	10/06/2017 16:26:00	NOx	<0.050	mg/L	0.025	0.050
6-4	10/06/2017 16:26:00	NH_3	<0.100	mg/L	0.017	0.100
6-4	10/06/2017 16:26:00	pH	6.99	pH	0	0.01
6-4	10/06/2017 16:26:00	TEMP	22.7	C	-40.0	-30.0
6-4	10/06/2017 16:26:00	TKN	0.313	mg/L	0.050	0.250
6-4	10/06/2017 16:26:00	BOD_{20}	5.5	mg/L	0.6	2.0
6-4	10/06/2017 16:26:00	$CBOD_{20}$-dslv	2.7	mg/L	0.6	2.0
6-4	10/06/2017 16:26:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
6-4	10/06/2017 16:26:00	BOD_5	<2.0	mg/L	0.6	2.0
7-1	10/06/2017 08:00:00	PO_4	<0.025	mg/L	0.009	0.025
7-1	10/06/2017 08:00:00	NOx	<0.050	mg/L	0.025	0.050
7-1	10/06/2017 08:00:00	TOC	4.58	mg/L	0.082	0.250
7-1	10/06/2017 08:00:00	pH	7.42	pH	0	0.01
7-1	10/06/2017 08:00:00	TEMP	21.9	C	-40.0	-30.0
7-1	10/06/2017 08:00:00	NH_3	0.138	mg/L	0.017	0.100
7-1	10/06/2017 08:00:00	TKN	0.567	mg/L	0.050	0.250
7-1	10/06/2017 08:00:00	TP	0.107	mg/L	0.005	0.025
7-1	10/06/2017 08:00:00	TDS	353	mg/L	10.0	25.0
7-1	10/06/2017 08:00:00	TSS	78	mg/L	0.800	10.0
7-1	10/06/2017 08:00:00	BOD_{20}	3.9	mg/L	0.6	2.0
7-1	10/06/2017 08:00:00	$CBOD_{20}$	2.8	mg/L	0.6	2.0
7-1	10/06/2017 08:00:00	CHL-A	6.94	mg/m^3	0.25	0.50
7-1	10/06/2017 08:00:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
7-1	10/06/2017 08:00:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
7-1	10/06/2017 08:00:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
7-1	10/06/2017 08:00:00	BOD_5	<2.0	mg/L	0.6	2.0
7-2	10/06/2017 09:42:00	NOx	<0.050	mg/L	0.025	0.050

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
7-2	10/06/2017 09:42:00	pH	7.63	pH	0	0.01
7-2	10/06/2017 09:42:00	TEMP	22.4	C	-40.0	-30.0
7-2	10/06/2017 09:42:00	NH_3	0.149	mg/L	0.017	0.100
7-2	10/06/2017 09:42:00	TKN	0.522	mg/L	0.050	0.250
7-2	10/06/2017 09:42:00	BOD_{20}	11.1	mg/L	0.6	2.0
7-2	10/06/2017 09:42:00	BOD_5	3.4	mg/L	0.6	2.0
7-2	10/06/2017 09:42:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
7-2	10/06/2017 09:42:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
7-3	10/06/2017 13:38:00	PO_4	<0.025	mg/L	0.009	0.025
7-3	10/06/2017 13:38:00	NOx	<0.050	mg/L	0.025	0.050
7-3	10/06/2017 13:38:00	TKN	<0.250	mg/L	0.050	0.250
7-3	10/06/2017 13:38:00	TOC	4.61	mg/L	0.082	0.250
7-3	10/06/2017 13:38:00	pH	7.6	pH	0	0.01
7-3	10/06/2017 13:38:00	TEMP	23.5	C	-40.0	-30.0
7-3	10/06/2017 13:38:00	NH_3	0.12	mg/L	0.017	0.100
7-3	10/06/2017 13:38:00	TP	0.062	mg/L	0.005	0.025
7-3	10/06/2017 13:38:00	TDS	322	mg/L	10.0	25.0
7-3	10/06/2017 13:38:00	TSS	5.12	mg/L	0.250	3.12
7-3	10/06/2017 13:38:00	BOD_{20}	5	mg/L	0.6	2.0
7-3	10/06/2017 13:38:00	$CBOD_{20}$	4.7	mg/L	0.6	2.0
7-3	10/06/2017 13:38:00	$CBOD_{20}$-dslv	3.1	mg/L	0.6	2.0
7-3	10/06/2017 13:38:00	CHL-A	13.4	mg/m^3	0.25	0.50
7-3	10/06/2017 13:38:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
7-3	10/06/2017 13:38:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
7-3	10/06/2017 13:38:00	BOD_5	<2.0	mg/L	0.6	2.0
7-4	10/06/2017 15:47:00	NOx	<0.050	mg/L	0.025	0.050
7-4	10/06/2017 15:47:00	NH_3	<0.100	mg/L	0.017	0.100
7-4	10/06/2017 15:47:00	TKN	<0.250	mg/L	0.050	0.250
7-4	10/06/2017 15:47:00	pH	7.86	pH	0	0.01
7-4	10/06/2017 15:47:00	TEMP	23.6	C	-40.0	-30.0
7-4	10/06/2017 15:47:00	BOD_{20}	6	mg/L	0.6	2.0
7-4	10/06/2017 15:47:00	$CBOD_{20}$-dslv	3.3	mg/L	0.6	2.0
7-4	10/06/2017 15:47:00	BOD_5	2.2	mg/L	0.6	2.0
7-4	10/06/2017 15:47:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
8-1	10/06/2017 08:35:00	NH_3	<0.100	mg/L	0.017	0.100
8-1	10/06/2017 08:35:00	TOC	4.11	mg/L	0.082	0.250
8-1	10/06/2017 08:35:00	pH	7.51	pH	0	0.01
8-1	10/06/2017 08:35:00	TEMP	22.4	C	-40.0	-30.0
8-1	10/06/2017 08:35:00	NOx	0.511	mg/L	0.025	0.050
8-1	10/06/2017 08:35:00	TKN	0.276	mg/L	0.050	0.250
8-1	10/06/2017 08:35:00	PO_4	0.084	mg/L	0.009	0.025
8-1	10/06/2017 08:35:00	TP	0.16	mg/L	0.005	0.025
8-1	10/06/2017 08:35:00	TDS	191	mg/L	10.0	25.0
8-1	10/06/2017 08:35:00	TSS	34.3	mg/L	0.667	8.33
8-1	10/06/2017 08:35:00	BOD_{20}	4.3	mg/L	0.6	2.0
8-1	10/06/2017 08:35:00	$CBOD_{20}$	2.1	mg/L	0.6	2.0
8-1	10/06/2017 08:35:00	CHL-A	13.2	mg/m^3	0.25	0.50
8-1	10/06/2017 08:35:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
8-1	10/06/2017 08:35:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
8-1	10/06/2017 08:35:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
8-1	10/06/2017 08:35:00	BOD_5	<2.0	mg/L	0.6	2.0
8-2	10/06/2017 10:04:00	TKN	<0.250	mg/L	0.050	0.250

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
8-2	10/06/2017 10:04:00	pH	7.72	pH	0	0.01
8-2	10/06/2017 10:04:00	TEMP	22.3	C	-40.0	-30.0
8-2	10/06/2017 10:04:00	NOx	0.435	mg/L	0.025	0.050
8-2	10/06/2017 10:04:00	NH_3	0.115	mg/L	0.017	0.100
8-2	10/06/2017 10:04:00	BOD_{20}	4.9	mg/L	0.6	2.0
8-2	10/06/2017 10:04:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
8-2	10/06/2017 10:04:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
8-2	10/06/2017 10:04:00	BOD_5	<2.0	mg/L	0.6	2.0
8-3	10/06/2017 14:04:00	NH_3	<0.100	mg/L	0.017	0.100
8-3	10/06/2017 14:04:00	TKN	<0.250	mg/L	0.050	0.250
8-3	10/06/2017 14:04:00	TOC	4.17	mg/L	0.082	0.250
8-3	10/06/2017 14:04:00	pH	7.62	pH	0	0.01
8-3	10/06/2017 14:04:00	TEMP	23	C	-40.0	-30.0
8-3	10/06/2017 14:04:00	NOx	0.395	mg/L	0.025	0.050
8-3	10/06/2017 14:04:00	PO_4	0.071	mg/L	0.009	0.025
8-3	10/06/2017 14:04:00	TP	0.158	mg/L	0.005	0.025
8-3	10/06/2017 14:04:00	TDS	201	mg/L	10.0	25.0
8-3	10/06/2017 14:04:00	TSS	36	mg/L	0.667	8.33
8-3	10/06/2017 14:04:00	BOD_{20}	6.3	mg/L	0.6	2.0
8-3	10/06/2017 14:04:00	$CBOD_{20}$	5	mg/L	0.6	2.0
8-3	10/06/2017 14:04:00	$CBOD_{20}$-dslv	3.5	mg/L	0.6	2.0
8-3	10/06/2017 14:04:00	CHL-A	24	mg/m^3	0.25	0.50
8-3	10/06/2017 14:04:00	BOD_5	2.3	mg/L	0.6	2.0
8-3	10/06/2017 14:04:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
8-3	10/06/2017 14:04:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
8-4	10/06/2017 16:08:00	NH_3	<0.100	mg/L	0.017	0.100
8-4	10/06/2017 16:08:00	TKN	<0.250	mg/L	0.050	0.250
8-4	10/06/2017 16:08:00	pH	7.98	pH	0	0.01
8-4	10/06/2017 16:08:00	TEMP	23.3	C	-40.0	-30.0
8-4	10/06/2017 16:08:00	NOx	0.485	mg/L	0.025	0.050
8-4	10/06/2017 16:08:00	BOD_{20}	7.6	mg/L	0.6	2.0
8-4	10/06/2017 16:08:00	$CBOD_{20}$-dslv	3.5	mg/L	0.6	2.0
8-4	10/06/2017 16:08:00	BOD_5	3	mg/L	0.6	2.0
8-4	10/06/2017 16:08:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
9-1	10/06/2017 10:39:00	PO_4	<0.025	mg/L	0.009	0.025
9-1	10/06/2017 10:39:00	NOx	<0.050	mg/L	0.025	0.050
9-1	10/06/2017 10:39:00	NH_3	<0.100	mg/L	0.017	0.100
9-1	10/06/2017 10:39:00	TKN	<0.250	mg/L	0.050	0.250
9-1	10/06/2017 10:39:00	pH	7.5	pH	0	0.01
9-1	10/06/2017 10:39:00	TEMP	22.4	C	-40.0	-30.0
9-1	10/06/2017 10:39:00	TP	0.053	mg/L	0.005	0.025
9-1	10/06/2017 10:39:00	BOD_{20}	4	mg/L	0.6	2.0
9-1	10/06/2017 10:39:00	$CBOD_{20}$	3.4	mg/L	0.6	2.0
9-1	10/06/2017 10:39:00	$CBOD_{20}$-dslv	2.1	mg/L	0.6	2.0
9-1	10/06/2017 10:39:00	$CBOD_5$	2.2	mg/L	0.6	2.0
9-1	10/06/2017 10:39:00	BOD_5	2.3	mg/L	0.6	2.0
9-1	10/06/2017 10:39:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
9-3	10/06/2017 16:35:00	PO_4	<0.025	mg/L	0.009	0.025
9-3	10/06/2017 16:35:00	NOx	<0.050	mg/L	0.025	0.050
9-3	10/06/2017 16:35:00	NH_3	<0.100	mg/L	0.017	0.100
9-3	10/06/2017 16:35:00	TKN	<0.250	mg/L	0.050	0.250
9-3	10/06/2017 16:35:00	pH	7.69	pH	0	0.01

Sample ID	Date Time	Parameter	Result	Units	Detection Limit	Reporting Limit
9-3	10/06/2017 16:35:00	TEMP	23.5	C	-40.0	-30.0
9-3	10/06/2017 16:35:00	TP	0.06	mg/L	0.005	0.025
9-3	10/06/2017 16:35:00	BOD_{20}	8	mg/L	0.6	2.0
9-3	10/06/2017 16:35:00	$CBOD_{20}$	8	mg/L	0.6	2.0
9-3	10/06/2017 16:35:00	$CBOD_{20}$-dslv	4.1	mg/L	0.6	2.0
9-3	10/06/2017 16:35:00	$CBOD_5$	3.7	mg/L	0.6	2.0
9-3	10/06/2017 16:35:00	BOD_5	3.8	mg/L	0.6	2.0
9-3	10/06/2017 16:35:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
10-1	10/06/2017 11:09:00	NH_3	<0.100	mg/L	0.017	0.100
10-1	10/06/2017 11:09:00	pH	7.53	pH	0	0.01
10-1	10/06/2017 11:09:00	TEMP	22.6	C	-40.0	-30.0
10-1	10/06/2017 11:09:00	NOx	0.67	mg/L	0.025	0.050
10-1	10/06/2017 11:09:00	TKN	0.299	mg/L	0.050	0.250
10-1	10/06/2017 11:09:00	PO_4	0.11	mg/L	0.009	0.025
10-1	10/06/2017 11:09:00	TP	0.199	mg/L	0.005	0.025
10-1	10/06/2017 11:09:00	BOD_{20}	4.5	mg/L	0.6	2.0
10-1	10/06/2017 11:09:00	BOD_5	2	mg/L	0.6	2.0
10-1	10/06/2017 11:09:00	$CBOD_{20}$	<2.0	mg/L	0.6	2.0
10-1	10/06/2017 11:09:00	$CBOD_{20}$-dslv	<2.0	mg/L	0.6	2.0
10-1	10/06/2017 11:09:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
10-1	10/06/2017 11:09:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
10-3	10/06/2017 17:03:00	NH_3	<0.100	mg/L	0.017	0.100
10-3	10/06/2017 17:03:00	TKN	<0.250	mg/L	0.050	0.250
10-3	10/06/2017 17:03:00	pH	7.82	pH	0	0.01
10-3	10/06/2017 17:03:00	TEMP	23.6	C	-40.0	-30.0
10-3	10/06/2017 17:03:00	NOx	0.778	mg/L	0.025	0.050
10-3	10/06/2017 17:03:00	PO_4	0.138	mg/L	0.009	0.025
10-3	10/06/2017 17:03:00	TP	0.217	mg/L	0.005	0.025
10-3	10/06/2017 17:03:00	BOD_{20}	5	mg/L	0.6	2.0
10-3	10/06/2017 17:03:00	$CBOD_{20}$	4.9	mg/L	0.6	2.0
10-3	10/06/2017 17:03:00	$CBOD_{20}$-dslv	3.5	mg/L	0.6	2.0
10-3	10/06/2017 17:03:00	$CBOD_5$	<2.0	mg/L	0.6	2.0
10-3	10/06/2017 17:03:00	$CBOD_5$-dslv	<2.0	mg/L	0.6	2.0
10-3	10/06/2017 17:03:00	BOD_5	<2.0	mg/L	0.6	2.0

APPENDIX B: SONDE DATA STATISTICS

Water temperature, pH, conductivity, and dissolved oxygen were measured on ten-minute intervals at seven locations along the Caney River using YSI Autologger sondes (Figure 4-2). In the following tables in this section, sonde results have been arranged from most upstream to most downstream (Table B-1. Caney River water quality sonde data: water temperature, Table B-2. Caney River water quality sonde data: pH, Table B-3. Caney River water quality sonde data: conductivity, and Table B-4. Caney River water quality sonde data: dissolved oxygen). The full suite of sonde data is available in a separate Excel workbook (Attachment B).

Table B-1. Caney River water quality sonde data: water temperature

Sonde Site	Location	Water Temperature (°C), 9/5/2017—9/10/2017				Water Temperature (°C), 10/2/2017—10/6/2017			
		Min	Max	Mean	Range	Min	Max	Mean	Range
1	5 mi US of intake	21.18	29.20	24.46	8.02	21.61	22.99	22.31	1.38
2	Dam pool	20.15	27.18	24.78	7.03	22.05	24.26	22.53	2.21
3	US of WWTF	20.18	30.81	24.60	10.63	22.04	23.28	22.38	1.24
4	SE Adams Blvd	21.00	28.05	24.80	7.05	22.19	23.84	22.70	1.65
5	Hillcrest Road	18.14	26.88	24.17	8.75	22.26	23.58	22.82	1.31
6	W 2400 Road	16.24	26.96	24.32	10.72	22.26	23.62	22.94	1.36
7	Highway 75	18.66	25.69	24.35	7.03	23.03	23.25	23.18	0.22

Table B-2. Caney River water quality sonde data: pH

Sonde Site	Location	pH (unitless), 9/5/2017—9/10/2017				pH (unitless), 10/2/2017—10/6/2017			
		Min	Max	Mean	Range	Min	Max	Mean	Range
1	5 mi US of intake	7.01	8.65	7.86	1.64	7.91	8.27	8.07	0.36
2	Dam pool	6.05	8.29	7.70	2.24	7.58	8.93	7.88	1.35
3	US of WWTF	6.48	8.42	7.80	1.94	7.63	8.02	7.75	0.39
4	SE Adams Blvd	7.29	8.62	8.02	1.33	7.86	8.22	7.96	0.36
5	Hillcrest Road	7.19	8.88	8.31	1.69	7.70	7.95	7.81	0.25
6	W 2400 Road	6.81	8.89	8.54	2.08	7.70	8.22	7.87	0.52
7	Highway 75	6.54	9.02	8.76	2.48	7.45	7.80	7.68	0.35

Table B-3. Caney River water quality sonde data: conductivity

Sonde Site	Location	Conductivity (uS/cm), 9/5/2017—9/10/2017				Conductivity (uS/cm), 10/2/2017—10/6/2017			
		Min	Max	Mean	Range	Min	Max	Mean	Range
1	5 mi US of intake	396.40	420.00	409.41	23.60	317.60	383.50	325.20	65.90
2	Dam pool	310.30	318.10	312.56	7.80	322.90	440.10	370.88	117.20
3	US of WWTF	317.00	330.90	324.50	13.90	327.40	450.70	390.99	123.30
4	SE Adams Blvd	352.80	378.80	362.29	26.00	327.00	471.40	421.80	144.40
5	Hillcrest Road	379.90	420.30	392.90	40.40	323.70	468.10	427.69	144.40
6	W 2400 Road	398.40	422.80	410.19	24.40	352.20	469.50	428.38	117.30
7	Highway 75	369.40	397.90	383.90	28.50	444.60	466.00	450.32	21.40

Table B-4. Caney River water quality sonde data: dissolved oxygen

Sonde Site	Location	Dissolved Oxygen (mg/l), 9/5/2017—9/10/2017				Dissolved Oxygen (mg/l), 10/2/2017—10/6/2017			
		Min	Max	Mean	Range	Min	Max	Mean	Range
1	5 mi US of intake	4.94	9.49	6.31	4.55	6.52	8.78	7.51	2.26
2	Dam pool	0.12	7.91	3.52	7.79	5.98	16.35	7.70	10.37
3	US of WWTF	6.57	9.46	7.62	2.89	7.54	8.82	7.85	1.28
4	SE Adams Blvd	6.93	10.47	8.47	3.54	7.13	9.00	7.64	1.87
5	Hillcrest Road	7.07	13.24	9.32	6.17	6.80	8.15	7.38	1.35
6	W 2400 Road	7.55	14.97	10.81	7.42	6.81	9.74	7.50	2.93
7	Highway 75	7.55	14.92	11.67	7.37	6.90	8.34	7.54	1.44

APPENDIX C: SYNOPTIC SAMPLING RESULTS

Synoptic sampling data that was measured in the field longitudinally from upstream to downstream during multiple days of each monitoring trip are detailed below Figure C-1. Synoptic average DO observed during longitudinal sampling: field trip 1, Figure C- 2. Synoptic average DO observed during longitudinal sampling: field trip 2 Table C-1. Longitudinal synoptic field sampling: September monitoring trip, and Table C-2. Longitudinal synoptic field sampling: October monitoring trip). Note that water temperature, DO concentration, and DO saturation were measured on two probes which were averaged for comparison efforts during model calibration. The data helped the modeling team understand variation in water quality between primary water quality stations and sonde locations.

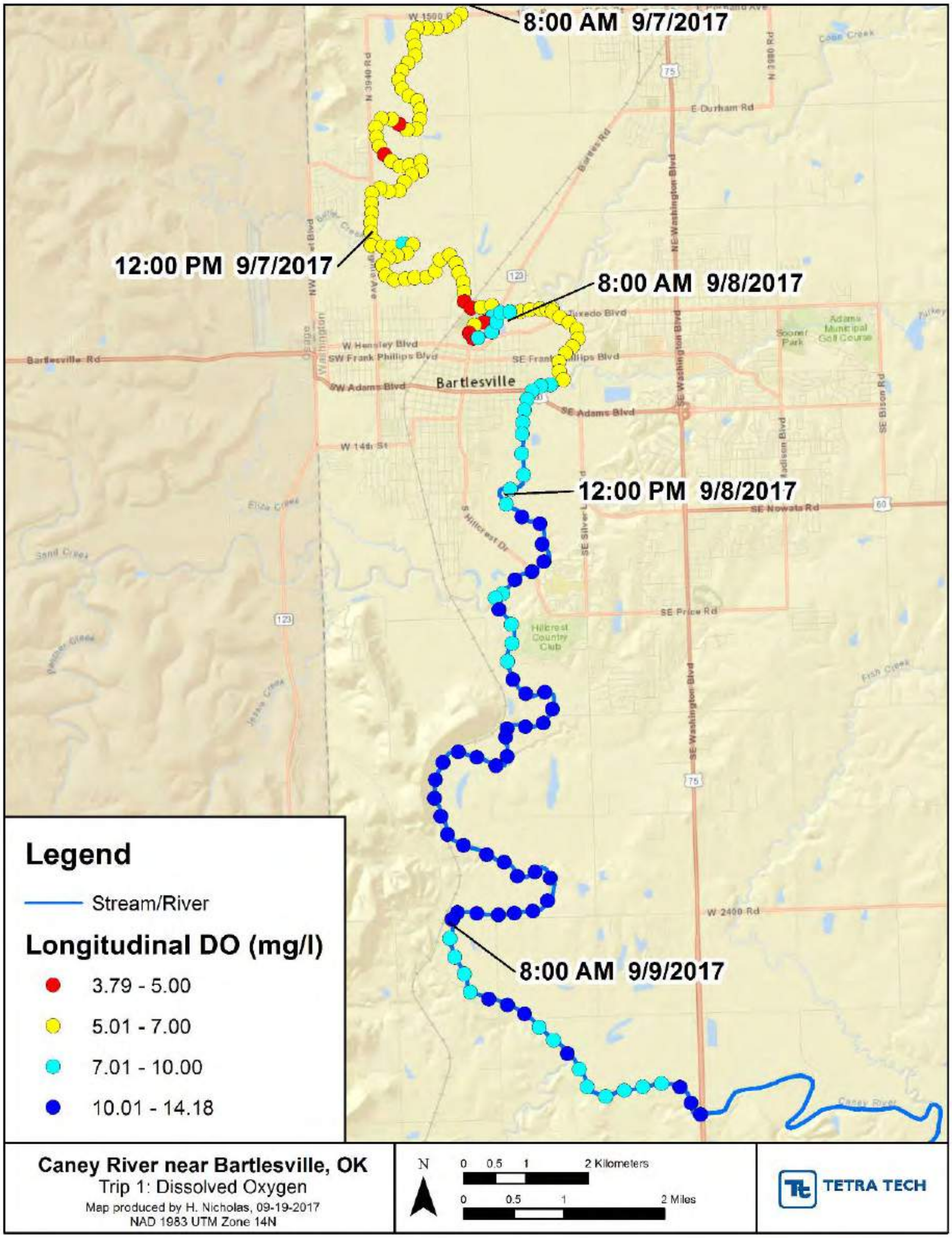

Figure C-1. Synoptic average DO observed during longitudinal sampling: field trip 1

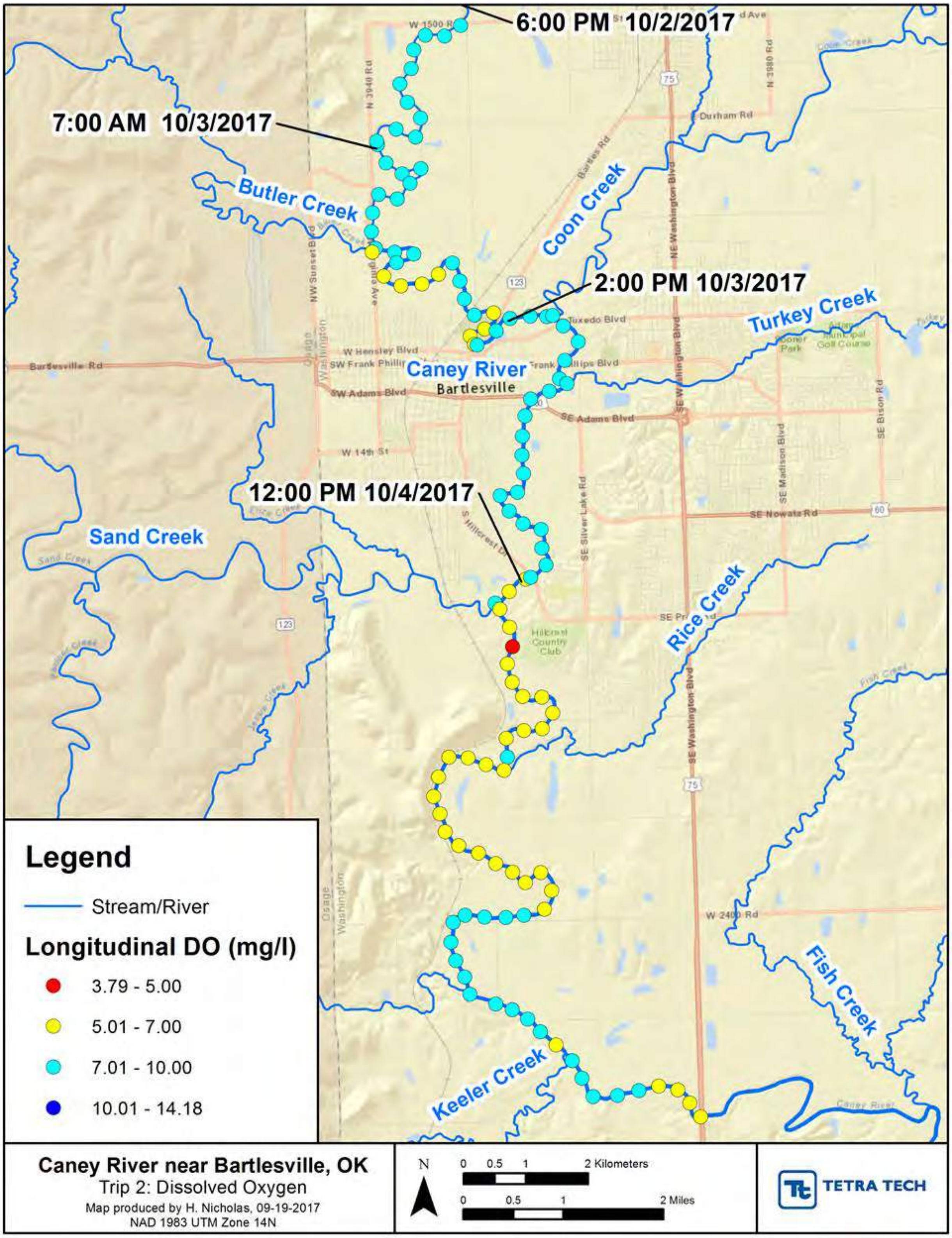

Figure C- 2. Synoptic average DO observed during longitudinal sampling: field trip 2

Table C-1. Longitudinal synoptic field sampling: September monitoring trip

Point	Date	Time	TEMP (°C) 1	TEMP (°C) 2	DOSAT (%) 1	DOSAT (%) 2	DO (mg/L) 1	DO (mg/L) 2	COND (µS)	pH
1	9/7/2017	8:23	23.6	23.7	70.6	72.2	5.98	6.07	401.7	7.82
2	9/7/2017	8:29	23.6	23.7	67	68.4	5.68	5.75	404.3	7.78
3	9/7/2017	8:34	23.5	23.7	64.8	66.6	5.5	5.6	404.6	7.76
4	9/7/2017	8:40	23.6	23.8	65.5	67.3	5.55	5.65	401.8	7.77
5	9/7/2017	8:46	23.5	23.7	64.5	66.1	5.47	5.56	402.2	7.75
6	9/7/2017	8:51	23.5	23.6	66.4	68.1	5.64	5.75	401.1	7.77
7	9/7/2017	8:55	23.5	23.6	68.7	70.5	5.83	5.93	402.1	7.79
8	9/7/2017	9:02	23.4	23.6	66.2	67.9	5.63	5.73	402.9	7.76
9	9/7/2017	9:09	23.5	23.6	65.1	67.4	5.53	5.68	400.2	7.75
10	9/7/2017	9:18	23.5	23.7	61.4	63.2	5.21	5.32	394.8	7.72
11	9/7/2017	9:21	23.5	23.7	62.5	64.4	5.3	5.42	394.5	7.73
12	9/7/2017	9:25	23.6	23.8	61.5	63.1	5.21	5.3	399.6	7.71
13	9/7/2017	9:28	23.6	23.8	63.5	65.2	5.38	5.48	412.8	7.71
14	9/7/2017	9:34	23.6	23.8	60.2	62.2	5.12	5.22	413.8	7.68
15	9/7/2017	9:38	23.5	23.7	61.6	63.1	5.22	5.31	406.4	7.7
16	9/7/2017	9:42	23.5	23.7	61.4	63.2	5.22	5.32	393.4	7.71
17	9/7/2017	9:46	23.6	23.7	59	60.6	5.01	5.1	392.1	7.69
18	9/7/2017	9:50	23.7	23.9	60.6	62.6	5.13	5.25	391	7.7
19	9/7/2017	9:54	23.9	24	59.4	61.7	5.02	5.17	392.6	7.69
20	9/7/2017	9:58	23.9	24.1	57.1	60.1	4.81	5.03	393.1	7.67
21	9/7/2017	10:14	23.7	23.8	59.3	61	5.01	5.12	392.4	7.71
22	9/7/2017	10:21	23.8	23.9	60.2	61.8	5.08	5.18	393.2	7.71
23	9/7/2017	10:25	23.6	23.8	60.8	62.9	5.15	5.3	395.5	7.69
24	9/7/2017	10:29	23.5	23.6	62	64.1	5.27	5.41	395.4	7.7
25	9/7/2017	10:33	23.2	23.5	60.6	62.8	5.17	5.31	392.8	7.69
26	9/7/2017	10:38	23.1	23.3	57.4	59.7	4.91	5.06	387.8	6.67
27	9/7/2017	10:43	23	23.2	60.9	62.9	5.22	5.34	388.2	7.71
28	9/7/2017	10:47	23	23.2	64.2	66.8	5.51	5.68	389.8	7.74
29	9/7/2017	10:51	23.1	23.5	70.5	73.6	6.03	6.22	387.2	7.81
30	9/7/2017	10:57	23.5	23.7	69.8	71.6	5.93	6.03	381.1	7.8
31	9/7/2017	11:02	23.6	23.8	65.4	68.5	5.55	5.76	375.4	7.76
32	9/7/2017	11:06	23.6	23.9	66.6	68	5.64	5.71	374	7.77
33	9/7/2017	11:09	23.6	23.8	69.4	71.1	5.88	5.97	375.1	7.79
34	9/7/2017	11:13	23.7	23.9	67.5	69.3	5.71	5.81	373.2	7.79
35	9/7/2017	11:17	23.7	24	69.6	72.5	5.89	6.07	371.4	7.81
36	9/7/2017	11:21	23.8	24	74.2	75.7	6.24	6.33	370.3	7.87
37	9/7/2017	11:48	23.5	23.8	66.9	70	5.67	5.88	368	7.77
38	9/7/2017	11:53	23.5	24.1	72.4	74.7	6.14	6.24	367.2	7.83
39	9/7/2017	11:57	23.5	23.7	71.9	73.7	6.1	6.2	365.6	7.82

Point	Date	Time	TEMP (°C) 1	TEMP (°C) 2	DOSAT (%) 1	DOSAT (%) 2	DO (mg/L) 1	DO (mg/L) 2	COND (µS)	pH
40	9/7/2017	12:00	23.6	23.7	74.6	77.2	6.31	6.5	364.5	7.88
41	9/7/2017	12:05	23.3	23.6	68.3	71.7	5.82	6.03	359.6	7.8
42	9/7/2017	12:09	23.3	23.6	69.8	71.7	5.94	6.05	357.1	7.83
43	9/7/2017	12:22	22	22.3	69.1	17.9	6.03	6.21	351.6	7.76
44	9/7/2017	12:29	23.6	23.7	69.2	70.7	5.86	5.96	353	7.8
45	9/7/2017	12:54	23.4	23.7	66.8	70.4	5.67	5.93	351.3	7.77
46	9/7/2017	12:58	24.1	24.5	83.9	91.1	7.05	7.54	347.8	7.95
47	9/7/2017	13:02	24.8	25.1	83.1	85.8	6.88	7.04	343.6	7.93
48	9/7/2017	13:06	24.4	24.7	4.1	78.1	6.15	6.46	342.8	7.8
49	9/7/2017	13:10	24	24.5	66.1	75.6	5.56	6.25	341	7.74
50	9/7/2017	13:14	24.2	24.5	74	76.2	6.2	6.33	338.3	7.82
51	9/7/2017	13:19	24.2	24.5	71.1	73.4	5.96	6.1	336.7	7.8
52	9/7/2017	13:30	24.1	24.4	66.9	69.7	5.62	5.8	333.8	7.75
53	9/7/2017	13:35	24.3	24.6	73.1	76.9	6.11	6.37	333.2	7.83
54	9/7/2017	13:40	24.7	25.1	77.2	82.1	6.41	6.73	327.1	7.84
55	9/7/2017	13:43	25.1	25	78	81.4	6.32	6.67	324.7	7.92
56	9/7/2017	13:47	24.7	25	68	74.5	5.6	6.29	321.6	7.77
57	9/7/2017	13:58	24.6	24.9	62.5	66.4	5.2	5.48	321.3	7.74
58	9/7/2017	14:03	24.6	25	63.7	67.6	5.28	5.56	320	7.73
59	9/7/2017	14:20	25.4	25.8	70.8	81.9	5.81	6.62	319.8	7.76
60	9/7/2017	14:25	25.1	25.6	59.2	70	4.82	5.7	318.7	7.66
61	9/7/2017	14:30	25.1	25.7	73.6	91.4	6.06	7.42	319.4	7.83
62	9/7/2017	14:40	25.2	25.4	74.8	82.3	6.15	6.71	317.8	7.85
63	9/7/2017	14:44	24.9	25.6	74.3	84.8	6.15	6.91	317.4	7.86
64	9/7/2017	14:49	24.6	24.9	50.2	60.8	4.17	5.06	316.4	7.68
65	9/7/2017	14:52	24.7	25.1	49.8	60.5	4.13	4.95	317.4	7.62
66	9/7/2017	14:56	25.4	25.9	81.9	88.7	6.72	7.25	317.4	7.89
67	9/7/2017	15:05	24.9	25.6	54.6	88	4.51	7.3	317.8	7.72
68	9/7/2017	15:12	25.5	25.7	115.6	102	9.08	8.13	316.9	8.07
69	9/7/2017	15:19	24.9	25.2	51.4	61.3	4.25	5.02	317.4	7.7
70	9/7/2017	15:23	25.2	25.3	66.3	67.9	5.45	5.54	316.9	7.75
71	9/7/2017	15:27	24.7	24.9	46	46.1	3.79	3.8	317.1	7.64
72	9/7/2017	15:32	25.5	25.4	59.9	59.5	4.92	4.85	318.9	7.71
73	9/7/2017	16:11	N/A	25.4	N/A	88.5	N/A	7.24	N/A	N/A
74	9/7/2017	16:21	N/A	25.7	N/A	93.3	N/A	7.57	N/A	N/A
75	9/7/2017	16:26	N/A	25.7	N/A	96.6	N/A	7.83	N/A	N/A
76	9/7/2017	16:32	N/A	26.2	N/A	102.2	N/A	8.21	N/A	N/A
77	9/7/2017	16:35	N/A	27.1	N/A	110.2	N/A	8.72	N/A	N/A
78	9/8/2017	8:29	N/A	23.3	N/A	80.3	N/A	6.81	N/A	N/A
79	9/8/2017	8:35	N/A	23.8	N/A	80.9	N/A	6.8	N/A	N/A
80	9/8/2017	8:38	N/A	23.4	N/A	78.2	N/A	6.62	N/A	N/A

Point	Date	Time	TEMP (°C) 1	TEMP (°C) 2	DOSAT (%) 1	DOSAT (%) 2	DO (mg/L) 1	DO (mg/L) 2	COND (µS)	pH
81	9/8/2017	8:42	N/A	23.2	N/A	74.8	N/A	6.36	N/A	N/A
82	9/8/2017	8:45	N/A	23.3	N/A	74.2	N/A	6.3	N/A	N/A
83	9/8/2017	8:49	N/A	23.4	N/A	73.7	N/A	6.24	N/A	N/A
84	9/8/2017	8:55	N/A	23.3	N/A	73.5	N/A	6.23	N/A	N/A
85	9/8/2017	9:12	N/A	23.3	N/A	73.8	N/A	6.26	N/A	N/A
86	9/8/2017	9:47	23.2	23.4	67.7	75.7	5.78	6.4	375	7.56
87	9/8/2017	9:54	23.3	23.5	69.5	72.1	5.93	6.09	380	7.49
88	9/8/2017	10:02	23.3	23.4	72.3	75.6	6.17	6.4	380	7.54
89	9/8/2017	10:11	23.4	23.5	72.5	76.7	6.17	6.48	380	7.53
90	9/8/2017	10:15	23.2	23.4	72.1	72.8	6.15	6.16	375	7.54
91	9/8/2017	10:20	23.7	238	83	81.4	6.95	6.84	368	7.6
92	9/8/2017	10:25	23.8	24	77.7	81.6	6.55	6.83	369	7.59
93	9/8/2017	10:29	23.89	24.1	80.4	82.9	6.77	6.93	368	7.64
94	9/8/2017	10:37	24.1	24.3	85.2	89.4	7.16	7.45	368	7.67
95	9/8/2017	10:41	24.2	24.5	92.6	96	7.74	7.96	368	7.77
96	9/8/2017	10:49	24.3	24.5	97.2	101	8.13	8.38	369	7.83
97	9/8/2017	10:57	24.2	24.4	89.4	100.4	7.58	8.34	369	7.85
98	9/8/2017	11:06	24.3	24.5	105.9	107	8.86	8.87	369	7.82
99	9/8/2017	11:36	24.6	24.8	111.7	114.4	9.29	9.43	369	8.01
100	9/8/2017	11:43	24.7	24.9	112.6	116.9	9.36	9.63	370	8.01
101	9/8/2017	11:52	24.4	24.7	102	112.4	8.52	9.3	371	8.06
102	9/8/2017	11:59	23.7	24	97.2	102.4	8.21	8.58	371	7.9
103	9/8/2017	12:07	23.2	23.5	94.7	101.7	8.08	8.58	370	7.91
104	9/8/2017	12:17	24.1	24.7	105.9	114.3	8.94	9.45	370	8.12
105	9/8/2017	12:25	24.1	24.5	166.4	170.8	13.9	14.2	366	8.58
106	9/8/2017	12:31	25.4	25.5	134.3	133.7	11.01	10.92	375	8.43
107	9/8/2017	12:38	24.4	24.9	134	133.4	11.12	10.98	371	8.46
108	9/8/2017	12:52	24.3	24.6	119.8	122.6	10.03	10.15	372	8.2
109	9/8/2017	13:36	24.9	25.2	126.2	128.9	10.43	10.56	371	8.3
110	9/8/2017	13:44	25.07	25.3	121.3	126	10.02	10.3	372	8.33
111	9/8/2017	13:51	23.9	24.5	116.2	121.4	9.76	10.06	372	8.3
112	9/8/2017	13:54	24.2	25.1	116.4	116	9.76	9.52	411	7.89
113	9/8/2017	14:07	24.7	25.2	125.6	134.7	10.41	11.01	382	8.29
114	9/8/2017	14:14	24.4	25.3	109.5	116.6	9.14	9.53	387	8.08
115	9/8/2017	14:21	25.5	25.7	114.9	123.6	9.47	10.02	394	8.15
116	9/8/2017	14:28	24.9	26	111.1	118.1	9.17	9.52	398	8.07
117	9/8/2017	14:35	24.2	25.7	122	130.9	10.24	10.61	394	8.18
118	9/8/2017	14:41	24.9	25.6	121.3	127.7	10.08	10.39	392	8.2
119	9/8/2017	14:46	25.2	25.6	122.3	127.6	10.06	10.38	392	8.17
120	9/8/2017	14:52	25.2	25.5	140.5	147.4	11.65	12.01	393	8.3
121	9/8/2017	15:04	26	26.3	137.9	142.9	11.16	11.47	393	8.37

Point	Date	Time	TEMP (°C) 1	TEMP (°C) 2	DOSAT (%) 1	DOSAT (%) 2	DO (mg/L) 1	DO (mg/L) 2	COND (μS)	pH
122	9/8/2017	15:11	25.99	26.4	131.5	142.4	10.65	11.48	397	8.39
123	9/8/2017	15:19	25.6	26	140.1	147.2	11.4	11.88	400	8.39
124	9/8/2017	15:25	25.6	25.7	139.2	145.4	11.39	11.8	400	8.34
125	9/8/2017	15:36	25.3	25.9	139.2	150.5	11.21	12.18	403	8.36
126	9/8/2017	15:51	24.96	25.2	133.3	137.2	11.01	11.25	405	8.23
127	9/8/2017	16:02	24.7	25	123.5	130.5	10.22	10.72	407	8.24
128	9/8/2017	16:09	24.3	24.6	115.5	125.2	9.69	10.38	410	8.23
129	9/8/2017	16:13	24.5	24.7	118.3	123.2	9.85	10.18	411	8.14
130	9/8/2017	16:19	24.9	25.3	122.5	134.7	10.11	11.01	411	8.25
131	9/8/2017	16:33	25.1	25.3	122.4	136.1	10.11	11.13	411	8.3
132	9/8/2017	16:39	25.3	25.7	139.4	146.8	11.44	11.92	409	8.42
133	9/8/2017	16:47	25.2	25.8	134.6	154.9	11.14	12.55	408	8.49
134	9/8/2017	16:53	24.7	24.9	127.6	135.6	10.59	11.16	408	8.43
135	9/8/2017	17:03	24.8	25.1	133.7	138.5	11.07	11.34	409	8.26
136	9/8/2017	17:09	25.4	25.6	130.8	148.1	10.74	12.02	409	8.46
137	9/8/2017	17:18	25.5	25.7	137	157	11.19	12.73	409	8.55
138	9/8/2017	17:26	25.6	26	163.2	175.8	13.3	14.19	406	8.65
139	9/8/2017	17:31	25.36	25.6	164	179.4	133.34	14.6	403	8.7
140	9/8/2017	17:41	24.9	25.1	162.1	168.7	13.41	13.84	401	8.65
141	9/8/2017	0:00	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
142	9/8/2017	17:54	25.1	25.3	154.7	159	12.75	13	400	8.56
143	9/8/2017	18:02	25.6	25.8	147.6	158.7	12.03	12.85	399	8.68
144	9/8/2017	18:07	25.8	25.9	159.6	164.8	13	13.32	398	8.72
145	9/8/2017	18:15	25.9	26.1	152.5	163.1	12.36	13.14	398	8.75
146	9/8/2017	18:21	26.2	26.4	153	173.6	12.32	13.91	397	8.77
147	9/8/2017	18:26	26.2	26.4	169.3	183.2	13.69	14.67	396	8.81
148	9/9/2017	7:33	22.8	23	86.1	94.3	7.41	7.99	411	8.13
149	9/9/2017	7:41	22.9	23.1	93.3	99.5	8	8.47	413	8.17
150	9/9/2017	7:47	23.1	23.2	93.6	101.2	8.01	8.6	411	8.17
151	9/9/2017	7:52	23.5	23.6	103.9	111.6	8.83	9.41	409	8.25
152	9/9/2017	8:00	23.9	24.1	113.4	122	9.56	10.2	407	8.33
153	9/9/2017	8:05	23.8	23.9	117	124.6	9.8	10.45	406	8.32
154	9/9/2017	8:11	23.4	23.6	123.1	133.2	10.42	11.23	404	8.38
155	9/9/2017	8:27	23.1	23.3	115	126.9	9.82	10.77	405	8.32
156	9/9/2017	8:35	23.2	23.4	108.2	118	9.24	9.99	406	8.27
157	9/9/2017	8:41	23.1	23.3	109.9	121.8	9.4	10.33	405	8.32
158	9/9/2017	8:47	23	23.2	115.9	125.8	9.93	10.69	404	8.39
159	9/9/2017	9:04	22.9	23.1	104.1	117.3	8.95	9.99	405	8.4
160	9/9/2017	9:10	22.6	22.8	104.6	114.3	9.01	9.78	405	8.39
161	9/9/2017	9:16	22.7	22.7	111.4	116.2	9.62	9.97	405	8.37
162	9/9/2017	9:28	22.4	22.6	104.3	111.5	9.04	9.58	405	8.42

Point	Date	Time	TEMP (°C) 1	TEMP (°C) 2	DOSAT (%) 1	DOSAT (%) 2	DO (mg/L) 1	DO (mg/L) 2	COND (µS)	pH
163	9/9/2017	9:36	22.5	22.7	109.2	113.8	9.46	9.77	405	8.48
164	9/9/2017	9:42	22.6	22.8	110.3	117.8	9.52	10.09	404	8.52
165	9/9/2017	9:48	22.7	22.9	118.1	124.4	10.15	10.63	403	8.6
166	9/9/2017	9:56	22.8	23	118.3	127.8	10.17	10.9	401	8.64
167	9/9/2017	10:04	23.2	23.4	125.6	127.9	10.73	10.83	401	8.64

Table C-2. Longitudinal synoptic field sampling: October monitoring trip

Point	Date	Time	TEMP (°C) 1	TEMP (°C) 2	DOSAT (%) 1	DOSAT (%) 2	DO (mg/L) 1	DO (mg/L) 2	COND (µS)	pH
1	10/2/2017	18:24	21.9	22.2	96.3	97.4	8.43	8.43	326.9	8.12
2	10/2/2017	18:31	21.8	22	96.5	97.4	8.46	8.46	326.6	7.99
3	10/2/2017	18:35	21.9	22.1	97.7	98.2	8.54	8.52	326.7	8.01
4	10/2/2017	18:41	21.9	22.1	97.5	98	8.53	8.5	327.7	8.02
5	10/2/2017	18:46	21.8	22	96.6	97.2	8.48	8.45	329.8	8.01
6	10/2/2017	18:51	21.7	21.9	96.6	97.2	8.48	8.46	333.6	8
7	10/2/2017	18:55	21.8	22	99.7	100.5	8.74	8.75	340.6	8.08
8	10/2/2017	19:01	21.7	22	101.6	102	8.92	8.87	358.6	8.1
9	10/2/2017	19:07	21.6	21.8	99.8	100.2	8.79	8.74	370.8	8.05
10	10/2/2017	19:13	21.5	21.7	99	99.5	8.74	8.7	388.4	8.03
11	10/2/2017	19:52	21.5	21.6	99.5	99.7	8.78	8.73	409.8	8.06
12	10/3/2017	7:18	21.6	21.7	84.1	85.1	7.4	7.44	332.3	7.97
13	10/3/2017	7:27	21.6	21.7	83.3	84.2	7.33	7.36	334.3	7.92
14	10/3/2017	7:32	21.6	21.7	83.1	84	7.32	7.35	336.7	7.91
15	10/3/2017	7:38	21.5	21.7	83.6	84.4	7.37	7.38	342.9	7.9
16	10/3/2017	7:50	21.5	21.6	85.1	86	7.51	7.54	359.5	7.91
17	10/3/2017	7:56	21.4	21.5	84.5	85.5	7.47	7.5	386.5	7.88
18	10/3/2017	8:02	21.4	21.5	84	85.1	7.43	7.47	400.1	7.87
19	10/3/2017	8:10	21.4	21.5	83.4	84.2	7.38	7.39	410.3	7.86
20	10/3/2017	8:15	21.4	21.5	83.8	84.7	7.4	7.43	422.4	7.87
21	10/3/2017	8:23	21.5	21.6	84.7	85.6	7.47	7.5	433.5	7.89
22	10/3/2017	8:28	21.5	21.7	66.2	67.4	5.85	5.82	448.4	7.69
23	10/3/2017	8:36	21.6	21.7	86.1	87.1	7.58	7.61	443.1	7.92
24	10/3/2017	8:43	21.7	21.8	87.5	88.1	7.69	7.69	459.4	7.93
25	10/3/2017	8:48	21.6	21.8	85.1	86	7.48	7.5	464	7.88
26	10/3/2017	8:58	21.6	21.8	78.1	79	6.87	6.89	465.4	7.81
27	10/3/2017	9:08	21.7	21.8	73.6	75.1	6.46	6.55	454.3	7.75
28	10/3/2017	9:16	21.7	21.9	74.6	78	6.55	6.79	430.1	7.7
29	10/3/2017	9:24	21.7	21.9	72.5	75.5	6.37	6.58	426.1	7.67
30	10/3/2017	9:30	22	22.1	80.9	82.9	7.06	7.2	429	7.76

Point	Date	Time	TEMP (°C) 1	TEMP (°C) 2	DOSAT (%) 1	DOSAT (%) 2	DO (mg/L) 1	DO (mg/L) 2	COND (µS)	pH
31	10/3/2017	9:35	22	22.2	82.1	83.1	7.15	7.19	431.8	7.78
32	10/3/2017	9:41	22.3	22.4	83.5	84.8	7.25	7.32	436.8	7.82
33	10/3/2017	9:47	22.4	22.6	79.9	87.2	6.92	7.5	439.7	7.76
34	10/3/2017	9:54	22.3	22.4	75	76.2	6.5	6.57	440.6	7.73
35	10/3/2017	10:00	22.4	22.5	71.8	74.7	6.23	6.43	448.8	7.69
36	10/3/2017	10:04	22.4	22.5	70.5	71.9	6.11	6.19	452.2	7.68
37	10/3/2017	10:08	22.4	22.6	69.2	71.6	5.99	6.16	454.1	7.66
38	10/3/2017	10:19	22.3	22.4	85.2	86.2	7.4	7.44	453.8	7.78
39	10/3/2017	10:48	22.3	22.4	86.2	87.2	7.48	7.52	454.4	7.77
40	10/3/2017	13:53	22.8	22.9	91.7	92.5	7.89	7.91	450.3	7.92
41	10/3/2017	14:19	23	23.2	92.2	93.2	7.9	7.92	460.7	7.82
42	10/3/2017	14:24	22.8	23	90.5	91.8	7.78	7.83	461.2	7.8
43	10/3/2017	14:27	22.8	23.2	97.5	103.6	8.37	8.8	510	7.89
44	10/3/2017	14:36	22.8	23	91.1	92.2	7.84	7.86	464	7.83
45	10/3/2017	14:43	23	23.2	95.5	96.3	8.19	8.17	465.6	7.87
46	10/3/2017	14:51	22.9	23.1	92.8	93.9	7.97	8	466	7.85
47	10/3/2017	14:58	22.8	23	92	93.1	7.91	7.94	465.6	7.85
48	10/3/2017	15:05	2.3	22.7	84.2	89.7	7.33	7.7	367.2	7.8
49	10/3/2017	15:11	22.8	23	91	92.1	7.83	7.85	465	7.84
50	10/3/2017	15:18	22.8	23	90.7	91.8	7.8	7.83	464.9	7.85
51	10/3/2017	15:40	22.9	23	92.4	93.4	7.94	7.97	464.3	7.91
52	10/3/2017	15:48	22.9	23.1	92.8	93.8	7.97	7.98	464.7	7.89
53	10/3/2017	15:53	22.9	23.1	92.7	93.7	7.95	7.98	464.6	7.89
54	10/3/2017	15:59	23	23.1	92.3	93.3	7.92	7.95	494.6	7.89
55	10/3/2017	16:06	23	23.2	90.9	91.8	7.79	7.8	464.3	7.89
56	10/3/2017	16:10	22.9	23.1	88.8	89.8	7.62	7.64	464.9	7.86
57	10/3/2017	16:15	22.8	23	86.3	87.3	7.42	7.45	466.6	7.83
58	10/3/2017	16:24	22.8	23	85.6	86.2	7.36	7.35	468.7	7.89
59	10/3/2017	16:31	22.6	22.8	82.9	84.1	7.15	7.2	473.1	7.8
60	10/3/2017	16:37	22.6	22.8	82.2	83.2	7.09	7.13	472.4	7.79
61	10/3/2017	16:42	22.6	22.8	81.4	82.5	7.03	7.06	473.1	7.77
62	10/3/2017	16:49	22.6	22.8	82.6	83.6	7.13	7.16	473.3	7.79
63	10/4/2017	11:38	22.6	22.7	79.4	80.7	6.85	6.92	454.5	7.85
64	10/4/2017	11:44	22.7	22.8	79.2	80.7	6.83	6.91	455.8	7.82
65	10/4/2017	11:50	22.8	22.9	86.1	86.9	7.4	7.3	491.9	7.79
66	10/4/2017	11:55	22.7	22.9	78.7	80.3	6.79	6.86	459.1	7.77
67	10/4/2017	12:02	22.8	22.9	78.8	80.3	6.78	6.86	462.1	7.77
68	10/4/2017	12:07	22.8	22.9	78.2	79.9	6.72	6.83	465.3	7.77
69	10/4/2017	12:12	22.8	23	72	79.5	6.7	6.78	465.8	776
70	10/4/2017	12:17	22.9	23	77.8	79.2	6.68	6.76	467.6	7.76
71	10/4/2017	12:22	22.9	23	77.2	78.6	6.63	6.71	468.6	7.74

Point	Date	Time	TEMP (°C) 1	TEMP (°C) 2	DOSAT (%) 1	DOSAT (%) 2	DO (mg/L) 1	DO (mg/L) 2	COND (µS)	pH
72	10/4/2017	12:27	22.9	23	77.5	79.1	6.66	6.74	468.2	7.76
73	10/4/2017	12:34	22.9	23	77.9	79.4	6.69	6.78	467.9	7.77
74	10/4/2017	12:51	22.8	N/A	80.7	N/A	6.94	N/A	466	7.79
75	10/4/2017	12:57	22.8	N/A	80.9	N/A	6.96	N/A	464.8	7.8
76	10/4/2017	13:03	22.8	N/A	81	N/A	6.97	N/A	464.9	7.8
77	10/4/2017	13:08	22.8	N/A	81.4	N/A	7.01	N/A	464.9	7.79
78	10/4/2017	13:13	22.8	N/A	81.2	N/A	6.99	N/A	464.4	7.8
79	10/4/2017	13:20	22.6	N/A	80.5	N/A	6.94	N/A	460.1	7.79
80	10/4/2017	13:26	22.7	N/A	81.2	N/A	7	N/A	463.4	7.8
81	10/4/2017	13:32	22.7	N/A	80.7	N/A	6.96	N/A	464.3	7.78
82	10/4/2017	13:39	22.6	N/A	80.5	N/A	6.94	N/A	463.5	7.78
83	10/4/2017	13:45	22.6	N/A	80.8	N/A	6.97	N/A	463	7.79
84	10/4/2017	13:51	22.6	N/A	80	N/A	6.9	N/A	463.8	7.79
85	10/4/2017	13:56	22.7	N/A	79.1	N/A	6.82	N/A	466	7.79
86	10/4/2017	14:17	22.6	N/A	79	N/A	6.82	N/A	464.9	7.8
87	10/4/2017	14:23	22.6	N/A	79.5	N/A	6.86	N/A	460.5	7.79
88	10/4/2017	14:33	22.7	N/A	79.6	N/A	6.87	N/A	462.7	7.81
89	10/4/2017	14:38	22.7	N/A	80	N/A	6.9	N/A	461	7.81
90	10/4/2017	14:45	22.7	N/A	79.7	N/A	6.87	N/A	462.2	7.81
91	10/4/2017	14:51	22.7	N/A	79.7	N/A	6.86	N/A	461.9	7.8
92	10/4/2017	14:57	22.8	N/A	79.4	N/A	6.82	N/A	462.3	7.8
93	10/4/2017	15:02	22.8	N/A	79.4	N/A	6.83	N/A	461.6	7.81
94	10/4/2017	15:18	22.8	N/A	82.5	N/A	7.1	N/A	454.3	7.85
95	10/4/2017	15:25	22.8	N/A	81.7	N/A	7.03	N/A	456.2	7.84
96	10/4/2017	15:31	22.7	N/A	84.6	N/A	7.29	N/A	433.1	7.87
97	10/4/2017	15:38	22.8	N/A	84.3	N/A	7.25	N/A	447.8	7.84
98	10/4/2017	15:47	22.8	N/A	83	N/A	7.14	N/A	448.4	7.87
99	10/4/2017	16:47	22.8	N/A	83.9	N/A	7.21	N/A	446.9	7.89
100	10/4/2017	16:53	22.9	N/A	84.5	N/A	7.26	N/A	446.3	7.9
101	10/4/2017	16:57	22.9	N/A	85.3	N/A	7.32	N/A	447.5	7.9
102	10/4/2017	17:04	22.9	N/A	83.9	N/A	7.2	N/A	450.1	7.91
103	10/4/2017	17:13	22.9	N/A	83.8	N/A	7.19	N/A	445.4	7.89
104	10/4/2017	17:19	23	N/A	84.2	N/A	7.22	N/A	442.7	7.9
105	10/4/2017	17:32	23	N/A	83.5	N/A	7.16	N/A	447.7	7.89
106	10/4/2017	17:37	22.9	N/A	82.2	N/A	7.06	N/A	444.4	7.87
107	10/4/2017	17:45	21.7	N/A	61.5	N/A	5.42	N/A	237.9	7.56
108	10/4/2017	17:54	21.5	N/A	83.6	N/A	7.38	N/A	106.6	7.72
109	10/4/2017	17:59	21.5	N/A	81	N/A	7.15	N/A	106.1	7.73
110	10/4/2017	18:10	21.5	N/A	80.6	N/A	7.12	N/A	107.8	7.72
111	10/4/2017	18:20	21.5	N/A	80.7	N/A	7.12	N/A	109.2	7.73
112	10/4/2017	18:27	21.5	N/A	80.5	N/A	7.11	N/A	109.7	7.73

Point	Date	Time	TEMP (°C) 1	TEMP (°C) 2	DOSAT (%) 1	DOSAT (%) 2	DO (mg/L) 1	DO (mg/L) 2	COND (µS)	pH
113	10/4/2017	18:33	21.5	N/A	79.2	N/A	6.98	N/A	109.5	7.75
114	10/4/2017	18:39	21.5	N/A	79.4	N/A	7	N/A	109.8	7.77
115	10/4/2017	18:43	21.5	N/A	78	N/A	6.89	N/A	110.7	7.77
116	10/4/2017	18:51	21.4	N/A	75.8	N/A	6.7	N/A	113.2	7.79

APPENDIX D: SEDIMENT OXYGEN DEMAND REPORT

The sediment oxygen demand report starts on the following page.

APPENDIX E: REAERATION REPORT

The reaeration report starts on the following page.

APPENDIX F: QUAL2K MODEL PARAMETERIZATION

Below are tables which summarize the QUAL2K model parameterization for rates and kinetics which were held constant for calibration, corroboration, and scenario applications (Table E-1 and Table E-2).

Table F-3. QUAL2K model "Light and heat" tab parameterization

Parameter	Value	Unit
Photosynthetically Available Radiation	0.2	
Background light extinction	0.2	/m
Linear chlorophyll light extinction	0.0088	1/m-(ugA/L)
Nonlinear chlorophyll light extinction	0.054	1/m-(ugA/L)2/3
ISS light extinction	0.052	1/m-(mgD/L)
Detritus light extinction	0.174	1/m-(mgD/L)
Solar shortwave radiation model		
Atmospheric attenuation model for solar	Bras	
Bras solar parameter (used if Bras solar model is selected)		
atmospheric turbidity coefficient (2=clear, 5=smoggy, default=2)	2	
Downwelling atmospheric longwave IR radiation		
atmospheric longwave emissivity model	Koberg	
Evaporation and air convection/conduction		
wind speed function for evaporation and air convection/conduction	Adams 2	
Sediment heat parameters		
Sediment thermal thickness	15	cm
Sediment thermal diffusivity	0.005	cm^2/s
Sediment density	1.6	g/cm^3
Water density	1	g/cm^3
Sediment heat capacity	0.4	cal/(g °C)
Water heat capacity	1	cal/(g °C)
Sediment diagenesis model		
Compute SOD and nutrient fluxes	No	

Table F-2. QUAL2K model "Rates" tab parameterization

Parameter	Value	Units
Stoichiometry:		
Carbon	40	gC
Nitrogen	7.2	gN
Phosphorus	1	gP
Dry weight	100	gD
Chlorophyll	1	gA
Inorganic suspended solids:		
Settling velocity	0.5	m/d
Oxygen:		

Parameter	Value	Units
Reaeration model	Tsivoglou-Neal	
User reaeration coefficient α	3.93	
User reaeration coefficient β	0.5	
User reaeration coefficient γ	1.5	
Temp correction	1.024	
Reaeration wind effect	Wanninkhof	
O2 for carbon oxidation	2.69	gO_2/gC
O2 for NH4 nitrification	4.57	gO_2/gN
Oxygen inhib model CBOD oxidation	Exponential	
Oxygen inhib parameter CBOD oxidation	0.60	L/mgO2
Oxygen inhib model nitrification	Exponential	
Oxygen inhib parameter nitrification	0.60	L/mgO2
Oxygen enhance model denitrification	Exponential	
Oxygen enhance parameter denitrification	0.60	L/mgO2
Oxygen inhib model phyto resp	Exponential	
Oxygen inhib parameter phyto resp	0.60	L/mgO2
Oxygen enhance model bot alg resp	Exponential	
Oxygen enhance parameter bot alg resp	0.60	L/mgO2
Slow CBOD:		
Hydrolysis rate	0	/d
Temp correction	1.07	
Oxidation rate	0.05	/d
Temp correction	1.047	
Fast CBOD:		
Oxidation rate	0.15	/d
Temp correction	1.047	
Organic N:		
Hydrolysis	0.1	/d
Temp correction	1.07	
Settling velocity	0	m/d
Ammonium:		
Nitrification	0.01	/d
Temp correction	1.07	
Nitrate:		
Denitrification	0.1	/d
Temp correction	1.07	
Sed denitrification transfer coeff	0	m/d
Temp correction	1.07	
Organic P:		
Hydrolysis	0.03	/d

Parameter	Value	Units
Temp correction	1.07	
Settling velocity	0	m/d
Inorganic P:		
Settling velocity	0.8	m/d
Inorganic P sorption coefficient	1000	L/mgD
Sed P oxygen attenuation half sat constant	1	mgO_2/L
Phytoplankton:		
Max Growth rate	1	/d
Temp correction	1.07	
Respiration rate	0.01	/d
Temp correction	1.07	
Excretion rate	0.01	/d
Temp correction	1.07	
Death rate	0.1	/d
Temp correction	1.07	
External Nitrogen half sat constant	100	ugN/L
External Phosphorus half sat constant	10	ugP/L
Inorganic carbon half sat constant	1.30E-05	moles/L
Light model	Half saturation	
Light constant	250	langleys/d
Ammonia preference	5	ugN/L
Subsistence quota for nitrogen	0.05	mgN/mgA
Subsistence quota for phosphorus	0.05	mgP/mgA
Maximum uptake rate for nitrogen	5	mgN/mgA/d
Maximum uptake rate for phosphorus	1	mgP/mgA/d
Internal nitrogen half sat constant	0.9	mgN/mgA
Internal phosphorus half sat constant	0.13	mgP/mgA
Settling velocity	0.01	m/d
Bottom Algae:		
Growth model	Zero-order	
Max Growth rate	200	mgA/m2/d or /d
Temp correction	1.07	
First-order model carrying capacity	1000	mgA/m2
Respiration rate	0.05	/d
Temp correction	1.07	
Excretion rate	0.01	/d
Temp correction	1.07	
Death rate	0.01	/d
Temp correction	1.07	
External nitrogen half sat constant	300	ugN/L

Parameter	Value	Units
External phosphorus half sat constant	100	ugP/L
Inorganic carbon half sat constant	1.30E-05	moles/L
Light model	Half saturation	
Light constant	100	langleys/d
Ammonia preference	25	ugN/L
Subsistence quota for nitrogen	0.72	mgN/mgA
Subsistence quota for phosphorus	0.1	mgP/mgA
Maximum uptake rate for nitrogen	72	mgN/mgA/d
Maximum uptake rate for phosphorus	5	mgP/mgA/d
Internal nitrogen half sat constant	0.9	mgN/mgA
Internal phosphorus half sat constant	0.13	mgP/mgA
Detritus (POM):		
Dissolution rate	0.05	/d
Temp correction	1.07	
Fraction of dissolution to fast CBOD	1	
Settling velocity	0.1	m/d
pH:		
Partial pressure of carbon dioxide	412.63	ppm

ATTACHMENT 1: MONITORING STUDY PLAN

The “Monitoring Study Plan: Caney River TMDL Study For Chickasaw Wastewater Treatment Plant” starts on the following page.

ATTACHMENT 2: SONDE DATA

The Caney River sonde data for September and October starts on the following page.

Appendix E

Letters of Local Support

bartlesville
chamber of commerce
CONNECTED/CREATIVE/VIBRANT

201 SW Keeler
Bartlesville, OK 74003

T 918-336-8708
F 918-337-0216
E Reception@Bartlesville.com

December 15, 2016

Katharine Dahm
Bureau of Reclamation
Water Resources and Planning
P.O. Box 25007
Denver, Colorado 80225

Dear Ms. Dahm:

The Bartlesville Regional Chamber of Commerce would like to offer our support for the City of Bartlesville's application to the Bureau of Reclamation for funding a study to determine the feasibility of water reuse.

The Chamber represents the business and non-profit community in the Bartlesville, Oklahoma area. We have a membership of 750 in four communities in northeast Oklahoma. The availability of water is vital to the continued growth of the businesses and industries in our region. We have partnered with the City of Bartlesville on water issues in the past and have held a seat on the Bartlesville Water Resources Committee since it was formed in 2001.

Good water supply is essential to the welfare and growth of Bartlesville, and projects like this study present a wonderful opportunity to examine innovative solutions to our water supply issues.

Sincerely,

Sherri Wilt

Sherri Wilt, President/CEO
Bartlesville Regional Chamber of Commerce

bartlesville
development authority
CONNECTED / CREATIVE / VIBRANT

201 SW Keeler Ave
Bartlesville, OK 74003

T 918 337 8086

RECEIVED
DEC 27 20[illegible]
By MK

December 15, 2016

Katharine Dahm
Bureau of Reclamation
Water Resources and Planning
P.O. Box 25007
Denver, Colorado 80225

Dear Ms. Dahm:

On behalf of the Bartlesville Development Authority, I am pleased to support the City of Bartlesville's application to fund a feasibility study for water reuse.

The BDA facilitates the recruitment, retention and expansion of primary industry jobs, and new destination retail businesses for the Bartlesville Area. We support programs that protect and enhance economic development, and a reliable long term water supply is vital to our economic future.

Bartlesville does not have enough water from current sources to meet future demand or to handle a drought of record. We must diversify our water supply portfolio. New laws in Oklahoma allow Bartlesville to examine water reuse as a viable option for long-term water supply. The BDA supports the city's efforts to perform such studies and requests your consideration of their application.

Sincerely,

David Wood, President & CEO
Bartlesville Development Authority

Bartlesville Fire Department

Protecting Community Life, Health, Property and the Environment

By Delivering Quality Life and Fire Safety Services.

December 27, 2016

Katharine Dahm

Bureau of Reclamation

Water Resources and Planning

P.O. Box 25007

Denver, Colorado 80225

Dear Ms. Dahm:

I am pleased to support the City of Bartlesville's application to the Bureau of Reclamation for funding a study to determine the feasibility of water reuse.

Water supply is an important subject to the Bartlesville Fire Department. We must provide the public with an effective level of fire protection, which means we need a system that is reliable and delivers adequate amounts of water to meet fire flow requirements.

The water system also has a direct impact on fire insurance rates in Bartlesville. The Insurance Services Office classifies communities based on their ability to provide fire protection. It is based on many factors, and the community's water supply accounts for nearly half of their evaluation.

Unfortunately, Bartlesville does not have enough water to meet future demands. Therefore, the Bartlesville Fire Department supports any efforts to study and consider new options.

Sincerely,

Chief John Banks

Bartlesville Fire Department

December 28, 2016

Katharine Dahm
Bureau of Reclamation
Water Resources and Planning
P.O. Box 25007
Denver, Colorado 80225

Dear Ms. Dahm:

On behalf of the City of Dewey, I am pleased to support the City of Bartlesville's application to fund a feasibility study for water reuse.

The City of Dewey, which has a population of approximately 3,500 persons, is located north of Bartlesville and purchases all of our potable water from the City of Bartlesville. The availability of water is critical to the welfare and continued growth of our City. We support programs that diversify and ensure a reliable long term water supply.

The City of Dewey supports Bartlesville's study to examine innovative solutions such as water reuse to secure long term water supply for our community and requests your consideration of their application.

Sincerely,

Kevin Trease
Dewey City Manager

RURAL WATER DISTRICT NO. 2 WASHINGTON COUNTY, OK
P.O. Box 420 Ochelata, OK 74051 Phone: 918-535-2302 800-448-3264 FAX: 918-535-2981

December 27, 2016

Katharine Dahm
Bureau of Reclamation
Water Resources and Planning
P.O. Box 25007
Denver, Colorado 80225

Dear Ms. Dahm:

On behalf of the Washington County Rural Water District #2, I am pleased to support the City of Bartlesville's application to fund a feasibility study for water reuse.

The Washington County Rural Water District #2, which serves a population of approximately 3,000 persons, purchases our potable water from the City of Bartlesville. The availability of water is critical to the welfare and continued growth of our water district. We support programs that diversify and ensure a reliable long term water supply.

The Washington County Rural Water District #2 supports Bartlesville's study to examine innovative solutions such as water reuse to secure long term water supply for our water district and requests your consideration of their application.

Sincerely,

David L. Anderson
Manager, Washington County Rural Water District #2

OSAGE CO. RURAL WATER DISTRICT NO. 1

P. O. Box 420 Ochelata, OK 74051 Phone: 918-535-2302 800-448-3264 FAX: 918-535-2981

December 27, 2016

Katharine Dahm
Bureau of Reclamation
Water Resources and Planning
P.O. Box 25007
Denver, Colorado 80225

Dear Ms. Dahm:

On behalf of the Osage Rural Water District #1, I am pleased to support the City of Bartlesville's application to fund a feasibility study for water reuse.

The Osage County Rural Water District #1, which serves a population of approximately 1,100 persons, purchases our potable water from the City of Bartlesville. The availability of water is critical to the welfare and continued growth of our water district. We support programs that diversify and ensure a reliable long term water supply.

The Osage County Rural Water District #1 supports Bartlesville's study to examine innovative solutions such as water reuse to secure long term water supply for our water district and requests your consideration of their application.

Sincerely,

David L. Anderson
Manager, Osage County Rural Water District #1

RURAL WATER DISTRICT NO. 5 WASHINGTON COUNTY, OK
P.O. Box 420 Ochelata, OK 74051 Phone: 918-535-2302 800-448-3264 FAX: 918-535-2981

December 27, 2016

Katharine Dahm
Bureau of Reclamation
Water Resources and Planning
P.O. Box 25007
Denver, Colorado 80225

Dear Ms. Dahm:

On behalf of the Washington County Rural Water District #5, I am pleased to support the City of Bartlesville's application to fund a feasibility study for water reuse.

Washington County Rural Water District #5, which serves a population of approximately 1,000 persons, purchases our potable water from the City of Bartlesville. The availability of water is critical to the welfare and continued growth of our water district. We support programs that diversify and ensure a reliable long term water supply.

Washington County Rural Water District #5 supports Bartlesville's study to examine innovative solutions such as water reuse to secure long term water supply for our water district and requests your consideration of their application.

Sincerely,

David L. Anderson
Manager, Washington Co. Water District #5

Appendix F

Funding Mechanism

Copy of City of Bartlesville Ordinance 3468

Published in the Bartlesville Examiner Enterprise on June 19, 2016

ORDINANCE 3468

AN ORDINANCE AMENDING CHAPTER 20 OF THE BARTLESVILLE MUNICIPAL CODE PERTAINING TO WATER AND WASTEWATER RATES, BILLING, AND FEES FOR SERVICES IN THE WATER AND WASTEWATER DEPARTMENTS EFFECTIVE JULY 1, 2016.

WHEREAS, it is necessary to increase the City of Bartlesville's water and wastewater operating fees to pay for operations, maintenance, and capital projects for fiscal year 2016-17 and beyond; and

NOW, THEREFORE, BE IT ORDAINED BY THE CITY COUNCIL OF THE CITY OF BARTLESVILLE, OKLAHOMA; that

Effective July 1, 2016, the following sections of Chapter 20 of the Bartlesville Municipal Code be and the same is hereby amended as follows:

Sec. 20-56. - Rates and billing.

(a) The following charges shall apply for water delivered from the city municipal water system to consumers within the corporate limits of the city:

(1) For accounts with meters smaller than 3 inches:

	Charge per 1,000 gallons of billable flow for bills starting:				
Usage	**July 1, 2016**	**July 1, 2017**	**July 1, 2018**	**July 1, 2019**	**July 1, 2020**
0-2,000	0.00	0.00	0.00	0.00	0.00
2,001-10,000	3.45	3.64	3.84	4.05	4.27
10,001-25,000	3.80	4.00	4.22	4.46	4.70
25,001-50,000	4.14	4.37	4.61	4.86	5.12
>50,000	4.49	4.73	4.99	5.27	5.55

(2) For accounts with meters 3 inches or larger:

Charge per 1,000 gallons of billable flow for bills starting:				
July 1, 2016	**July 1, 2017**	**July 1, 2018**	**July 1, 2019**	**July 1, 2020**
3.45	3.64	3.84	4.05	4.27

Sec. 20-57. - Service charges.

(a) In addition to the charges set out in Section 20-56, each customer shall be billed:

(1) A monthly meter service charge for each meter based on the size of the meter, as follows:

	Monthly charge for bills starting:				
Meter Size	**July 1, 2016**	**July 1, 2017**	**July 1, 2018**	**July 1, 2019**	**July 1, 2020**
Less than 1"	12.74	13.31	13.91	14.54	15.19
1"	31.85	33.28	34.78	36.35	37.98
1.5"	63.70	66.55	69.55	72.70	75.95
2"	101.92	106.48	111.28	116.32	121.52
3"	191.10	199.65	208.65	218.10	227.85
4"	318.50	332.75	347.75	363.50	379.75
6"	637.00	665.50	695.50	727.00	759.50
8"	1,019.20	1,064.80	1,112.80	1,163.20	1,215.20

(2) A water capital investment fee of $0.85 per 1,000 gallons of billable flow.

(b) Consumers located outside the municipal limits shall pay rates provided in sections 20-56 and 20-57 multiplied by one and twenty-five hundredths (1.25) or as provided by a contract for water service with the city.

(c) A late payment penalty shall be applied to each account which has not paid the bill in full within twenty (20) days of the date of the bill.

Sec. 20-251. - Charges, designated.

(a) When sewer service is furnished to water consumers, the sewer service charge shall be based upon metered water consumption as shown by the water meters on the various dwelling or business units in accordance with this title, or upon the sewage discharge as shown by the direct sewage metering system where such system has been installed and approved by the city, or upon a fair and reasonable determination of the percentage of metered water returned to the sewer collection system. Such percentage determination shall be established for a specific, defined category of users.

(b) Monthly bills for sewer service shall be based upon water consumption except as otherwise provided in subsection (f) of this section. Rates are hereby established for all users in three (3) parts:

(1) Unit charge for flows based upon water consumption; and

(2) A billing charge based upon the overhead and administrative costs of billing and accounting.

(3) An additional unit charge also based upon water consumption and restricted for use in wastewater system improvements, titled "Wastewater Capital Investment Fee."

(c) The billing charge shall be applied to all users equally regardless of the volume of metered water.

(d) Pursuant to paragraph (b) of this section, the following charges shall be applied to all users of the sanitary sewer system:

(1) Unit charge per one thousand (1,000) gallons of billable flow

Charge per 1,000 gallons of billable flow for bills starting:				
July 1, 2016	**July 1, 2017**	**July 1, 2018**	**July 1, 2019**	**July 1, 2020**
3.09	3.14	3.19	3.24	3.29

(2) Billing charge per customer per month

Monthly charge for bills starting:				
July 1, 2016	**July 1, 2017**	**July 1, 2018**	**July 1, 2019**	**July 1, 2020**
4.33	5.73	7.13	8.53	9.93

(3) Wastewater capital investment fee per one thousand (1,000) gallons of billable flow

Charge per 1,000 gallons of billable flow for bills starting:				
July 1, 2016	**July 1, 2017**	**July 1, 2018**	**July 1, 2019**	**July 1, 2020**
1.44	1.66	1.91	2.20	2.53

(e) Sewer service charges for residential customers on bills issued on approximately January 1, February 1, March 1, and April 1, shall be based on a volume equal to the metered water consumption on the same bill. Sewer service charges for bills issued the remaining eight (8) months of the year shall reflect a charge for sewer volume equal to the lowest of:

(1) The actual water metered for the same period; or

(2) One and two-tenths (1.2) times the average water consumption for the lowest three (3) of the four (4) bills issued on approximately January 1, February 1, March 1, and April 1, except that if the actual consumption in any of those months is lower than two thousand (2,000) gallons, then two thousand (2,000) gallons shall be used in computing the average and in the case of a new resident where no average for the customer is available, the average shall be assumed to be seven thousand (7,000) gallons.

(f) Sanitary sewer service charges for all customers other than residential customers shall be based on a volume equal to the metered water consumption reflected on the same bill unless there is installed at the customer's expense a meter for measuring the sewer volume discharged into the sanitary sewer system.

(g) Consumers located outside the municipal limits shall pay rates provided in sections 20-251 multiplied by one and twenty-five hundredths (1.25) or as provided by a contract for wastewater service with the city.

APPROVED BY THE CITY COUNCIL AND SIGNED BY THE MAYOR OF THE CITY OF BARTLESVILLE THIS 6th DAY OF June , 2016.

Thomas A. Gorman
Mayor
City of Bartlesville
401 S. Johnstone Ave.
Bartlesville, OK 74003

CITY OF BARTLESVILLE OFFICIAL SEAL OKLAHOMA

ATTEST:

City Clerk

VOTE:
DR. CALLAHAN (aye) no
VICE MAYOR COPELAND (aye) no
MR. KANE (aye) no
MR. LOCKIN (aye) no
MAYOR GORMAN (aye) no