STAFF REPORT **_WATER UTILITIES DEPARTMENT_**

TO: Ed Gordon, City Manager

FROM: Terry Lauritsen, P.E. Director of Water Utilities

Cc: Mike Bailey, CFO
Micah Siemers, P.E., Director of Engineering

DATE: September 26, 2017

SUBJECT: Discuss and take action on a recommendation from the Sanitary Sewer Improvement Oversight Committee regarding the Updated Wastewater Treatment Plan including a Reuse Feasibility Study

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Since the early 1990's the City has been investing significantly to upgrade the wastewater system to eliminate bypass events (sewage backs up and flows out of the wastewater collection system, typically during rain events where storm water infiltrates into the collection system and overloads it). To date, the City has spent over $40 million to upgrade the wastewater system to mitigate these bypass locations. As we have conducted studies to identify and fix the capacity issues, ultimately this would require improvements to the treatment plant and several pump stations that send wastewater to the plant. In 2010, a facility plan was completed that identified these improvements. This plan looked at two scenarios, either expand the existing plant or build a new secondary plant south of town. Based on the City's growth patterns, comparable capital costs between the options, and long-term desire to move the treatment plant away from populated areas, Council selected the secondary plant south of town option. However in 2012, the state enacted a law that opened up the possibility for water reuse, which utilizes treated wastewater for either potable (drinking) or non-potable (irrigation or process) applications. We have a unique set up in that our wastewater plant is roughly ¼ mile downstream of a potable raw water intake on the Caney River, which is shown below.

The City has been pursuing additional potable raw water to stabilize our long-term supply since a drought in 2001 revealed vulnerabilities to our water supply system. While several studies have been completed that identified sources of raw water, none of them investigated a reuse scenario. For the City, our reuse scenario would take treated wastewater, pump it upstream of the potable raw water intake, recapture this water, and then treat and utilize it for potable applications. In May 2016, Council approved a contract with Tetra Tech to re-evaluate the 2010 facility plan and investigate the feasibility of water reuse.

On September 5, 2017 the Sanitary Sewer Improvement Oversight Committee (SSIOC) received a presentation on the results of the study, including the options to either expand the existing wastewater treatment plant or build a secondary wastewater treatment plant south of town. These results were also presented at the Sept. 5 Council meeting. The SSIOC voted unanimously recommending Council to approve the expand the existing wastewater treatment plant option. Staff also recommends the expand the existing wastewater treatment plant option and requests authorization to proceed on the project as required for implementation.

Amendment to

WWTP Facility Plan and Reuse Feasibility Study

Technical Memorandum No. 1

Population, Flow, and Waste Load

For:

CITY OF BARTLESVILLE

401 S. Johnstone Avenue

Bartlesville, OK 74003

Prepared By:

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CA 2388, EXP. 06/30/19

Tetra Tech Project No. 200-11458-16002

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ACRONYMS/ABBREVIATIONS

Name	Abbreviation
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
MGD	Million gallons per Day
NRCS	National Resources Conservation Services
O&M	Operation & Maintenance
ODEQ	Oklahoma Department of Environmental Quality
OWRB	Oklahoma Water Resources Board
PAS	Planning Assistance to States
QA/QC	Quality Assurance / Quality Control
RWD	Rural Water District
TL-WTP	Ted D. Lockin Water Treatment Plant (City of Bartlesville)
USACE	US Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USCOE	US Corps of Engineers
USEPA	U.S. Environmental Protection Agency
WQ	Water Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant
CWWTP	Chickasaw Wastewater Treatment Plant
FEB	Flow Equalization Basin
GPM	Gallons per Minute
IFAS	Integrated Fixed Film Activated Sludge

Name	Abbreviation
LS	Lift Station
MGD	Million Gallons per Day
MBBR	Moving-Bed Bioreactor
TM	Technical Memorandum
VFD	Variable Frequency Drive

1.0 INTRODUCTION

Technical Memorandum No. 1 (TM-1) summarizes the evaluation of historical wastewater flows and wasteloads and the development of values used to establish the projected flows and wasteloads necessary to analyze the existing facilities and size of the improvements.

2.0 POPULATION PROJECTION UPDATE

2.1 PLANNING PERIOD

A planning period of 35 years was considered appropriate by City staff for planning the wastewater treatment system. The planning period was established to begin in 2015 and extend through 2050. Historical data from 2001 to 2015 was utilized to develop flow projections for the planning period.

2.2 POPULATION PROJECTION

The population distributions were determined using 2010 census tract data as shown in Figure 1. It was assumed that the population estimates for the sanitary sewer service area are essentially the same as that of the City of Bartlesville. An average annual growth of 0.37% was applied to the total 2010 population to project the total service area population in 5-year intervals from 2015 to the projected planning year, 2050. The total additional population was allocated to the major basins based on percent growth rates provided by the City's planning department. Table TM1.1 displays the final population projection for each basin.

Table TM1.1 - Current and Projected Population

		2015 Population	2050 Population		
Collection System Basins	**Sub-Basin No.**	**Population Distribution**	**Population Distribution**	**Additional Population**	**Percent of Total Growth**
Chickasaw	C01-C07	7,294	8,299	1,005	20%
Tuxedo	T01-T06	13,974	15,632	1,658	33%
Woodland	T07-T10	1,930	2,181	251	5%
Shawnee					
North	S01-S06	5,119	5,823	704	14%
Hillcrest	S07-S08	720	821	101	2%
South	S09 & S14	631	732	101	2%
Rice Creek	S10-S15	6,748	7,954	1,206	24%
Totals:		**36,416**	**41,441**	**5,025**	**100%**

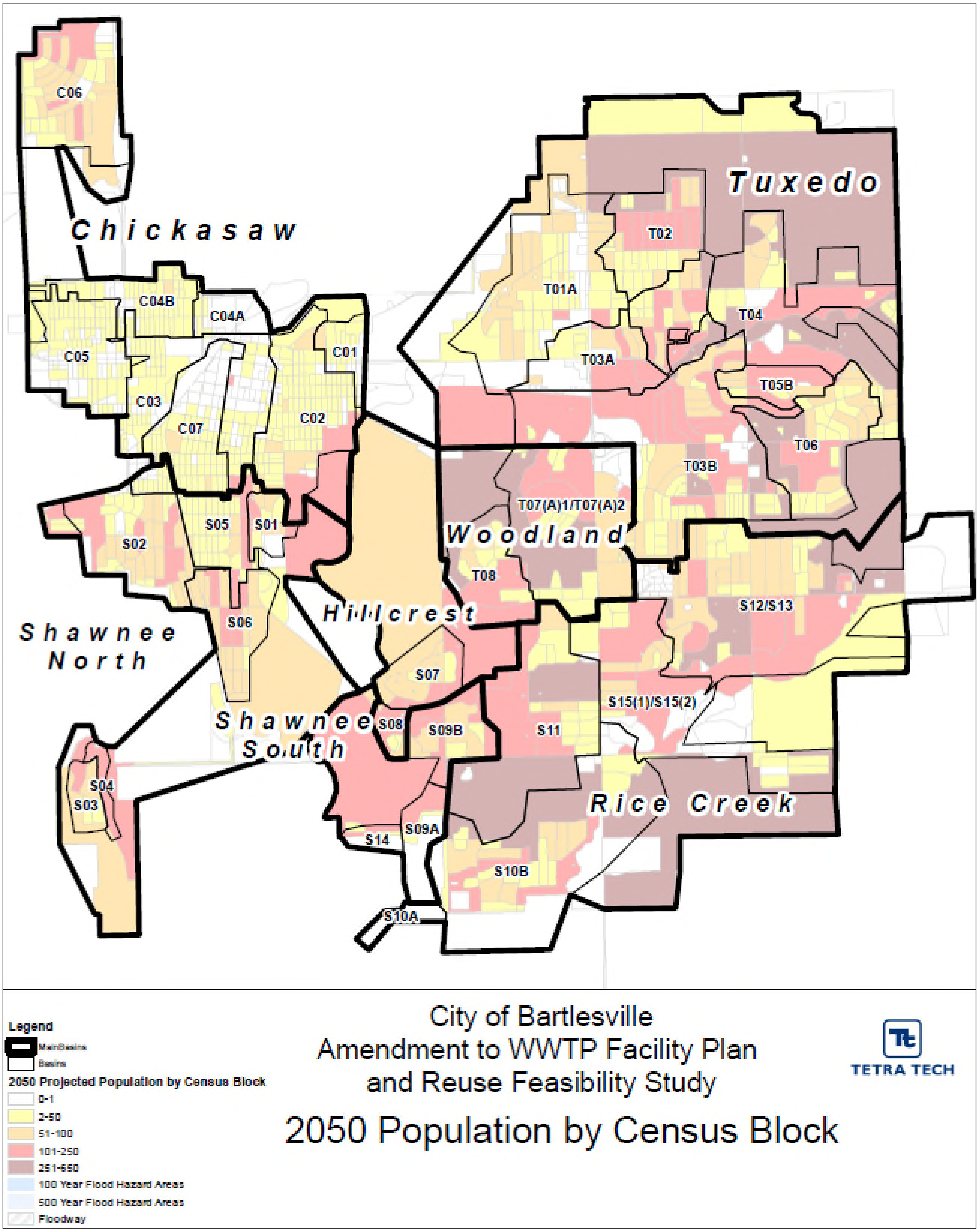

Figure TM1.1 - 2050 Basin Population Projections

3.0 FLOW AND WASTELOAD UPDATE

3.1 HISTORICAL, CURRENT AND PROJECTED FLOWS

Table TM1.2 shows the average annual, average day of the maximum month, and peak flows recorded at the Chickasaw Wastewater Treatment Plant (WWTP) for the 15-year period spanning 2001-2015. It includes the estimated service population for each year as well as the recorded total annual precipitation.

Table TM1.2 - Historical Flows and Rainfall

Year	Annual Average Daily Flow (MGD)	Average Daily flow of Max. Month (MGD)	Ratio Max. Month to Annual Average Daily Flow	Peak Daily Flow (MGD)	Estimated Service Population[1]	Average Per Capita Flow (gpcd)	Annual Rainfall[2]
2001	6.300	9.890	1.570	16.360	34,790	181	29.0
2002	5.492	7.460	1.358	13.943	34,833	158	32.9
2003	6.443	8.498	1.319	17.000	34,875	185	40.4
2004	8.139	10.742	1.320	23.600	34,918	233	46.4
2005	7.033	9.274	1.319	17.074	34,960	201	33.0
2006	6.180	10.190	1.649	20.824	35,137	176	27.4
2007	7.542	11.505	1.525	27.168	35,314	214	45.1
2008	7.840	12.065	1.539	19.832	35,491	221	53.0
2009	7.157	11.342	1.585	26.064	35,668	201	42.4
2010	7.269	8.475	1.166	16.285	35,750	203	40.3
2011	6.924	9.100	1.314	19.983	35,882	192	31.6
2012	6.337	9.690	1.529	22.885	36,015	174	30.5
2013	6.884	9.630	1.399	27.475	36,148	190	44.2
2014	6.517	8.160	1.252	12.585	36,282	178	33.8
2015	7.668	11.220	1.463	23.967	36,416	210	44.4
2016							
		Average:	1.420			194	38.3
		Normal Annual Rainfall[3]:					40.42

[1]2010 to 2015 population data provided by City of Bartlesville. Remaining data developed from 2010 census tract data with an annual growth rate of 0.37% applied.

[2]Rainfall for 2001-2009 obtained from Oklahoma Climatological Survey. Rainfall data for 2010 – 2015 is from the National Oceanic and Atmospheric Administration (NOAA) Online Weather Data for Bartlesville Municipal Airport. http://w2.weather.gov/climate/xmacis.php?wfo=tsa

[3]Obtained from NOAA Averaged for 1981-2010

Using the estimated populations, the average annual per capita flow was calculated. As shown, the average per capita flow (including wet weather effects) ranged from a low of 158 gallons per capita per day (gpcd) to a high of 233 gpcd. The 158 gpcd value occurred in 2002 which was the second year of a very dry two-year period. The 233 gpcd value occurred in 2004, which was a wetter than normal year but not the wettest year of the period. A value of 221 gpcd was estimated for the wettest year which was 2008. Annual rainfall for the 15-year period varied from as low as 27.4 inches (32 percent below normal) to 53.0 inches (31 percent above normal). Overall rainfall for the complete period was slightly below normal. The data for the 15-year period reflect a good distribution of both wet and dry years.

3.1.1 Average Dry Weather Flow

Average dry weather flows are used to size the treatment facilities and perform the economic evaluations of alternatives. The average dry weather flows were based on 2012 flow monitoring and hydraulic modeling completed by Tetra Tech. This is the most recent information available for this study. Other components of the total flow from each basin such as dry weather infiltration, commercial flow and industrial flow are included in the per capita flow rates, thus the final average from each basin/sub-basin is not purely population based and varies from basin to basin. Table TM1.3 shows the resulting gallons per capita per day (gpcd) flow rates.

Table TM1.3 - 2012 Dry Weather Flow by Basin

Collection System Basins	Sub-Basin No.	2012[1] Average Dry Weather (gpcd)	2012 Population Distribution	2012 Average Dry Weather Flow (MGD)
Chickasaw	C01-C07	168	7,201	1.209
Tuxedo	T01-T06	117	13,793	1.611
Woodland	T07-T10	131	1,745	0.228
Shawnee				
North	S01-S06	121	4,979	0.6
Hillcrest	S07-S08	136	676.13	0.092
South	S09 & S14	117	565.93	0.066
Rice Creek	S10-S15	173	6,720	1.162
Totals:			**35,681**	**4.968**

[1]Refer to Final Report of the Update of the Collection System Analysis, January 2013, prepared by Tetra Tech.

The dry weather flow gpcd rates were applied to the current and future population totals for each major basin to determine the average dry weather flows in 2050. Table TM1.4 summarizes these results. The total Average Dry Weather Flow for 2050 is projected to be 5.778 MGD.

Table TM1.4 - Current and Projected Average Dry Weather Flows

			2015 Population		2050 Population			
Collection System Basins	**Sub-Basin No.**	**Average Dry Weather (gpcd)**	**Population Distribution**	**Average Dry Weather Flow (MGD)**	**Population Distribution**	**Additional Population**	**Percent of Total Growth**	**Average Dry Weather Flow (MGD)**
Chickasaw	C01-C07	168	7,294	1.225	8,299	1,005	20%	1.393
Tuxedo	T01-T06	117	13,974	1.632	15,632	1,658	33%	1.826
Woodland	T07-T10	131	1,930	0.252	2,181	251	5%	0.285
Shawnee								
North	S01-S06	121	5,119	0.617	5,823	704	14%	0.702
Hillcrest	S07-S08	136	720	0.098	821	101	2%	0.112
South	S09 & S14	117	631	0.074	732	101	2%	0.085
Rice Creek	S10-S15	173	6,748	1.167	7,954	1,206	24%	1.375
Totals:			**36,416**	**5.064**	**41,441**	**5,025**	**100%**	**5.778**

3.1.2 Annual Average Daily Flow

Annual average daily flows were calculated based on the monthly operating reports. The average annual daily flows include wet weather days in the calculation. From Table TM1.2, the annual average daily flow for 2015 was 7.668 MGD, and the average per capita flow rate in 2015 was 210 gpcd.

Over the entire 15-year period reflected in Table TM1.2, the average per capita flow was 194 gpcd, and the average rainfall during the 15-year period was 38.3 inches which is slightly below the normal annual rainfall total of 40.42 inches. To determine a true average per capita flow under average rainfall conditions, a line was fitted to the per capita flow and rainfall data. Figure TM1.2 shows a plot of the raw data and the fitted line.

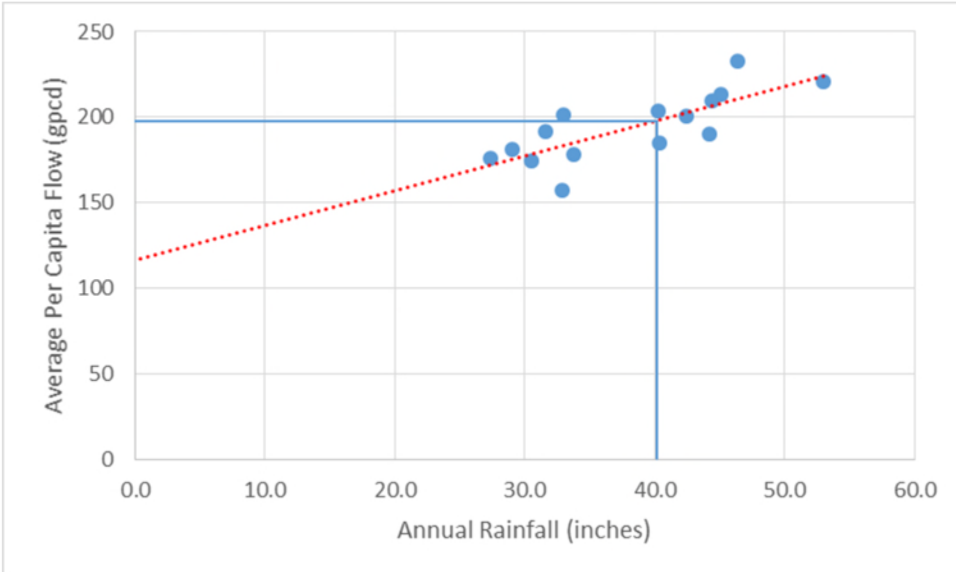

Figure TM1.2 - Average Annual Per Capita Flow vs. Annual Rainfall

The fitted line resulted in a per capita flow of 116 gpcd with no rainfall (the y-intercept). This is a reasonable value for a community with the demographics of Bartlesville and supports the quality of the data and fitted line. At the average rainfall of 40 inches per year, the average annual per capita flow from the fitted line is 198 gpcd. Using this per capita flow rate, the projected annual average daily flow for the design year of 2050 is 8.206 MGD.

3.1.3 Average Daily Flow – Maximum Month

Conditions experienced during the maximum month of the year are estimated for use in the design of particular unit processes in treatment plans (units with long retention times such as the biological treatment units). Such flows are estimated using the ratio of the average annual flow to the average day of the maximum month. During the 12-year period (see Table TM1.2), the ratio for flow varied from as low as 1.25 to as high as 1.65 with an average of 1.42. The 2015 average daily flow of the maximum month for Bartlesville was 11.22 MGD. The projected average daily flow for the maximum month in 2050 using the average ratio of 1.42 is 11.65 MGD.

3.1.4 Peak Daily Flow – Process

The peak daily flows summarized in Table TM1.2 represent the capacity of the existing Chickasaw Wastewater Treatment Plant (CWWTP) to process high volumes of wastewater while maintaining compliance. The peak flows shown in the table range from as low as 12.6 to as high as 27.5 MGD.

3.1.5 Peak Daily Flow – Influent

Peak influent flows are the highest flow rates expected to be discharged at the treatment plant before any excess flow is diverted to the flow equalization basins. Components of these flows include base flows (wastewater from contributors plus dry weather infiltration) and extraneous flows which enter the system when it rains. Under the design wet weather conditions, the extraneous flows are the dominant component of the peak influent rates.

To establish the peak influent flows in the system under the current and future average base flow conditions, the following steps were taken:

- The 2012 hydraulic model was run for a 5-year, 1-hour storm event to determine the peak inflow from each collection system basin.

- The inflow components (attenuated by the model) from the collection system were reduced by 20 percent to account for predicted peak inflow removal rates. The resulting rates were then increased by 5 percent to account for potential inflow from future new construction.
- Base flows were distributed using the population distribution shown in Table TM1.4 and the current and future projected populations.
- The adjusted inflow rates and the adjusted base flows were added together to establish a new peak design rate for each basin.

Table TM1.5 - Current and Projected Peak Flows

		Current Flows			2050 Projected Flows		
Collection System Basins	**Sub-Basin No.**	**Average Dry Weather Flow (MGD)**	**Peak Inflow (MGD, 2012 Model 5-year 1-hr Storm)**	**Total Peak Flow (MGD)**	**Average Dry Weather Flow (MGD)**	**Peak Inflow (MGD, 5-year 1-hr Storm)**	**Total Peak Flow (MGD)**
Chickasaw	C01-C07	1.209	23.514	24.723	1.393	21.015	22.408
Tuxedo	T01-T06	1.611	23.335	24.946	1.826	21.204	23.030
Woodland	T07-T10	0.228	7.167	7.395	0.285	6.286	6.571
Shawnee							
North	S01-S06	0.600	17.717	18.317	0.702	15.569	16.271
Hillcrest	S07-S08	0.092	3.460	3.552	0.112	3.019	3.131
South	S09 & S14	0.066	1.213	1.279	0.085	1.087	1.173
Rice Creek	S10-S15	1.162	15.130	16.292	1.375	13.848	15.224
Totals:		**4.968**	**--**	**--**	**5.778**	**--**	**--**

3.1.6 Flow Summary

Table TM1.6 provides a summary of the key flows which will be used to complete the alternative development and analysis.

Table TM1.6 - Flow Summary

	2015 Flows				**2050 Projected Flows**			
Collection System Basins	**Average Dry Weather Flow (MGD)**	**Average Annual Daily Flow (MGD)**	**Avg. Day Max. Month (MGD)**	**Total Peak Flow (MGD)**	**Average Dry Weather Flow (MGD)**	**Average Annual Daily Flow (MGD)**	***Avg. Day Max. Month (MGD)***	***Total Peak Flow (MGD)***
Chickasaw	1.225	1.532	2.241	24.723	1.393	1.643	2.333	22.408
Tuxedo	1.632	2.935	4.293	24.946	1.826	3.095	4.395	23.030
Woodland	0.252	0.405	0.593	7.395	0.285	0.432	0.613	6.571
Shawnee								
North	0.617	1.075	1.573	18.317	0.702	1.153	1.637	16.271
Hillcrest	0.098	0.151	0.221	3.552	0.112	0.163	0.231	3.131
South	0.074	0.133	0.194	1.279	0.085	0.145	0.206	1.173
Rice Creek	1.167	1.417	2.073	16.292	1.375	1.575	2.236	15.224
Totals:	**5.064**	**7.647**	**11.188**	**--**	**5.778**	**8.206**	**11.652**	**--**

3.1.7 Comparison to 2010 Wastewater Facility Study Projections

3.1.7.1 Population Projections. The 2010 Wastewater Facility Plan utilized 2040 as the projected planning year. The projected population for 2050 is 41,441 which is lower than the projected 2040 population from the 2010 study (42,739). This difference is due in part to the inclusion of the town of Ramona in the previous population projections. The previous study also included flows from the town of Ramona in the determination of average and peak daily flows. Based on subsequent discussions between Bartlesville and Ramona, flows from Ramona will not be incorporated into the future planning scenarios.

3.1.7.2 Flow Projections. Table TM1.7 compares the current 2050 projected flows versus the flows previously projected for 2040. The flow projections did not change appreciably from the 2010 WW Facility Plan. The projected annual average daily flow for 2050 is 8.206 which is within 1% of the 2040 annual average flow projected in the 2010 WW Facility Plan.

Table TM1.7 - Projected Flows Comparison to 2010 Study

		2050 Projected Flows (Current Facility Plan)				Previous Facility Plan 2040 Projected Flows		
Collection System Basins	**Sub-Basin No.**	**Average Dry Weather Flow (MGD)**	**Average Annual Daily Flow (MGD)**	**Avg. Day Max. Month (MGD)**	**Total Peak Flow (MGD)**	**Average Annual Daily Flow (MGD)**	**Avg. Day Max. Month (MGD)**	**Total Peak Flow (MGD)**
Chickasaw	C01-C07	1.393	1.643	2.333	22.408	1.781	2.633	15.000
Tuxedo	T01-T06	1.826	3.095	4.395	23.030	1.908	2.819	21.000
Woodland	T07-T10	0.285	0.432	0.613	6.571	0.281	0.416	5.000
Shawnee								
North	S01-S06	0.702	1.153	1.637	16.271	0.826	1.220	5.800
Hillcrest	S07-S08	0.112	0.163	0.231	3.131	0.129	0.190	1.200
South	S09 & S14	0.085	0.145	0.206	1.173	0.064	0.095	1.400
Rice Creek	S10-S15	1.375	1.575	2.236	15.224	3.187	4.710	12.500
Totals:		**5.778**	**8.206**	**11.652**	**--**	**8.176**	**12.083**	**--**

3.2 HISTORICAL, CURRENT AND PROJECTED WASTELOAD

Historical wasteload and process data were obtained for calendar years 2013, 2014, and 2015. The data was taken from the Discharge Monitoring Reports and the Monthly Operating Reports prepared by the contract operator at the CWWTP, Veolia Water North America. A spreadsheet included in TM1-Appendix A is a complete summary of the data. Influent wasteload data for biochemical oxygen demand (BOD), total suspended solids (TSS) and ammonia nitrogen (NH3-N) were developed along with alkalinity, pH, and sludge production rates. Current and projected values are discussed individually below.

3.2.1 BOD

Influent BOD concentrations averaged 195, 201 and 261 mg/L for 2013, 2014, and 2015, respectively. The maximum BOD concentrations ranged from 240 to 481 mg/L. Average influent mass loadings of BOD were approximately 10,400, 10,700 and 14,500 lbs/day for the three years, yielding per capita average production rates of 0.288, 0.295, and 0.398 lbs/capita/day. Peaking factors for the average day of the maximum month ranged from 1.25 to 1.47. Maximum day peaking factors ranged from 2.19 to 4.0. A BOD production of 0.327 lbs/capita/day, a maximum month peaking factor of 1.30, and a maximum day peaking factor (the rounded average from each year) of 3.03 were selected for use for planning. These values were determined by averaging data from 2013, 2014, and 2015. Table TM1.8 shows the values for each parameter.

3.2.2 TSS

Influent TSS concentrations averaged 308, 281, and 479 mg/L for 2013, 2014, and 2015, respectively. The maximum values ranged from 365 to 952 mg/L. Average influent mass loadings of TSS were approximately 16,600, 15,000, and 26,400 for the three years yielding per capita production rates of 0.461, 0.413, and 0.727 lbs/capita/day. Peaking factors for the average day of the maximum month ranged from 1.33 to 1.45. Maximum day peaking factors ranged from 2.86 to 3.58. A TSS production of 0.534 lbs/capita/day, a maximum month peaking factor of 1.41, and a maximum day peaking factor (the rounded average from each year) of 3.10 were selected for use for planning. These values were determined by averaging data from 2013, 2014, and 2015. Table TM1.8 shows the values for each parameter.

3.2.3 Ammonia

Influent NH3-N averaged 19.2, 22.9, and 16.8 mg/L for 2013, 2014, and 2015, respectively. The maximum values ranged from 21.7 to 47.4 mg/L. Average influent mass loadings of NH3-N were approximately 1,000, 1,200 and 950 lbs/day for the three years yielding per capita production rates of 0.029, 0.033, and 0.026 lbs/capita/day. Peaking factors for the average day of the maximum month ranged from 1.15 to 1.89. Maximum day peaking factors ranged from 2.73 to 10.58. A NH3-N production rate of 0.029 lbs/capita/day, a maximum month peaking factor of 1.46, and a maximum day peaking factor (the rounded average from each year) of 6.14 were selected for use for planning. These values were determined by averaging data from 2013, 2014, and 2015.Table TM1.8 shows the values for each parameter.

3.2.4 Alkalinity

Alkalinity is a measure of the capacity of the wastewater to resist changes in pH. It is an important parameter in wastewater treatment particularly at plants that nitrify (convert ammonia to nitrate) like the CWWTP. Nitrification consumes alkalinity, and insufficient alkalinity will inhibit nitrification leading to violations of the effluent ammonia discharge limits. Low alkalinity can also lead to low pH which is also regulated in the discharge. As expected (operators have never had trouble with low alkalinity at the CWWTP), the wastewater coming to the CWWTP has sufficient alkalinity to support nitrification and other processes. Each part of NH3-N converted to nitrate consumes one part of alkalinity. Influent total alkalinity values ranged from 140 to 398 mg/L with a 50th Percentile of 124 mg/L.

Table TM1.8 summarizes the influent wasteload parameters and values which will be used in the planning and design of the treatment facilities.

Table TM1.8 - Influent Wasteload Characteristics

Parameter	Peak Factor	Per Capita Production	2013, 2014, 2015	Planning Year 2050
		(lbs/cap/day)	(lbs/day)	(lbs/day)
BOD				
Average	--	0.327	11,800	13,600
Avg Day of Max. Mo.	1.30	--	15,300	17,600
Peak Day	3.03	--	35,700	41,100
TSS				
Average	--	0.534	19,300	22,100
Avg Day of Max. Mo.	1.41	--	27,100	31,100
Peak Day	3.10	--	59,900	68,500
NH3-N				
Average	--	0.029	1,070	1,220
Avg Day of Max. Mo.	1.46	--	1,560	1,780
Peak Day	6.14	--	6,570	7,490
Alkalinity, Minimum[1]	--	--	--	164

[1]No alkalinity values were provided in the 2013, 2014, and 2015 Monthly Operating Reports. The alkalinity value estimated for 2050 is based on the 2010 Facility Report.

3.2.5 Sludge Production Rates

While fairly predictable, the rates at which waste solids (sludge) are produced at biological treatment plants do vary depending on the processes employed and the nature of the wastewater treated. Production rates are required to size solids treatment, processing, storage, and disposal systems. Yields of total waste solids before digestion (from both the primary and secondary treatment systems) were calculated to be 0.84 lbs dry solids/lbs BOD during 2013; 0.89 lbs dry solids/lbs BOD during 2014; and 0.74 lbs dry solids/lbs BOD during 2015. These values used actual waste sludge production data for this period and then added the primary biological solids capture at 35%. The average solids yield after primary clarification and secondary treatment is estimated to be approximately 0.83 lbs dry solids/lb BOD. In terms of average influent flow, the total solids (includes primary sludge, secondary sludge, and non-bio gradable solids) is estimated to be 1,975 LB/MGD.

3.2.6 Summary of Influent Design Criteria

The design criteria to be used in the development and analysis of alternatives are summarized in Table TM1.9 below.

Table TM1.9 - Summary of Design Criteria

Parameter	Peaking Factor	2013, 2014, 2015			Planning Year 2050		
		Flow (MGD)	Conc. (mg/L)	Mass (lbs/day)	Flow (MGD)	Conc. (mg/L)	Mass (lbs/day)
Process Flow							
Average Annual Daily	--	7.021	--	--	8.205	--	--
Max Mo. Average Daily	1.37	9.632	--	--	11.652	--	--
Max. Day	3.99	28.000	--	--	28.000	--	--
Influent BOD							
Average	--	--	202	11,800	--	199	13,500
Avg. Day of Max. Mo.	1.30	--	190	15,300	--	181	17,500
Max. Day.	3.03	--	641	35,700	--	176	40,800
Influent TSS							
Average	--	--	330	19,300	--	323	22,100
Avg. Day of Max. Mo.	1.41	--	337	27,100	--	320	31,100
Max. Day.	3.10	--	257	59,900	--	293	68,500
Influent NH_3-N							
Average	--	--	18	1,070	--	18	1,220
Avg. Day of Max. Mo.	1.46	--	19	1,560	--	18	1,780
Max. Day.	6.14	--	28	6,570	--	32	7,490
Influent Alkalinity, Min.[1]	--	--	--	--	--	124	--
Observed Yield – (Primary & Secondary Sludge):		0.83	lbs DS/lbs BOD				
Total Solids Loading to Digester		1,975	Lbs DS / MGD				

[1] 50th Percentile Effluent Alkalinity, 2012-2017 data.

TM1- APPENDIX A

PLANT PERFORMANCE DATA - 2013, 2014 & 2015

CHICKASAW WASTEWATER TREATMENT PLANT
CITY OF BARTLESVILLE, OKLAHOMA

Month	Total Rainfall	Process Flow			Influent Characteristics																Sludge Production	
		Average Day	Minimum Day	Maximum Day	BOD				TSS				Ammonia				pH		Alkalinity Average		Mass of Dry Solids	Observed Yield
					Average Day		Maximum Day		Average Day		Maximum Day		Average Day		Maximum Day		Minimum	Maximum				
	(inches)	(MGD)	(MGD)	(MGD)	(mg/L)	(lbs/day)	(mg/L)	(lbs/day)	(mg/L)	(lbs/day)	(mg/L)	(lbs/day)	(mg/L)	(lbs/day)	(mg/L)	(lbs/day)	(S.U.)	(S.U.)	(mg/L)		(lbs DS/mo)	(lb DS/lb BOD)
YEAR 2013																						
JAN	2.10	5.173	4.781	8.329	251.6	10,826	378.8	15,287	360.3	15,577	518.0	21,462	23.1	1,031	33.8	1,681	6.63	7.22			106,600	0.33
FEB	2.20	6.525	4.980	11.007	193.3	10,693	294.6	19,939	288.2	16,305	458.0	35,434	19.0	1,064	29.1	2,451	6.67	7.65			168,600	0.53
MAR	1.30	6.698	5.711	8.829	270.7	14,186	420.8	21,555	388.3	20,275	576.0	27,821	18.2	956	24.2	1,208	7.18	7.78			144,700	0.33
APR	3.95	8.291	5.531	11.866	192.7	13,159	311.4	24,605	342.5	24,019	602.0	47,566	16.7	1,193	31.4	2,838	6.99	7.72			175,200	0.44
MAY	8.59	7.138	5.090	15.108	222.8	13,510	328.5	29,898	314.8	18,953	584.0	32,358	17.4	1,020	31.4	1,564	6.79	7.60			142,200	0.35
JUN	3.60	9.628	4.783	27.475	87.2	5,130	137.9	8,852	206.7	11,965	440.0	17,552	15.0	935	21.2	1,770	6.72	7.23			145,700	1.03
JUL	5.70	6.284	4.552	9.645	208.1	10,199	328.1	16,700	397.5	19,508	636.0	37,178	21.5	1,112	42.1	2,500	6.52	7.19			188,900	0.61
AUG	6.90	8.107	5.046	12.842	155.5	8,903	338.8	15,865	275.1	16,416	590.0	39,512	14.7	898	22.3	1,430	6.58	7.11			136,200	0.51
SEP	4.65	6.134	4.349	8.884	200.5	9,756	329.2	17,687	312.2	15,239	624.0	31,069	21.8	1,072	38.4	1,890	6.58	7.10			113,800	0.38
OCT	3.40	6.145	4.532	8.867	181.1	8,878	265.3	15,405	291.3	14,651	450.0	25,143	20.4	1,027	31.1	2,300	6.73	7.10			123,300	0.46
NOV	1.75	6.357	4.713	9.651	192.7	9,783	307.0	15,913	279.4	14,294	626.0	28,307	21.1	1,069	34.2	1,735	6.59	7.19			134,100	0.46
DEC	0.20	6.186	4.751	8.595	186.5	9,544	399.1	20,873	244.3	12,495	446.0	23,326	21.4	1,106	30.3	1,816	6.69	7.17			174,500	0.61
Total:	44.34																				1,753,800	
Average:	38.99	6.889			195.2	10,381			308.4	16,641			19.2	1,040								0.48
Minimum:			4.349														6.52					
Maximum:		9.628		27.475	270.7	14,186	420.8	29,898	397.5	24,019	636.0	47,566	23.1	1,193	42.1	2,838		7.78				
Pkg Factor:		1.40	0.63	3.99		1.37		2.88		1.44		2.86		1.15		2.73						
Avg. Pop.:	36,082																					
Per Cap. Avg. Daily Flow:		191 gpcd																				
Per Cap. Min. Daily Flow:			121 gpcd																			
Per Cap. Avg. BOD:							0.288 lb/cap/day															
Per Cap. Avg. TSS:											0.461 lb/cap/day											
Per Cap. Avg. NH3-N:															0.029 lb/cap/day							
YEAR 2014																						
JAN	0.10	6.285	5.556	7.416	159.4	8,391	235.7	12,644	199.1	10,469	288.0	16,083	20.7	1,088	31.4	1,700	6.86	7.42			141,100	0.57
FEB	0.10	5.900	5.522	6.359	186.0	9,247	280.5	14,065	235.5	11,697	348.0	18,369	23.2	1,129	32.2	1,483	6.78	7.60			106,500	0.39
MAR	1.35	6.303	5.142	9.264	198.6	10,645	276.3	13,121	257.9	13,844	376.0	20,621	20.9	1,096	45.3	2,151	6.92	7.44			201,500	0.64
APR	1.34	5.842	5.095	6.802	233.5	11,747	396.4	22,487	338.6	16,975	482.0	26,776	20.7	1,035	30.3	1,594	6.81	7.32			104,600	0.30
MAY	2.79	5.811	5.123	7.380	240.3	11,720	326.6	16,254	327.1	15,998	598.0	33,345	26.3	1,261	138.0	6,406	6.82	7.34			149,800	0.42
JUN	9.10	8.160	6.203	11.029	176.6	11,530	283.4	17,750	248.5	16,378	408.0	28,049	14.5	972	25.2	1,787	6.69	7.38			170,000	0.49
JUL	3.10	6.508	5.523	9.679	234.5	12,712	465.8	23,414	365.0	19,910	1066.0	53,583	16.9	916	21.8	1,542	6.70	7.92			151,200	0.39
AUG	1.20	6.045	4.504	8.823	226.0	10,950	525.0	20,399	318.1	15,408	708.0	27,510	47.4	2,277	283.0	12,715	6.68	7.21			157,200	0.47
SEP	6.65	6.456	5.192	9.869	191.5	10,107	281.3	17,403	273.3	14,517	462.0	28,582	19.4	1,037	33.9	1,851	6.58	7.06			285,900	0.93
OCT	6.70	7.645	5.688	12.585	166.1	9,580	271.7	14,373	222.8	12,873	404.0	22,949	20.6	1,269	31.3	2,861	6.77	7.53			238,300	0.83
NOV	1.83	6.536	5.523	10.615	195.8	10,665	369.7	19,148	304.9	16,519	516.0	34,577	25.4	1,344	64.3	3,092	6.89	7.49			137,600	0.43
DEC	1.25	6.679	5.345	8.640	198.5	10,896	287.0	16,396	275.7	15,070	474.0	23,904	18.4	1,008	25.3	1,369	6.83	7.40			225,900	0.68
Total:	35.51																				2,069,600	
Average:	38.99	6.514			200.6	10,682			280.5	14,972			22.9	1,203								0.55
Minimum:			4.504														6.58					
Maximum:		8.160		12.585	240.3	12,712	525.0	23,414	365.0	19,910	1,066.0	53,583	47.4	2,277	283.0	12,715		7.92	0			
Pkg Factor:		1.25	0.69	1.93		1.19		2.19		1.33		3.58		1.89		10.57						
Avg. Pop.:	36,195																					
Per Cap. Avg. Daily Flow:		180 gpcd																				
Per Cap. Min. Daily Flow:			124 gpcd																			
Per Cap. Avg. BOD:							0.295 lb/cap/day															
Per Cap. Avg. TSS:											0.414 lb/cap/day											
Per Cap. Avg. NH3-N:															0.033 lb/cap/day							

TM1- APPENDIX A

PLANT PERFORMANCE DATA - 2013, 2014 & 2015

CHICKASAW WASTEWATER TREATMENT PLANT

CITY OF BARTLESVILLE, OKLAHOMA

YEAR 2015																					
JAN	0.20	6.116	4.943	6.928	283.1	14,150	553.5	25,588	502.0	24,975	992.0	46,808	21.7	1,087	28.1	1,440	6.84	7.47		483,600	1.12
FEB	0.50	6.425	5.758	7.255	310.3	16,532	486.8	25,789	579.4	30,805	1,412.0	75,791	19.4	1,039	33.5	1,757	6.87	7.50		204,600	0.41
MAR	1.05	7.343	6.089	10.304	251.5	15,222	404.2	23,004	508.8	30,812	954.0	55,925	15.7	958	27.9	1,768	6.80	7.67		178,400	0.38
APR	2.50	7.924	6.506	17.532	229.3	14,862	470.2	32,079	417.0	26,728	1,160.0	71,271	18.7	1,214	79.1	4,905	6.85	7.55		134,600	0.30
MAY	6.90	11.228	6.452	18.916	123.7	9,969	242.0	16,840	212.9	18,683	364.0	45,793	15.9	1,281	53.6	3,431	6.58	7.34		147,200	0.52
JUN	5.10	11.216	6.201	18.320	132.0	10,937	262.8	19,825	256.9	21,623	532.0	39,996	8.8	735	19.6	1,161	6.86	7.53		188,800	0.58
JUL	7.10	7.145	5.401	14.438	274.3	15,398	498.0	25,074	367.3	21,390	488.0	38,291	15.6	876	33.9	1,714	6.78	7.39		131,100	0.28
AUG	5.23	6.310	5.237	8.485	243.9	12,825	342.0	23,233	386.6	20,612	640.0	44,665	18.5	980	26.6	1,493	6.48	7.23		126,400	0.32
SEP	2.60	5.865	4.469	8.019	286.7	13,692	505.5	20,726	485.5	23,602	762.0	34,197	17.8	864	29.6	1,343	6.67	7.20		72,900	0.17
OCT	1.41	5.003	4.024	8.261	480.6	19,286	640.5	26,655	952.4	38,212	1,814.0	67,020	21.1	851	33.2	1,418	6.82	7.49		94,600	0.16
NOV	7.60	6.997	4.316	15.808	334.7	16,573	640.5	33,921	766.7	37,536	1,512.0	73,367	20.1	974	42.6	2,038	6.69	7.66		108,600	0.22
DEC	6.10	10.365	5.470	23.967	181.5	14,200	487.5	57,953	306.9	22,237	932.0	58,294	7.9	621	22.7	1,614	6.56	7.42		139,600	0.34
Total:	46.29																			2,010,400	
Average:	38.99	7.661			261.0	14,471			478.5	26,435			16.8	957							0.39
Minimum:			4.024														6.48				
Maximum:		11.228		23.967	480.6	19,286	640.5	57,953	952.4	38,212	1,814.0	75,791	21.7	1,281	79.1	4,905		7.67			
Pkg Factor:		1.47	0.53	3.13		1.33		4.00		1.45		2.87		1.34		5.13					
Avg. Pop.:	36,329																				
Per Cap. Avg. Daily Flow:		211 gpcd																			
Per Cap. Min. Daily Flow:			111 gpcd																		
Per Cap. Avg. BOD:						0.398 lb/cap/day															
Per Cap. Avg. TSS:										0.728 lb/cap/day											
Per Cap. Avg. NH3-N:														0.026 lb/cap/day							

Average Conditions 2013, 2014, 2015			
Flow:			
Per Cap. Avg. Flow:	194 gpcd	Average of 2013, 2014, 2015 Per Cap. Avg. Daily Flow	3.99 Max Day Peaking Factor 2013, 2014, 2015
Population:	36,202	Average of 2013, 2014, 2015 Population	
Existing Average Flow:	7.021		0.47 Average Observed Sludge Yield (lbs DS/ lbs BOD)
Min. Daily Flow:	4.293	119 gpcd Average of 2013, 2014, 2015 Per Cap. Min. Daily Flow	
Avg/Max Mo Daily Flow PF:	1.372	Average 2013, 2014, 2015 Peaking Factors	
Avg/Max Mo. Daily Flow:	9.632		
Influent BOD:			
Per Cap. Avg. BOD:		0.327 lb/cap/day	
Avg Daily BOD:	202 mg/L	11,800 lb/day	
Avg/Max Mo Daily BOD PF:		1.30	
Avg/Max Mo Daily BOD:	190 mg/L	15,300 lbs/day	
Avg/Max Day BOD PF:		3.03	
Max Day BOD:		35,700 lbs/day	
Influent TSS:			
Per Cap. Avg. TSS:		0.534 lb/cap/day	
Avg Daily TSS:	330 mg/L	19,300 lbs/day	
Avg/Max Mo Daily TSS PF:		1.41	
Avg/Max Mo Daily TSS:	337 mg/L	27,100 lbs/day	
Avg/Max Day TSS PF:		3.10	
Max Day TSS:		59,900 lbs/day	
Infl. NH3-N:			
Per Cap. Avg. NH3-N:		0.029 lb/cap/day	
Avg Daily NH3-N:	18.3 mg/L	1,070 lbs/day	
Avg/Max Mo Daily NH3-N PF:		1.46	
Avg/Max Mo Daily NH3-N:	19.4 mg/L	1,560 lbs/day	
Avg/Max Day NH3-N PF:		6.14	
Max Day NH3-N:		6,570 lbs/day	

TM1- APPENDIX A

PLANT PERFORMANCE DATA - 2013, 2014 & 2015

CHICKASAW WASTEWATER TREATMENT PLANT
CITY OF BARTLESVILLE, OKLAHOMA

2050 Projected Conditions												
Flow:												
Per Cap. Avg. Flow:	198	gpcd										
Population:	41,441	Value taken from TM1, page 9										
Average Flow:	8.205											
Min. Daily Flow:	4.914	118.578421 gpcd										
Avg/Max Mo Daily Flow PF:	1.420											
Avg/Max Mo. Daily Flow:	11.652	Value taken from TM1, page 7										
Influent BOD:												
Per Cap. Avg. BOD:				0.327	lb/cap/day							
Avg Daily BOD:			199 mg/L	13,600	lb/day							
Avg/Max Mo Daily BOD PF:				1.30								
Avg/Max Mo Daily BOD:			181 mg/L	17,600	lbs/day							
Avg/Max Day BOD PF:				3.03								
Max Day BOD:				41,100	lbs/day							
Influent TSS:												
Per Cap. Avg. TSS:							0.534	lb/cap/day				
Avg Daily TSS:						323 mg/L	22,100	lbs/day				
Avg/Max Mo Daily TSS PF:							1.41					
Avg/Max Mo Daily TSS:						320 mg/L	31,100	lbs/day				
Avg/Max Day TSS PF:							3.10					
Max Day TSS:							68,500	lbs/day				
Infl. NH3-N:												
Per Cap. Avg. NH3-N:											0.029	lb/cap/day
Avg Daily NH3-N:										17.8 mg/L	1,220	lbs/day
Avg/Max Mo Daily NH3-N PF:											1.46	
Avg/Max Mo Daily NH3-N:										18.3 mg/L	1,780	lbs/day
Avg/Max Day NH3-N PF:											6.14	
Max Day NH3-N:											7,490	lbs/day

PLANT PERFORMANCE DATA - 2013, 2014, 2015

CHICKASAW WASTEWATER TREATMENT PLANT
CITY OF BARTLESVILLE, OKLAHOMA

Month	Process Flow			Effluent Characteristics															
				BOD				TSS				Ammonia					Alkalinity	pH	
	Average Day	Minimum Day	Maximum Day	Average Day		Maximum Day		Average Day		Maximum Day		Average Day		Week Avg	Maximum Day		Average	Minimum	Maximum
	(MGD)	(MGD)	(MGD)	(mg/L)	(lbs/day)	(mg/L)	(lbs/day)	(mg/L)	(lbs/day)	(mg/L)	(lbs/day)	(mg/L)	(lbs/day)	(mg/L)	(mg/L)	(lbs/day)	(mg/L)	(S.U.)	(S.U.)
2013																			
JAN	5.173	4.781	8.329	7.2	323	14.6	677	7.9	347	15.8	677	0.2	10	0.2	1.6	69		6.6	7.0
FEB	6.525	4.980	11.007	5.6	344	11.0	1,010	4.5	275	7.9	725	0.2	11	0.2	0.8	51		6.6	7.3
MAR	6.698	5.711	8.829	4.4	231	5.8	312	3.9	206	5.6	301	0.1	4	0.1	0.3	21		6.9	7.3
APR	8.291	5.531	11.866	4.2	318	10.6	962	3.8	279	5.3	479	0.3	27	0.3	1.9	174		6.8	7.2
MAY	7.138	5.090	15.108	6.0	384	13.3	774	4.1	279	7.1	655	0.1	6	0.1	0.8	56		6.8	7.3
JUN	9.628	4.783	27.475	7.1	564	12.7	1,514	6.6	466	12.7	918	0.0	3	0.0	0.1	10		6.6	7.3
JUL	6.284	4.552	9.645	5.2	270	11.1	675	3.8	188	5.3	282	0.6	35	0.6	3.5	208		6.6	7.1
AUG	8.107	5.046	12.842	4.1	258	9.7	612	5.1	320	9.8	536	0.2	11	0.2	1.4	76		6.8	7.5
SEP	6.134	4.349	8.884	3.9	199	9.5	615	3.8	197	8.8	570	0.1	5	0.1	0.4	13		6.6	7.2
OCT	6.145	4.532	8.867	5.6	293	13.3	768	5.6	281	11.6	531	0.2	11	0.2	0.9	68		6.7	7.1
NOV	6.357	4.713	9.651	6.8	366	14.8	979	6.7	375	13.6	1,088	0.3	15	0.3	0.8	54		6.5	7.1
DEC	6.186	4.751	8.595	6.2	327	15.8	827	4.8	251	8.7	539	0.4	20	0.3	1.5	77		6.6	7.1
Total:																			
Average:	**6.889**			5.5	323			5.1	289			0.2	13						
Minimum:		4.349																6.5	
Maximum:	9.628		27.475		564	15.8	1,514		466	15.8	1,088	0.6	35	0.6	3.5	208			7.5
Pkg Factor:	**1.40**	0.63	3.99																
2014																			
JAN	6.285	5.556	7.416	8.9	465	17.6	1,022	6.0	312	9.4	509	0.3	15	0.3	1.9	106		6.7	7.1
FEB	5.900	5.522	6.359	9.1	448	14.9	738	6.4	317	12.2	607	0.4	17	0.4	1.6	81		6.6	7.0
MAR	6.303	5.142	9.264	7.8	418	16.2	889	5.3	286	11.8	616	0.3	18	0.4	1.6	76		6.6	7.2
APR	5.842	5.095	6.802	10.5	519	19.4	980	7.0	352	13.0	738	0.2	12	0.2	0.9	47		6.7	7.1
MAY	5.811	5.123	7.380	4.9	235	7.1	332	3.9	191	7.6	359	0.1	3	0.1	0.1	7		6.8	7.2
JUN	8.160	6.203	11.029	4.3	289	7.4	537	3.7	250	7.6	617	0.1	6	0.1	0.1	9		6.8	7.3
JUL	6.508	5.523	9.679	3.5	188	8.5	393	3.1	168	5.4	291	0.1	6	0.1	0.2	12		6.7	7.6
AUG	6.045	4.504	8.823	3.0	149	4.4	210	3.3	165	4.6	268	0.1	4	0.1	0.2	9		6.7	7.3
SEP	6.456	5.192	9.869	3.6	195	6.5	401	4.1	220	7.8	366	0.2	12	0.2	0.7	61		6.6	7.0
OCT	7.645	5.688	12.585	4.6	285	8.8	587	3.9	250	5.6	492	0.2	15	0.2	1.2	77		6.6	7.4
NOV	6.536	5.523	10.615	5.4	322	10.9	961	4.7	263	7.0	584	0.1	6	0.1	1.3	63		6.7	7.3
DEC	6.679	5.345	8.640	4.2	234	6.1	337	3.9	215	5.7	346	0.1	3	0.1	0.1	9		6.7	7.3
Total:																			
Average:	**6.514**			5.8	312			4.6	249			0.2	10						
Minimum:		4.504																6.6	
Maximum:	8.160		12.585		519	19.4	1,022		352	13.0	738	0.4	18	0.4	1.9	106			7.6
Pkg Factor:	**1.25**	0.69	1.93																
2015																			
JAN	6.116	4.943	6.928	4.9	248	8.1	390	3.3	166	8.8	422	0.1	5	0.1	0.4	22		6.7	7.1
FEB	6.425	5.758	7.255	4.7	251	6.3	342	3.1	164	3.8	230	0.1	3	0.1	0.2	11		6.7	7.2
MAR	7.343	6.089	10.304	4.2	257	7.4	471	3.4	208	8.6	545	0.2	10	0.2	0.6	39		6.7	7.1
APR	7.924	6.506	17.532	5.4	351	9.1	602	4.4	293	6.7	643	0.1	9	0.1	0.6	40		6.8	7.1
MAY	11.228	6.452	18.916	7.8	749	12.8	1,783	8.1	799	14.6	2,303	0.3	35	0.4	2.0	194		6.8	7.4
JUN	11.216	6.201	18.320	5.4	479	12.4	893	4.6	412	10.6	951	0.3	32	0.4	3.0	324		6.9	7.7
JUL	7.145	5.401	14.438	4.2	248	10.6	710	4.5	266	7.0	626	0.3	15	0.2	1.7	119		6.7	7.2
AUG	6.310	5.237	8.485	4.5	242	14.5	978	4.9	259	8.8	475	0.1	6	0.1	0.4	22		6.6	7.2
SEP	5.865	4.469	8.019	4.2	203	8.1	374	5.6	274	11.4	449	0.1	7	0.1	0.5	34		6.5	7.1
OCT	5.003	4.024	8.261	5.8	236	10.4	421	10.8	436	33.1	1,414	0.2	9	0.2	1.7	72		6.7	7.3
NOV	6.997	4.316	15.808	5.8	336	8.6	815	7.0	414	11.2	1,165	0.1	4	0.1	0.4	25		6.9	7.4
DEC	10.365	5.470	23.967	8.3	908	22.6	4,517	9.8	1,087	26.8	5,037	0.2	33	0.2	2.0	292		6.6	7.2
Total:																			
Average:	**7.661**			5.4	376			5.8	398			0.2	14						
Minimum:		4.024																6.5	
Maximum:	11.228		23.967		908	22.6	4,517		1,087	33.1	5,037	0.3	35	0.4	3.0	324			7.7
Pkg Factor:	**1.47**	0.53	3.13																

Amendment to

WWTP Facility Plan and Reuse Feasibility Study

Technical Memorandum No. 2

Existing Chickasaw Wastewater Treatment Plant Update

For:

CITY OF BARTLESVILLE

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ACRONYMS/ABBREVIATIONS

Acronyms/Abbreviations	Definition
AD	Anaerobic Digesters
AOR	Actual Oxygen Reduction
BOD	Biochemical Oxygen Demand
BOD	Biological Oxygen Demand
CWWTP	Chickasaw Wastewater Treatment Plant
DAF	Dissolved Air Flotation
DEQ	Department of Environmental Quality
DS	Dry Solids
FC	Final Clarifier
FEB	Flow Equalization Basin
GPCD	Gallons per Capita per Day
MGD	Million Gallons per Day
MLSS	Mixed Liquor Suspended Solids
NH3-N	Ammonia Nitrogen
RAS	Return Activated Sludge
SCADA	Supervisory Control and Data Acquisition
SOR	Surface Overflow Rate
TM	Technical Memorandum
TSS	Total Suspended Solids
VFD	Variable Frequency Drive
WAS	Waste Activated Sludge
WW	Wastewater

1.0 INTRODUCTION

Technical Memorandum No.2 (TM-2) covers the evaluation of the existing wastewater handling and treatment facilities at the Chickasaw wastewater treatment plant (CWWTP) including the Chickasaw lift station.

2.0 EXISTING TREATMENT FACILITIES

2.1 CHICKASAW LIFT STATION

The Chickasaw lift station (Figure TM2.2-1) is located at the CWWTP site directly south of the existing headworks structure and adjacent to the flow equalization basin. The lift station receives flow from gravity interceptors serving Chickasaw basins C01 through C07, as well as internal recycle flows from the treatment process and return flow from the 20 MG flow equalization basin (FEB) which is located at the CWWTP site. Flow from the station is discharged to the screening and degritter structure at the plant headworks.

The lift station is equipped with three vertical, dry-pit centrifugal pumps which are controlled by variable frequency drives (VFD). The station is served by a standby power generator which also serves the treatment plant and automatically activates upon loss of line power.

Figure TM2.2-1 Chickasaw Lift Station

The station structure was constructed in 1983. The current pumps and VFDs were installed in 2003. The standby power generator was installed in 1983.

The wet well influent channel floods the lower level during high flows when one or more pumps are out of service (see Figure TM2.2-2). The air circulation inlet at the basement is also prone to flooding. There is a single grinder unit but no redundancy. Groundwater is seeping through the southeast wall of the structure (see Figure TM2.2-3).

As noted in TM1, the projected peak wet weather flow for the Chickasaw basins is 22.41 MGD. As noted in Table TM2.2-1, the firm capacity of the Chickasaw lift station is approximately 16.1 MGD and total capacity with all three pumps is approximately 18.4 MGD. Therefore, the existing Chickasaw FEB in conjunction with the Chickasaw lift station must be used to handle the projected peak wet weather flow.

The lift station meets the key ODEQ standards as noted in Table TM2.2-2.

Figure TM2.2-2 Influent Channel Flooding

Figure TM2.2-3 Groundwater Seepage through Walls

The key characteristics of the lift station are summarized in the following table.

Table TM2.2-1 Chickasaw Lift Station Summary

Process Pumps	
Pump Type	Centrifugal, Dry-Pit (2003)
Number	3
Horsepower, Each	150
Capacity, Each (gpm)	7,000
Capacity, Firm (MGD)	16.1
Capacity, Maximum (MGD)	18.4
Force Main Size (inches in diameter)	18
Control	VFDs (2003)
Screening	None
Standby Power	
Type	Diesel
Number	1
Generator	750 kW (1983)

Table TM2.2-2 Chickasaw Lift Station Conformance with DEQ Standards			
DEQ Standard 252:656		**Regulatory Compliance**	
Criteria	**Description**	**Existing 7.0 MGD**	**Proposed 8.2 MGD**
7-1(b)	Multiple pumps	yes	yes
7-1(c)(1)	Force main 4"min; velocity 2 fps min	yes	yes

2.2 LIQUID TREATMENT PROCESSES

The following summarizes the existing liquid treatment unit processes at the CWWTP. Each unit process is also assessed for compliance with the current treatment plant design standards of the Oklahoma Department of Environmental Quality (DEQ), OAC 252:656. A process flow schematic and a site plan of the CWWTP are shown on Figure TM2.2-4 and Figure TM2.2-5, respectively.

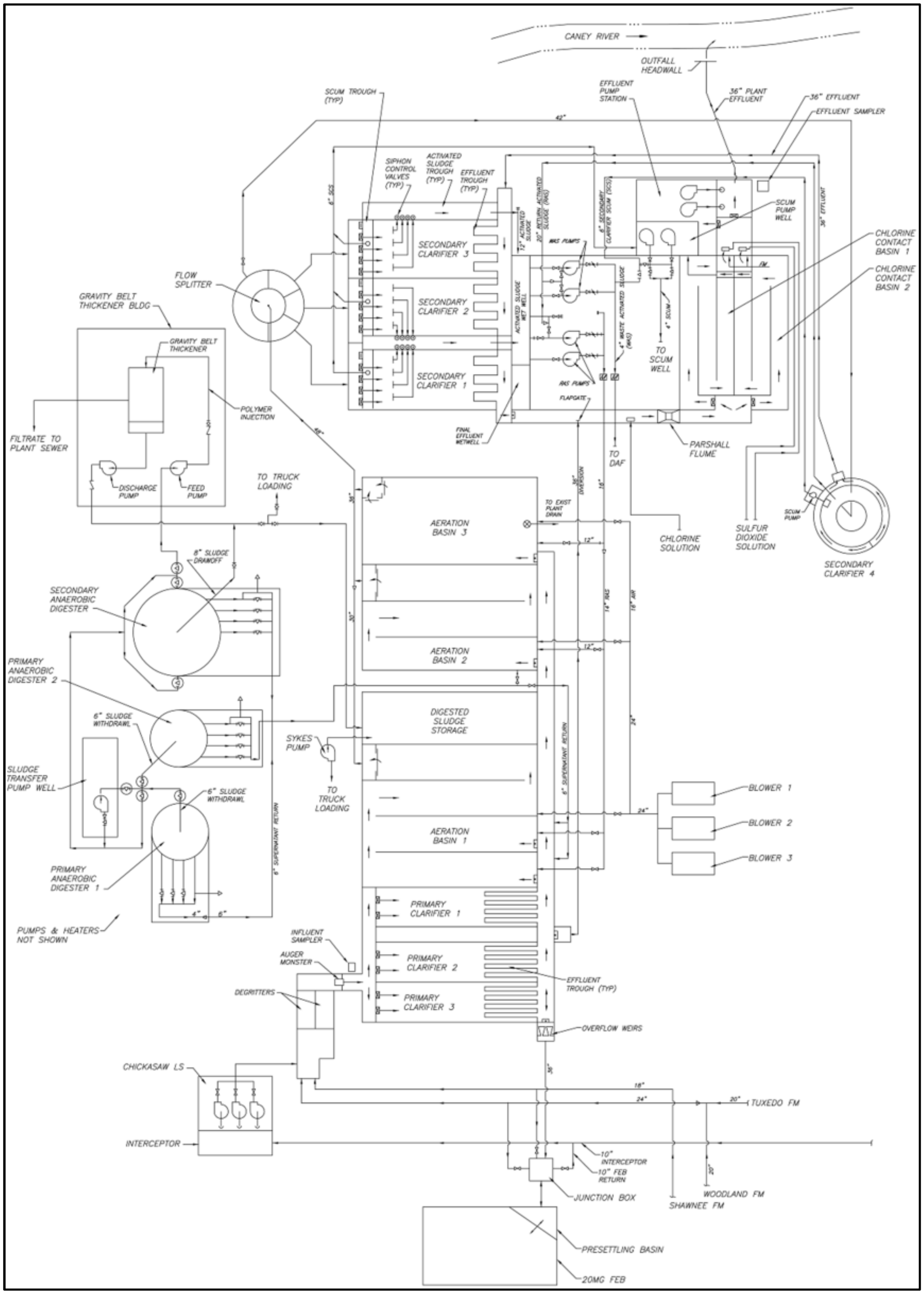

Figure TM2.2-4 Chickasaw Wastewater Treatment Plant Process Schematic –Existing

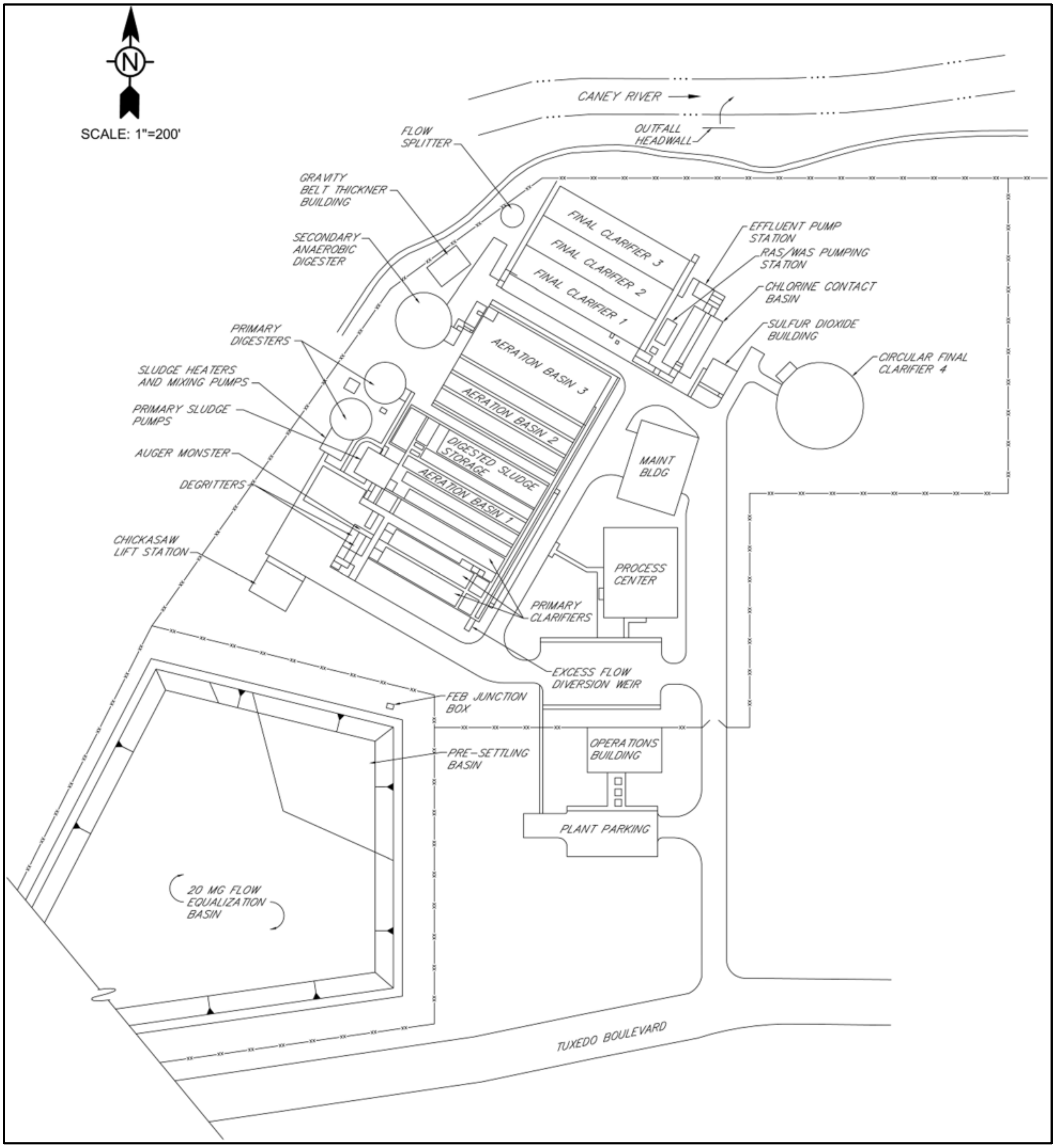

Figure TM2.2-5 Chickasaw Wastewater Treatment Plant Site Plan- Existing

2.2.1 Flow Equalization Basin

A single, 20 MG FEB is located at the CWWTP site. The FEB is divided into two cells - a concrete lined pre-sedimentation cell and an earthen basin lined with a synthetic liner for storage. Primary clarifier effluent (also pre-settled) flow is diverted to the FEB via an overflow weir immediately downstream of the primary clarifiers.

Raw wastewater can be manually diverted to the FEB via valved connections at a junction box to the Tuxedo, Shawnee, and Woodland force mains. The FEB, including the synthetic liner, was constructed in 1986. The basin is in good condition; however, the synthetic liner is in poor condition. The City awarded a contract to replace the liner in 2016, and the project is expected to be completed by early 2017.

Figure TM2.2-6 Flow Equalization Basin

The key characteristics of the FEB are summarized in the following table.

Table TM2.2-3 Flow Equalization Basin Summary	
Type	Earthen, Synthetic Liner (1986)
Pre-sedimentation	Concrete Lined Basin
Aeration	None
Capacity (MG)	20.0

The following table reviews conformance with DEQ regulations.

Table TM2.2-4 FEB Conformance with DEQ Standards			
DEQ Standard 252:656		**Regulatory Compliance**	
Criteria	Description	Existing 7.0 MGD	Proposed 8.2 MGD
13-4			
13-4(c)	>5 mg requires 2 basins	yes	yes
3-5(e)	Size for 7-day 10 year event	Yes (18.9 MGD)	Yes (18.9 MGD)
13-4(g)	Manual or automatic flow diversion	yes	yes
13-4(g)	Flow measurement required	no	no

The FEB volume outside of the concrete lined basin is provided with synthetic liner. DEQ regulation requires the basin volume to be adequate to handle the 7-day, 10-year storm event. The FEB volume size needed is discussed in Technical Memorandum 3.

2.2.2 Headworks

The headworks structure is located adjacent to and upstream of the primary clarifiers. The structure receives flows from the Chickasaw, Shawnee, Tuxedo, and Woodland lift stations (Woodland force main discharges into the Tuxedo lift station force main near the plant).

The headworks structure is equipped with two aerated, rectangular grit chambers with chain and bucket removal systems (Figure TM2.2-8). A single Auger Monster screen is located in the effluent channel between the degritters and the primary clarifiers (Figure TM2.2-9). The Auger Monster provides grinding of influent solids and screening, removing solids greater than ¼-inch in diameter. Screenings are collected by an auger that washes and transports the screenings out of the channel for collection and storage in a refuse container. Grit and screenings from the Auger Monster are discharged to a roll-off which is located under a covered structure. After screening, flow is discharged to the primary clarifier influent channel.

Figure TM2.2-7 Headworks Structure

An influent composite sampler is located just downstream of the Auger Monster and ahead of the primary clarifiers. Currently there is no influent flow measurement device at the plant.

DEQ standards require redundant screens which the plant does not currently have. The 2010 facility plan included a new structure for excess flow diversion and screening. This should be retained in the updated plan.

The headworks structure was constructed in 1983. The degritters were rehabilitated in 1993 and are in need of rehabilitation. The Auger Monster was added in 2001.

The plant accepts approximately 100,000 gallons of septage each month. It is currently discharged directly into a manhole on site which does not allow the contents to be sampled and tested before it is introduced into the treatment process. To better monitor and manage, a septage receiving station is recommended.

The plant operator, Veolia Water, indicates that the grit equipment is in need of comprehensive rehabilitation. The degritters meet the DEQ requirements for redundancy, but the screening device, consisting of a single Auger Monster, does not. There is no room in the existing structure for a second screen or the flow measurement devices. The headworks gate valves upstream of the grit units leak. Each alternative will need to address this deficiency.

The key characteristics of the existing headworks are summarized in Table TM2.2-5 Headworks Summary.

Figure TM2.2-8 Grit Units

Figure TM2.2-9 Auger Monster

Figure TM2.2-10 Septage Receiving Station

Table TM2.2-5 Headworks Summary	
Grit Removal	
Type	Aerated Grit Chamber
Collector	Chain and Bucket (1983)
Number	2
Dimensions, Each	
Length (ft.)	30
Width (ft.)	7.5
Depth (ft.)	6.5
Capacity, Each (MGD)	15.7
Screening	
Fine Screening	Auger Monster (2001)
Number	1
Grinder Tooth Spacing (in)	½
Screen Opening (in)	¼
Horsepower	5
Rated Capacity (MGD)	28.1
Influent Flow Measurement	None

The following table reviews conformance with DEQ regulations.

Table TM2.2-6 Headworks Conformance with DEQ Standards

DEQ Standard 252:656			Regulatory Compliance	
Item	**Criteria**	**Description**	**Existing 7.0 MGD**	**Proposed 8.2 MGD**
Screening				
	3-5(e)	Redundant units	no	no
	13-1(b)(2)	1.75 in. max. screen opening	yes	yes
Degritting				
	3-5(e)	Redundant units	yes	yes
	13-2(d)	Grit washing	yes	yes

Summary conclusions are:

- Existing headwork and grit removal facilities are old, reached their useful life and does not completely meet current DEQ requirements..
- Existing septage station is functionally not adequate.

2.2.3 Primary Clarifiers

Effluent from the headworks flows to the primary clarifier influent channel which distributes the flow to three rectangular primary clarifiers. Sludge is collected using a chain and drag scraper system which moves sludge to the influent end of the clarifier (west end) for discharge to the primary sludge pumps. The clarifiers discharge over finger weirs with V-notches located at the end opposite of the influent. Primary clarifier 1 was constructed in 1934 and upgraded in 1983. Primary clarifiers 2 and 3 (see photograph below) were constructed in 1983. Inlet baffling was added to each clarifier in the early 1990s which significantly improved their performance.

Figure TM2.2-11 Primary Clarifiers

The 1991 CWWTP Predesign Investigation Report, prepared by Black & Veatch, cites historical removal efficiencies of 54% for TSS and 30% for BOD5. Subsequent operational data provided by Veolia Water (plant operator) established a BOD5 removal efficiency of 39.2%. This may reflect the effectiveness of influent baffles which were installed subsequent to the Black & Veatch report.

The DEQ standards limit the hydraulic overflow rate to 1,000 gal/ft^2/day at design average flows and 1,500 gal/ft^2/day for peak hourly flows. At these overflow rates, the primary clarification system has a rated average design capacity of 7.8 MGD and a peak capacity of 11.7 MGD; however, they have demonstrated the ability to perform well at rates exceeding 16.0 MGD.

The primary clarifiers have floating sludge. Veolia reports that DEQ has repeatedly commented on this as an issue that should be corrected. This is most likely due to septic conditions which create H_2S gas. This gas causes the sludge to rise to the surface. This is often caused by the scrapers sitting too high off the floor. This should be addressed by modifying the scrapers to more effectively move the sludge to the hoppers. Increasing the speed of the collectors will also help reduce the amount of time the sludge resides in the clarifier.

Figure TM2.2-12 Effluent Channels and Weirs

Figure TM2.2-13 Peristaltic Pumps

Primary clarifier effluent discharges to an effluent channel which carries it to the aeration basins (Figure TM2.2-12). However, during high flow events, effluent from the primary clarifiers can be diverted to the FEB at this point in the process. The primary clarifier weirs are corroded and in need of replacement and/or leveling.

Table TM2.2-7 Primary Clarifier Summary

Primary Clarifiers	
Type	Rectangular (1934, 1983)
Number	3
Dimensions, Each	
Length (ft.)	130
Width (ft.)	20
Depth (ft.)	1 @ 7.5 and 2 @ 8.5
Rated Average Design Capacity (MGD)	7.8
Peak Design Capacity (MGD)	11.7
Primary Sludge Pumping	
Pump Type	Peristaltic (2006)
Number	3
Horsepower	7.5
Capacity, Each (gpm)	120

Sludge is pumped from the clarifier sludge hoppers to the anaerobic digesters. The original piston type primary sludge pumps were replaced with peristaltic pumps in 2006. The pumps have variable frequency drives and run off of timers (Figure TM2.2-13).

The geometrics of the primary clarifier sludge boxes allow for only one of two to be used at one time. The north hopper of Primary Clarifier #2 cannot be pumped. Veolia reports that it has been cleaned several times. After cleaning, it plugs up again very quickly. The exact cause of this plugging is unknown and will require further investigation using television inspection methods. If the primary clarifiers are retained, these issues should be addressed.

The key characteristics of the primary clarifiers are summarized in Table TM2.2-7.

The following table reviews conformance with DEQ regulations.

Table TM2.2-8 Primary Clarifiers Conformance with DEQ

DEQ Standard 252:656		Regulatory Compliance	
Criteria	**Description**	**Existing 7.0 MGD**	**Proposed 8.2 MGD**
3-5(e)	Redundant units	yes	yes
17-2(b)	SOR 1000 gpd/sf max. average flow	yes	no
17-2(b)	SOR 1500 gpd/sf max. peak flow	no	no
17-2(e)(2)	10,000 gpd/ft average weir loading	yes	yes
17-2(e)(2)	15,000 gpd/ft max. weir loading	no	no
17-2(e)(5)	Freeboard 12" min.	yes	yes
17-3(a)	Scum Removal	yes	yes
17-3(b)	Rapid sludge removal - 3 fps	yes	yes

Summary conclusions:

- The clarifier units are more than 34 years old (Primary Clarifier 1 is more than 64 years old). Units have demonstrated adequate performance. However, to meet the project design flow, additional primary clarifiers may be needed depending on the alternatives selected.
- Improve sludge collection to minimize floating solids and septic conditions.
- Replace weirs and troughs.
- Correct hydraulics through the sludge draw off boxes.

2.2.4 Aeration Basins

Effluent from the primary clarifiers is routed to three aeration basins. Basin 1 is a three-pass plug flow basin with a volume of 0.920 MG. Basin 2 is a three-pass plug flow basin with a process volume of 0.960 MG. Basin 3 is a complete-mix basin with a volume of 0.927 MG. Each basin has a side water depth of 13.5 ft. Basin 1 was constructed in 1934 and modified in 1983. Basin 2 was constructed in 1983, and Basin 3 was constructed in 1993 to boost the nitrification capability of the process to a maximum month flow of 7.0 MGD. Proper flow splitting to the aeration basins continues to be a problem and needs to be remedied in the plan. Effluent from the aeration basins flows to the secondary clarifier splitter structure.

Figure TM2.2-14 Aeration Basin

Aeration is provided by three 250 horsepower centrifugal blowers which were installed in 1983. In 2001 the original coarse bubble aeration equipment in all three aeration basins was replaced with fine-bubble diffusers. This retrofit was performed to increase the transfer efficiency of the aeration system to reduce power costs. The aeration system was designed to meet an Actual Oxygen Requirement (AOR) of 16,327 lb. O2/day. Veolia reports that all three blowers are powered by a common breaker. This resulted in shutdown of all blowers when the breaker failed. The blowers are of old technology (inefficient) and near the end of their useful service life. They should be replaced.

Veolia Water reports that the underground air piping at the plant is corroded and leaks significantly. The buried piping should be replaced with new above ground piping.

New Ceramic discs have been installed in aeration basin #3 (2015), aeration basin #2 (2016) (Figure TM2.2-15), and aeration basin #1 (2017).

The existing CWWTP is a conventional suspended growth activated sludge process. Current DEQ design standards require that the basin volume provide a minimum hydraulic retention time of 6-8 hours and a BOD loading of 30-40 lb BOD_5 per 1000 cft.

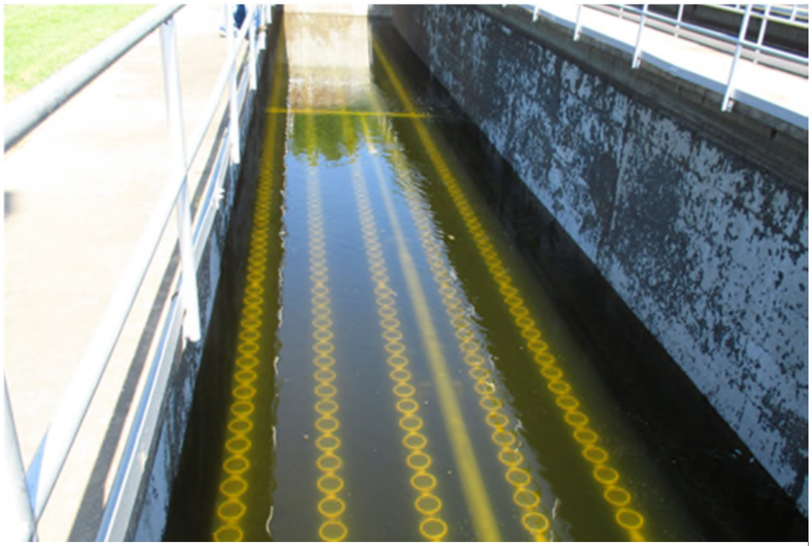

Figure TM2.2-15 Aeration Basin- Ceramic Disc Diffusers

As summarized in Table TM2.2-9, operational performance since the third aeration basin was installed indicates that the current SRT and HRT values produce reliable performance at average flows above 7.0 MGD.

Veolia Water indicates that there is no means to control the flow split to each aeration basin. They would like to see flow meters added.

The key characteristics of the aeration basins are summarized in Table TM2.2-9.

Table TM2.2-9 Aeration Basin Summary

Number	3
Basin Types	
Basin 1 and 2	Plug Flow (1934, 1983)
Basin 3	Complete Mix (1993)
Total Aeration Volume (MG)	2.807
Sidewater Depths (ft.)	13.5
Current Design Flow (MGD)	7
HRT at Design Flow (hrs.) BOD Loading (lb BOD_5 /1000 cft)	9.6 36
Aeration System	
Type	Diffused
Diffuser Type	Ceramic, Fine Bubble (2001) (2015/2016/2017)
Diffuser Submergence (ft.)	12
Maximum Rated Airflow, Total (scfm)	11,600
Design Peak AOR, System (lb. O2/day)	16,327
Blowers	
Number	3
Type	Centrifugal (1983)
Horsepower	250
Capacity (SCFM)	6,100
Discharge Pressure (psig)	6.5
Rated Maximum Month Capacity (MGD)	12.6

The following table reviews conformance with DEQ regulations.

Table TM2.2-10 Aeration Conformance with DEG Standards			
DEQ Standard 252:656			Regulatory Compliance
	Criteria	Description	Existing 7.0 MGD
	16-1(b)	Must be preceded by Primary Treatment	yes
	16-1(d)(2)(C)	At least two basins required	yes
	16-1(d)(2)(E)	Freeboard 18" min.	yes
	16-1(e)(1)	Min. D.O. 2 mg/l	yes
	16-1(e)(1)(B)	4.6 lb O2/lb ammonia	yes
	16-1(e)(1)(B)	Adequate alkalinity 7.14 mg/l per 1 mg/l of NH4-N	yes
	16-1(e)(2)(C)	Multiple Blowers	yes
	16-1(e)(2)(D)	Peak oxygen demand or 200% of average	Yes
	16-1(h)(2)	Multiple RAS Pumps, Firm capacity.	no
	16-1(i)	WAS 0.5% to 25% of average flow	no
	16-1(j)	Flow Measurement (raw sewage, primary effluent, WAS, RAS and air to each basin)	no
	16-3(b)((5)	10 days SRT required for nitrification at low temperature	yes
	Good Practice	Limited to 2,800 mg/L MLSS - clarifier limit.	yes
	16-1(e)(1)(A)	1.8 lb O2/lb BOD	yes
	16-1(e)(2)(A)	1500 cf air/lb. peak BOD	no
	16-1(h)(1)(D)	RAS 50% to 150% of Average Flow	no
	Appendix A	Aeration retention time (6-8 hours)	yes
	Appendix A	12-15 lbs BOD/1000cf/day	no
	Appendix A	0.05-0.10 FM	yes

Numerous regulatory deficiencies were noted at this treatment process. Much of this is attributed to increased organic loadings since 1992, and some are due to regulatory changes.

Summary conclusions are:

- Improve flow splitting between aeration basins.
- Replace existing blowers and underground air piping.
- Add additional blowers to meet BOD demand.
- Consider other aeration processes such as IFAS to allow for increased loadings in existing basins.
- Add RAS and WAS pumping capacity.
- Add flow measurement.

2.2.5 Final Clarifiers and Return Activated Sludge Pumping.

Mixed liquor suspended solids (MLSS) from the aeration basins flow by gravity through a 48-inch-diameter line to a circular final clarifier weir splitter structure which was constructed in 1993. This structure splits the total flow between three rectangular final clarifiers (clarifiers 1, 2 and 3, Figure TM2.2-16) and one circular final clarifier (clarifier 4, Figure TM2.2-17).


Figure TM2.2-16 Rectangular Clarifiers

The split is set up for 48 percent of the MLSS to be diverted to Clarifiers 1 through 3 and 52 percent to Clarifier 4 even though the distribution of clarifier surface area is 69 percent and 31 percent, respectively. This was done to reduce the solids loading to Clarifiers 1 through 3 because of their low sidewater depth (12 feet) and the limited capacity of the sludge collectors (siphons). Operators indicate that the split has performed well.


Figure TM2.2-17 Final Clarifier #4

Sludge collection is accomplished in the rectangular units by floating siphons which move up and down the unit. Sludge removal from the circular unit is accomplished by a mechanical scraper mechanism. Clarifiers 1 through 3 were constructed in 1983 and Clarifier 4 was constructed in 1993.

Each rectangular clarifier is 165 ft. long by 35 ft. wide for a surface area of 5,775 ft^2. Each unit has a sidewater depth of 12 ft. The circular unit has a diameter of 95 ft. and a surface area of 7,088 ft^2. The circular clarifier has a sidewater depth of 15 ft.

DEQ standards limit the surface overflow rate (SOR) for final clarifiers following conventional extended aeration systems to 600 gal/ft^2/day at design average flows and 1,200 gal/ft^2/day at peak hourly flows. At these SORs, the secondary clarifiers have a rated average design capacity of 10.4 MGD and a rated peak capacity of 20.8 MGD. DEQ standards also limit the peak solids loading rate to 35 lb./ft^2/day at peak hourly flows for activated sludge processes. The peak solids loading rate would allow for a peak flow of approximately 28 MGD if both types of clarifiers are considered equal. However, the capacity of the rectangular clarifiers to handle solids is limited as described above. Considering these limitations and based on historical performance of the clarifier documented by system operator, the clarifier system will allow a peak flow of approximately 16.0 MGD before solids washout could be a problem.

Operations staff indicates that maintaining the weirs and launders free of algae and other phototrophic biological growth is difficult on all clarifiers. They devised a traveling chlorinator to run up and down the weir troughs of the rectangular clarifiers, but the troughs are hard to reach with the wash-down hoses. FC#1 has brushes that do a good job on weirs, but do not clean outside of the trough. A high-pressure spray system is needed to clean the bottom of the troughs. FC#2, FC#3 and FC#4 have no algae cleaning. All of the clarifiers need to have automated weir washers installed to control algae growth.

Figure TM2.2-18 Final Clarifier #2

The traveling mechanism for FC#2 will not travel to the east wall (Figure TM2.2-18). It stops about 6 ft. short. This results in sludge accumulation and water quality issues in that portion of the tank. The mechanism needs to be repaired or replaced.

The capacity of the siphons limits the allowable solids loading to the clarifiers as described above. In the past the operators have limited the MLSS in the aeration basins to approximately 2,800 mg/L to accommodate the clarifier limitations.

Return activated sludge (RAS) is collected and pumped by a return sludge pump station located adjacent to the rectangular clarifiers. The station includes two vertical non-clog RAS pumps and two waste activated sludge (WAS) pumps. The RAS pumps are controlled by VFDs. One VFD was installed in 1993 and the other in 2000. The firm capacity of the RAS pump station (one pump operating) is 5.4 MGD. Using the DEQ standard for conventional activated sludge aeration (required RAS rate of 75 percent of average flow), the average flow supported by the firm capacity of the RAS pumps is 3.6 MGD. This rate does not meet the current DEQ design standards even at current flows. However, , but the operators report that they have not had capacity issues with the RAS pumping system.

Figure TM2.2-19 Return Sludge Pump Station

WAS is pumped to the WAS dissolved air floatation thickener by two vertical, non-clog pumps each with a capacity of 160 gpm. These pumps were installed in 1983. One pump can support influent flows up to 0.92 MGD per DEQ design criteria requiring pumping of 25% of average daily flow.

The key characteristics of the final clarifiers are summarized in Table TM2.2-11.

Table TM2.2-11 Final Clarifiers and RAS/WAS Pumping	
Final Clarifier Basins (Clarifiers 1 – 3)	
Type	Rectangular (1983)
Length x Width (ft.)	165 x 32
SWD (ft.)	12
Surface Area, Each (ft2)	5,775
Total Surface Area (ft2)	17,325
Sludge Removal	Traveling Siphon
Final Clarifier Basin (Clarifier 4)	
Type	Circular (1993)
Diameter (ft.)	95
SWD (ft.)	15
Surface Area (ft2)	7,088
Sludge Removal	Circular Scraper
Total Surface Area (ft2)	24,413
Total Rated Capacity, Average (MGD)	9.7
Total Rated Capacity, Peak Hour (MGD)	24.4
RAS Pumping	
Type	Vertical, Non-Clog (1983)
Number	2
Horsepower	40
Drive	VFDs (1993 and 2000)
Capacity, Each (gpm)	3,760
Capacity, Firm (MGD)	5.4
WAS Pumps	
Type	Vertical, Non-Clog (1983)
Number	2
Horsepower	5
Drive	Starters (1983)
Capacity, Firm (gpm)	160

The following table reviews conformance with DEQ regulations.

Table TM2.2-12 Final Clarifier Conformance with DEQ Standards				
DEQ Standard 252:656			**Regulatory Compliance**	
Item	**Criteria**	**Description**	**Existing 7.0 MGD**	**Proposed 8.2 MGD**
Secondary Clarifiers				
	Appendix B	Min 12 ft side water depth	yes	yes
Extended Aeration and Nitrification				
	Appendix B	SOR 600 gpd/sf max. average flow	yes	yes
	Appendix B	SOR 1200 gpd/sf max. peak flow	yes	yes
	Appendix B	SLR 35 lb/day/sf	yes	yes

Regulatory deficiencies for RAS and WAS pumping were identified in the previous section, 2.2.4 Aeration Basin regulatory compliance table. The existing secondary clarifiers comply with the regulations.

Summary conclusions are:

- Increase WAS Pumping.
- Improve algae removal for all clarifiers.
- Repair FC#2 to allow bridge to travel to the east wall.

2.2.6 Effluent Flow Measurement.

The final clarifiers discharge to the disinfection system. The effluent flow rate is measured between the final clarifiers and the chlorine contact tank by a Parshall flume. The flume has a 36-inch throat and a capacity of approximately 23 MGD (assuming no downstream effects). The flume was installed in 1983 and remains in good condition (Figure TM2.2-20). The instrumentation is also in good condition

No regulatory deficiencies were noted at this location. No improvements are required.

Figure TM2.2-20 Parshall Flume

2.2.7 Effluent Disinfection and Sampling

Effluent from the final clarifiers is routed to the disinfection system. Flow is split between two chlorine contact basins, each with a volume of 80,500 gallons. Chlorine gas is converted to chlorine solution and used as the disinfection agent. The solution is fed upstream of the splitter structure via 2 feeders, each rated at 500 ppd. The feeders are flow paced. The existing chlorine storage facility is in good condition and has adequate storage for a maximum of four 1-ton cylinders. DEQ requires that 30 days of storage be provided which allows for flows of 4.0 MGD.

Three (3) gates do not operate properly; these need to be replaced. Additionally, some failing concrete needs to be repaired around several gate stands (Figure TM2.2-21, Figure TM2.2-22, and Figure TM2.2-23). The chlorination system, including the feeders, was installed in 1983 and remains in good condition.

DEQ standards require a minimum contact period of 15 minutes at peak hourly flow rates. At that contact period, the chlorine contact basins are rated at a peak hourly capacity of 15.5 MGD. ODEQ construction standards require a minimum feed capacity of 8 mg/l at average flow. At that feed concentration, each existing chlorine feeder has a capacity of 7.5 MGD for a combined capacity of 15 MGD. Each has manual feed rate controllers for redundancy.

Figure TM2.2-21 Effluent Pump Station- 1 of 3 Gates to be Replaced

Figure TM2.2-22 Chlorine Contact Basins 2 of 3 to be Replaced

Figure TM2.2-23 Concrete Failure by Stand

Dechlorination is accomplished by feeding a sulfur dioxide solution at the discharge weir of the chlorine contact basins. The sulfur dioxide storage facility was constructed in 1993 and is in good condition. The facility has two feeders, one with a capacity of 100 lbs./day, and the other with a capacity of 200 lbs./day. Storage is available for up to four 1-ton cylinders. At a peak flow of 15.5 MGD, the dechlorination feed system can neutralize a maximum chlorine residual concentration of 2.3 mg/L. Each feeder includes a manual feed rate controller for redundancy. At 2 mg/L, the sulfur dioxide feed system can handle approximately 18 MGD.

An effluent composite sampler is located at the discharge of the disinfection unit. The effluent composite sampler needs to be placed in an enclosure to prevent freezing problems during the winter months.

City staff expressed interest in converting the disinfection system at this plant from chlorination to ultraviolet irradiation due to the safety concerns presented by chlorine and sulfur dioxide gas.

If UV is selected, the Chickasaw plant will need to retain some chlorination capability to allow the operators to use chlorine solution for filament control in the activated sludge process. Chlorine may also have some benefits for water reuse.

The key characteristics of the effluent disinfection system are summarized in Table TM2.2-13 Effluent Disinfection Summary.

Table TM2.2-13 Effluent Disinfection Summary

Type	Chlorination/Dechlorination
Chlorination	(1983)
Chemical	Chlorine Gas
Feeders	2 (1983)
Dosage Control	Flow Paced
Capacity, Each (ppd)	500
Storage Capacity, No. of 1-ton cylinders	4
Capacity @ 8 mg/L concentration (MGD)	15
Chlorine Contact Basins	
Number	2
Volume, Each (gal)	80,500
Total Volume (gal)	161,000
Rated Capacity (MGD)	15.5
Dechlorination	(1993)
Chemical	Sulfur Dioxide
Feeders	2
Dosage Control	Flow Paced
Capacity, Each (ppd)	1 @ 100 and 1 @ 200
Storage Capacity, No. of 1-ton cylinders	2
Rated Capacity @ 2 mg/L (MGD)	18.0

The following table reviews conformance with DEQ regulations.

Table TM2.2-14 Effluent Disinfection Conformance with DEQ Standards

DEQ Standard 252:656			Regulatory Compliance
Item	Criteria	Description	Existing 7.0 MGD
Disinfection			
	21-1(d)	Alarms to warn of leaks	yes
	21-1(e)	Multiple units, firm capacity	yes
Chlorination			
	21-2(a)(2)	Sized for 8 mg/l dosage (Design Flow)	yes
	21-2(b)(2)(A)	Category 3 & 4 water reuse - 15 minutes at peak flow	yes
	21-2(b)(3)	Multiple tanks.	yes
	21-2(b)(3)	Skimmer.	yes
	21-2(c)(1)	Separate store and feed areas.	yes
	21-2(e)	Scales and recorder to weigh cylinders	yes
	21-2(f)	One ton containers for facilities exceeding 150 lbs/day.	yes
	21-2(m)	Dechlorination to less than 0.1 mg/l.	yes
	252:626-11-3(m)	30 day storage required	no
	252:626-11-3(m)	Dechlorination sized for 2 mg/l dosage (Peak)	yes

Summary conclusions are:

- Repair several slide gates.
- Either upsize chlorination equipment to meet current regulations or convert to UV disinfection system.
- Add an enclosure for the effluent sampler.

2.2.8 Effluent Pump Station

Effluent from the chlorine contact basins flows by gravity to the Caney River outfall under normal discharge conditions. However, during high flood events effluent pumping is required. An effluent pump station is located adjacent to the chlorine contact basins. The station consists of a wet well and two vertical turbine pumps, each with a capacity of 5,400 gpm. The station was constructed in 1983.

The key characteristics of the effluent pump station are summarized in Table TM2.2-15.

Table TM2.2-15 Effluent Pump Station Summary

Effluent Pumps	
Type	Vertical Turbine (1983)
Number	2
Horsepower, Each	25
Capacity, Each, (gpm)	5,400
Controls	Starters (1983)
Firm Capacity (MGD)	7.8
Maximum Capacity (MGD)	15.6

This pump station will need to be upsized to match the rest of the facility design to handle a 2:1 peaking factor (16.4 MGD).

The following table reviews conformance with DEQ regulations.

Table TM2.2-16 Effluent Pump Station Conformance with DEQ Standards

DEQ Standard 252:656		**Regulatory Compliance**
Criteria	**Description**	**Existing 7.0 MGD**
7-1(b)	Multiple pumps	no
7-1(c)(1)	Force main 4"min; velocity 2 fps min	yes

Summary conclusions are:

- Effluent pumps do not have the required firm capacity. A third 5,400 gpm pump is required to provide a firm capacity of 10,800 gpm (15.6 MGD).
- Effluent reuse pump station will allow use of reuse water for in-plant non-potable reuse.

2.2.9 Outfall and Effluent Aeration

The CWWTP discharges to the Caney River from an outfall headwall located on the south bank of the river. Under normal conditions, the effluent cascades down a rip-rapped surface before contacting the river flow. The existing outfall headwall was constructed in 1995 and is in good condition.

No current regulatory deficiencies were noted.

2.2.10 Other Condition Assessment Items

- Where practical buried valves need to have manholes constructed around them to allow for access for maintenance, repair, replacement, and operation.
- An arc-flash study should be performed to assess the condition of the aging motor control centers (MCC).
- Standby Power. The facility has a 750 kW generator that was installed in 1983. To add backup power for the existing blowers, as required by the 2015 DEQ Standards, would require a 1000 kW generator or a second source of power from the electric utility provider.

The following table reviews conformance with DEQ regulations.

Table TM2.2-17 Standby Power Conformance with DEQ Standards

DEQ Standard 252:656			Regulatory Compliance
Item	Criteria	Description	Existing 7.0 MGD
Standby Power			
	9-2(a)	Pumping	yes
	9-2(a)	Aeration	no
	9-2(a)	Disinfection	yes

2.3 SOLIDS TREATMENT PROCESS

The following summarizes the solids treatment unit processes and sludge disposal at the CWWTP. A process flow schematic and a site plan of the CWWTP are shown on Figure TM2.2-4and Figure TM2.2-5, respectively.

2.3.1 WAS Thickening

WAS is pumped from the secondary clarifiers to a dissolved air flotation (DAF) unit for thickening prior to pumping to the anaerobic digesters (Figure TM2.2-24). The WAS pumps are located adjacent to the RAS pumps. According to the operations staff, these pumps function well.

There is only one DAF unit, and DEQ standards require redundancy. The operators indicate that the DAF is near the end of its useful life and it needs to be replaced. Other thickening technologies such as rotary drum thickeners should be considered. This issue will be addressed in the alternative development. Critical unit information is summarized below. The DAF unit was installed in 1983.

The key characteristics of the DAF unit are summarized in Table TM2.2-18.

Figure TM2.2-24 Dissolved Air Flotation Thickener

Table TM2.2-18 WAS Thickening Summary

WAS Thickener	
Type	Dissolved Air Flotation (1983)
Number	1
Effective Area (ft^2)	200
Allowable Loading ($lb./hr./ft^2$)	1.5

The following table reviews conformance with DEQ regulations.

Table TM2.2-19 WAS Thickening Conformance with DEQ Standards

DEQ Standard 252:656		**Regulatory Compliance**
Criteria	**Description**	**Existing 7.0 MGD**
19-2(a)(1)	Multiple units	no
	1.5 lb/hr/sf (MOP-8, pg. 1174)	no

Summary conclusions:

- Existing DAF unit is near its useful life and requires replacement or an alternative technology (discussed in a separate another Technical memorandum).

2.3.2 Anaerobic Digestion

Sludge digestion is accomplished by anaerobic digestion. Primary sludge and thickened secondary sludge from the DAF unit is routed to two primary anaerobic digesters (Figure TM2.2-25). The feed to the digesters is switched between raw primary sludge and thickened WAS each day. This needs to be automated if the anaerobic digesters are retained.

The primary digesters have fixed covers and are mixed and heated. Each primary digester has a volume of 40,000 cf (0.296 MG). The primary digesters discharge to a secondary digester with a volume of 80,900 cf (0.605 MG). The secondary digester has a floating cover and is unmixed and unheated. Although some VS reduction will occur in the secondary digester, without heating and mixing it is considered storage and not part of the stabilization process. At the DEQ standard loading rate of 80 lbs. solids/1000 cf, an average influent TSS value of 200 mg/L, a 54% removal rate for TSS, and a WAS yield of 0.80 in the activated sludge system, the primary digesters can support an influent flow of 6.4 MGD

Figure TM2.2-25 Primary Digesters

Primary Digester 2 was constructed in 1934 and upgraded in 1983 (Figure TM2.2-26). Primary Digester 1 and the secondary digester were constructed in 1983. The gas mixing systems in the primary digesters were replaced with a hydraulic (RotoMix nozzles) system in 2006. The recirculation pumps were also replaced and the digester covers were rehabilitated. The sludge heaters use digester gas for an energy source when enough gas is produced by the system. Natural gas is used to supplement the digester gas. The heat exchangers (2) were part of the 1983 improvements (Figure TM2.2-28). They need to be replaced if anaerobic digestion is retained.

All of the digesters need considerable improvements. Anaerobic digesters (AD) #1 and #2 have issues with floating covers hanging up and rotating off track. Both floating covers, piping, and equipment have significant corrosion that at a minimum require blasting and recoating and may require replacement. AD #2 has temporary piping installed on the roof that must be replaced.

AD#3 is missing a flame arrester and PRV (Figure TM2.2-27). These are required for safe operation. The floating cover, equipment, and piping needs to be cleaned and painted.

The key characteristics of the anaerobic digesters are summarized in Table TM2.2-20 Anaerobic Digestion Summary.

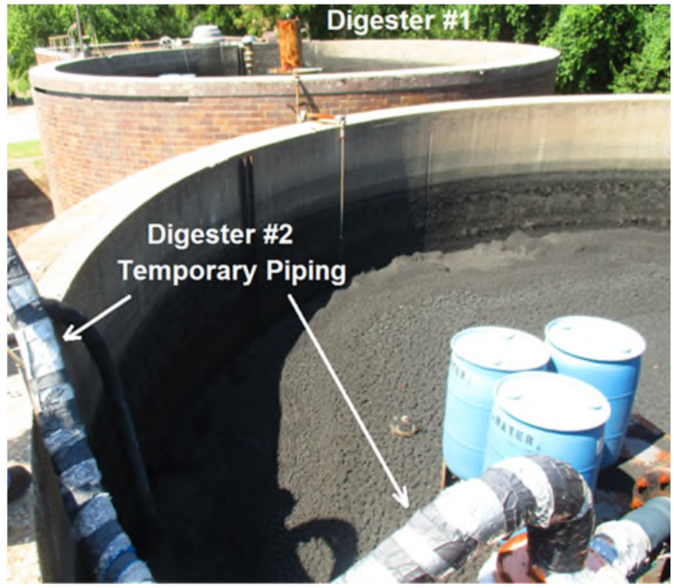

Figure TM2.2-26 Primary Digesters (AD#1 & AD#2)


Figure TM2.2-27 Secondary Digester (AD#3)

Figure TM2.2-28 Digester Heaters

Table TM2.2-20 Anaerobic Digestion Summary	
Primary Anaerobic Digesters	
Type	Mixed, Heated (1934, 1983)
Number	2
Diameter (ft.)	45
Volume, Each (gal)	296,000
Total Volume (gal)	592,000
Heating	
Type	Countercurrent Tube (1983)
Number	2 (1 per primary digester)
Capacity, Each (BTU/hr.)	750,000
Recirculation Pumps	
Type	Centrifugal (2006)
Number	4 (2 per primary digester)
Horsepower	10
Capacity (gpm)	380
Mixing	
Type	RotoMix (nozzles) (2006)
Pumps	
Type	Centrifugal (2006)
Number	4 (2 per primary digester) (2006)
Horsepower	15
Capacity (gpm)	900
Secondary Anaerobic Digester	
Type	Unmixed/Unheated (1983)
Number	1
Diameter (ft.)	60
Volume, each, (gal)	605,000

The following table reviews conformance with DEQ regulations.

Table TM2.2-21 Anaerobic Digestion Conformance with DEQ Standards		
DEQ Standard 252:656		**Regulatory Compliance**
Criteria	Description	Existing 7.0 MGD
19-2(a)(1)	Multiple tanks	yes
19-2(a)(2)	Sidewater Depth 20 ft. min.	yes
19-2(a)(3)	Slope 1:4 for gravity removal	yes
19-3(c)(2)	35 degrees C for 15 days SRT	no
19-2(c)(3)(B)	80 lbs/1000cf/day VS Loading	no
19-2(d)	Gas collection, waste gas burners, gas production meter.	no
19-2(e)	Supernatant withdrawal & sampling	yes
19-2(f)	Temperature probe and recording	no

Summary conclusions are:

- Due to condition, complete rehabilitation of the digesters or converting to aerobic digestion is necessary.
- Consider adding mixing and heating for Digester #3.

2.3.3 Digested Sludge Thickening

Digested sludge from the anaerobic digesters is pumped to a gravity belt thickener (GBT). The GBT facility was constructed in 2001 (Figure TM2.2-29).

Figure TM2.2-29 Gravity Belt Thickener Building

The building houses a digested sludge feed pump, a 2-meter GBT, a thickened sludge transfer pump, and polymer equipment. The capacity of the thickener is 2,500 lbs./hour. Thickened sludge is pumped directly to liquid transport trucks for subsequent land application or to the digested sludge storage tank. The sludge is thickened to an average of approximately 3.5 percent solids before storage. A redundant thickener is not in place; however, the digested sludge can be thickened by decanting in the secondary digester and the holding basin when the GBT is out of service.

Operations staff indicates that the thickener works well; however, the heating and ventilation system needs improvement. During the winter months, the building heating system is unable to maintain a suitable temperature while the building is ventilated. The ventilation makeup air heating units need to be replaced or expanded.

The key characteristics of the digested sludge thickener are summarized in Table TM2.2-22.

Table TM2.2-22 Digest Sludge Thickening Summary

Thickener	
Type	Gravity Belt Thickener (2001)
Number	1
Belt Width (m)	2
Capacity (lb. DS/hr.)	2,500
Digested Sludge Feed Pump	
Type	Progressive Cavity (2001)
Number	1
Horsepower Each	15
Capacity (gpm)	300 (variable)
Controls	Starter
Thickened Sludge Transfer Pump	
Type	Progressive Cavity (2001)
Number	1
Horsepower, each	15
Capacity, (gpm)	300
Drive	Variable Frequency
Capacity (MGD)	28

The following table reviews conformance with DEQ regulations.

Table TM2.2-23 Sludge Thickening Conformance with DEQ Standards

DEQ Standard 252:656		Regulatory Compliance	
Criteria	Description	Existing 7.0 MGD	Proposed 8.2 MGD
19-2(a)(1)	Multiple units.	no	no

Summary conclusions are:

- Existing building heating and ventilation system needs upgrade.
- Sludge can be processed without the gravity thickener which meets the intent of the redundancy unit requirements.

2.3.4 Thickened Sludge Storage

Thickened sludge from the GBT is normally pumped to the sludge storage tank. It can also be pumped directly to the liquid transport trucks.

Figure TM2.2-30 Sludge Storage Basin

The sludge storage basin is an unused final clarifier (originally constructed in 1934) that was converted to sludge storage and it has a capacity of 325,000 gallons (Figure TM2.2-30). It is an unmixed basin, but the operators also manually decant supernatant from the storage tank to increase the solids content of the stored sludge as necessary. The CWWTP also uses the unheated/unmixed secondary anaerobic digester as storage. The total storage volume including the secondary anaerobic digester and the storage tank is 930,000 gallons. As previously discussed, the CWWTP does not conform to DEQ sludge digestion regulations and will require that this digester be used for sludge stabilization - not storage. The CWWTPs firm sludge storage capacity (325,000 gallons) is approximately 14 days.

The plant operators indicate that the basin does not have enough capacity to get through winter or extended rain periods.

The key characteristics of the sludge storage basin are summarized in Table TM2.2-24.

Table TM2.2-24 Thickened Sludge Storage Summary

Storage	
Type	Converted Final Clarifier (1934)
Volume (gal)	325,000
Storage @ 7 MGD (Months)	<1

The following table reviews conformance with DEQ regulations.

Table TM2.2-25 Sludge Storage Conformance with DEQ Standards

DEQ Standard 252:656		Regulatory Compliance	
Criteria	Description	Existing 7.0 MGD	Proposed 8.2 MGD
19-6	Provide 3 to 6 months of storage	no	no

Summary conclusions are:

- Additional dewatering and storage facility are needed.

2.3.5 Sludge Disposal Equipment

Sludge from the storage basin is pumped to transport trucks using a portable, engine-driven (Sykes) pump (Figure TM2.2-31). This pump is also used to mix the basin so that the sludge does not become too thick for the transport trucks. This pump works well; however, it is also needed at other locations. The plant operator would like to see a permanent stationary pump(s) installed for this basin.

Figure TM2.2-31 Sludge Transfer Pump (SYKES)

Two transport trucks are used to move the liquid sludge to the land application sites and transfer the sludge to an injection unit (Figure TM2.2-32). Sludge is usually directly injected into the ground. Equipment is available for surface spreading and incorporation by a disc.

The potential for converting the land application disposal of sludge from a liquid system to a dewatered sludge (cake) system was discussed. Right now most of the land used for biosolids application is pasture land that is not conducive to cake application due to the requirement in Oklahoma that all surface spreading must be incorporated within 24 hours. Liquid disposal allows for direct injection which does not significantly disturb the ground surface. Incorporation of cake would greatly disturb the land surface, and the land owners likely would not agree to that approach.

Figure TM2.2-32 Sludge Transport Trucks

Veolia Water indicates that one transport tanker is in fair condition and one is in poor condition. The cost of replacement for the tanker was not included in the analysis of the alternatives.

The key characteristics of the sludge disposal equipment are summarized in Table TM2.2-26.

Table TM2.2-26 Sludge Handling and Disposal	
Digested Sludge Loading	
Type	Engine-Driven, Self-Primed (Sykes) Pump (1993)
Capacity (gpm)	300
Transport Trucks/Trailers	
Type	Tractor/Trailer (1 unit 2000) (1 unit 2009)
Number	2
Tank Capacity (gal)	1 @ 5,000 and 1 @ 6,000
Application Equipment	
Injector	Calumet V3250 w/ 3,250 gallon tank (2003)
Tractor	Case IH, MX 230 (2003)
Disc	International 770, 13.5 ft.
Chisel	Brushing, 12 ft.

Summary conclusions are:

- If liquid storage is retained, add an additional pump or a dedicated pump for sludge disposal.
- Add additional on-site liquid storage or add a sludge dewatering facility with covered storage to reduce the amount of product to be stored on-site and transported to final disposal.

2.4 SUPPORT FACILITY

The following summarizes the support facilities at the CWWTP.

2.4.1 Administration and Laboratory Building.

A 4,190 ft^2 administration and laboratory building was constructed during the major plant expansion project in 1983. Foundation problems have plagued the building since it was first constructed.

The building has many serious structural cracks and a mold problem. A new building with comparable facilities is needed. It will be assumed that all laboratory equipment will be reused from the old building. It was also noted that the laboratory space in the existing building is limited and additional space is needed.

The administration building had approximately 5' of water in it during the 1986 flood (500-year). It also had water in it during the 100-year flood of 2007.

The new building should be constructed at a higher elevation. Consideration should be given to diking the existing Chickasaw plant site for flood protection.

The improvements should include the addition of a SCADA system that monitors the CWWTP and lift stations. The plant operator would also like to see an Automation Control System mounted in each aeration tank as part of the SCADA.

Figure TM2.2-33 Administration and Laboratory Building

Figure TM2.2-34 Maintenance Building

2.4.2 Maintenance Building.

A 4,800 ft^2 maintenance building was constructed in 1984 and provides adequate room for parts storage and maintenance activities.

The sludge transport trucks are also kept in the building. The building is in fair condition and no improvement is necessary.

Amendment to Wastewater Treatment Facility Plan and Reuse Feasibility Study

Technical Memorandum No. 2.1

Alternative 1 Evaluation- Maintain all Flows at Existing Chickasaw Wastewater Treatment Plant

For:

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Tetra Tech Project No. 200-11458-16002

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ACRONYMS/ABBREVIATIONS

Acronyms/Abbreviations	Definition
AD	Anaerobic Digesters
AOR	Actual Oxygen Reduction
BOD	Biochemical Oxygen Demand
BOD	Biological Oxygen Demand
CCWWTP	Chickasaw Wastewater Treatment Plant
DAF	Dissolved Air Flotation
DEQ	Department of Environmental Quality
DS	Dry Solids
FC	Final Clarifier
FEB	Flow Equalization Basin
IFAS	Integrated Fixed Film Activated Sludge
GPCD	Gallons per Capita per Day
MGD	Million Gallons per Day
MBBR	Moving Bed Biofilm Reactor
MLSS	Mixed Liquor Suspended Solids
NH3-N	Ammonia Nitrogen
RAS	Return Activated Sludge
SCADA	Supervisory Control and Data Acquisition
SOR	Surface Overflow Rate
TBD	To be Determined
TM	Technical Memorandum
TSS	Total Suspended Solids
VFD	Variable Frequency Drive
WAS	Waste Activated Sludge
WW	Wastewater

1.0 INTRODUCTION

Technical Memorandum No.2.1 (TM-2.1) summarizes Alternative 1 CCWWTP improvements required for treating all flows at the Chickasaw CWWTP. Alternative 1 is updated to reflect current conditions, current construction costs, and the revised flow and plant loadings developed for the 2050 planning year (30-year planning period). Multiple options are considered.

- Option 1 upgrades and expands the existing process. This includes new headworks screening and grit removal; additional primary clarifier; additional aeration; new secondary clarifiers; filtration; UV disinfection; additional effluent pumping; additional anaerobic digestion; sludge dewatering; and a new Administration and Laboratory building.

- Option 2 upgrades and expands the existing process using extended aeration and aerobic digestion. This option eliminates the existing primary clarifiers and the existing rectangular final clarifiers. This approach converts the anaerobic digesters to aerobic digesters.

Under Alternate 1 CWWTP improvements, additional land will be required adjacent to the existing CWWTP site. The plant site as well as the surrounding areas are in the floodway and/or in the flood plain. Therefore, mitigation measures will be required to expand at the existing plant site.

The process design loadings used to develop these options are provided in the following table.

Table TM2.1-1 SUMMARY OF DESIGN CRITERIA

Parameter	Peaking Factor	2013, 2014, 2015			Planning Year 2050		
		Flow (MGD)	Conc. (mg/L)	Mass (lbs/day)	Flow (MGD)	Conc. (mg/L)	Mass (lbs/day)
Process Flow							
Average Annual Daily	--	7.021	--	--	8.21	--	--
Max Mo. Average Daily	1.37	9.632	--	--	11.65	--	--
Max. Day[1]	3.99	28.000	--	--	16.42	--	--
Influent BOD							
Average	--	--	202	11,800	--	199	13,600
Avg. Day of Max. Mo.	1.30	--	190	15,300	--	--	17,600
Max. Day.	3.03	--	641	35,700	--	--	40,800
Influent TSS							
Average	--	--	330	19,300	--	323	22,100
Avg. Day of Max. Mo.	1.41	--	337	27,100	--	320	31,100
Max. Day.	3.10	--	257	59,900	--	293	68,500
Influent NH_3-N							
Average	--	--	18	1,070	--	18	1,220
Avg. Day of Max. Mo.	1.46	--	19	1,560	--	18	1,780
Max. Day.	6.14	--	28	6,570	--	32	7,490
Influent Alkalinity, Min.[2]	--	--	--	--	--	228	--
Observed Yield - Raw Sludge:		0.80	lbs DS/lbs BOD				

[1]Assumes Flow above the 2050 Max. Day will diverted to a FEB.
[2]50th Percentile Influent alkalinity value from 2012-2017 plant data.

2.0 TREATMENT FACILITIES IMPROVEMENTS

2.1 OPTION 1 – UPGRADE AND EXPAND THE EXISTING PROCESSES

This option is an update of the 2010 evaluation. It maintains as many of the existing treatment processes as possible. It includes additional improvements that are required due to the system aging and by DEQ regulations. Where applicable, the cost of improvements from the 2010 report was used and adjusted by ENR Construction Cost index.

Following is a summary of recommended and necessary improvements.

2.1.1 Chickasaw Lift Station

The lift station pump capacity is adequate for this option. However, a second grinder unit is needed for redundancy and to protect the pumps. The southeast wall needs repair to prevent groundwater seepage from migrating into the station. The following improvements are proposed:

- Add a second influent grinder for redundancy.
- Repair the southeast wall to mitigate groundwater seepage.
- Adjust/relocate air duct to above maximum water level.
- The valve separating the wet wells is inoperable and needs to be replaced.

2.1.2 Flow Equalization Basin (FEB)

The Chickasaw FEB volume is adequate for this option. The synthetic liner was recently replaced by the plant operator. Such liners have a typical service life of 30 years which should last through the 30-year planning period. Currently the flow diversion to the FEB is manually achieved, and there is no flow measurement for process control. A new flow diversion structure with flow measurement and automated flow diversion control is recommended. The following improvements are proposed:

- New flow diversion and control structure with automatic diversion gate.
- Add flow measurement to measure flow into and out of the FEB.

2.1.3 Headwork and Degritters

Currently there is a single augur grinder unit exposed to the weather. DEQ standards require redundancy. There is not enough space adjacent to the existing unit to accommodate a second unit. A new headwork structure is proposed adjacent to and upstream of the existing headwork facility. The structure will collect all of the influent flows (from three forcemains) and will have two parallel channels to accommodate redundant screens (such as channel mounted fine screens instead of Augur Monster existing now). One will be the existing augur screen relocated to the building, and the second will be the new screen. The structure will also have a flow measurement device.

The existing degritters were last rehabilitated in 1993. The degritters meet the DEQ requirements for redundancy, but the screening device, consisting of a single auger monster, does not. There is no room in the existing structure for a second screen or the flow measurement devices. The headwork gate valves upstream of the grit units leak. The existing grit unit is much less efficient than newer technologies such as a vortex type grit removal system. A new vortex type grit system is recommended.

The existing septic truck receiving facility also needs rehabilitation to provide a holding volume for testing and verification and to screen the septic dump prior to discharge to the plant process. The following improvements are proposed:

- New headwork structure with dual screening facility.
- New vortex type dual grit removal facility.
- Add a second influent grinder for redundancy.
- Repair the southeast wall to mitigate groundwater seepage.
- Adjust/relocate air duct to above maximum water level.
- New septic truck receiving station.

2.1.4 Primary Clarifiers

Primary Clarifier 1 was constructed in 1934 and upgraded in 1983. Primary Clarifiers 2 and 3 (see photograph below) were constructed in 1983. The concrete structure shows signs of aging but generally appears in satisfactory condition. However, the sludge removal mechanism, the overflow weir, and baffles need rehabilitation.

DEQ standards limit the hydraulic overflow rate to 1,000 gal/ft^2/day at design average flows and 1,500 gal/ft^2/day for peak hourly flows. At these overflow rates, the primary clarification system has a rated average design capacity of 7.8 MGD and a peak capacity of 11.7 MGD; however, they have demonstrated the ability to perform well at rates exceeding 16.0 MGD. Based on the historical performance of the primary clarifiers and the availability of limited space within the plant footprint for additional primary clarifiers, the DEQ will be requested to grant a variance for the exceeding the DEQ design criteria. Therefore, new additional primary clarifier volume is not included in the report. However, the following improvements are included to rehabilitate existing mechanisms:

- Rehabilitate/replace sludge removal mechanisms on existing primary clarifiers.
- Replace weirs and baffles on the existing primary clarifiers.
- Investigate and correct hydraulic bottleneck within the primary clarifier bottom sludge draw-off sump.

2.1.5 Aeration Basin

Basin 1 is a three-pass plug flow basin with a volume of 0.920 MG. Basin 2 is a three-pass plug flow basin with a process volume of 0.960 MG. Basin 3 is a complete-mix basin with a volume of 0.927 MG. Each basin has a side water depth of 13.5 ft. Basin 1 was constructed in 1934 and modified in 1983. Basin 2 was constructed in 1983, and Basin 3 was constructed in 1993 to boost the nitrification capability of the process to a maximum month flow of 7.0 MGD. Proper flow splitting to the aeration basins continues to be a problem and needs to be remedied in the plan. Aeration basin influent valves are deteriorating and needs replacement

The existing CWWTP is a conventional suspended growth activated sludge process. Current DEQ design standards require that the basin volume provide a minimum hydraulic retention time (HRT) of 6-8 hours and a BOD loading of 30-40 lbs BOD_5 per 1000 cft. Three existing basins together provide a process volume of 2.807 MG. This volume will provide approximately 8.2 hours HRT at the 2050 design flow of 8.21 MGD. On the basis of BOD loading, assuming DEQ loading criteria of 35 BOD_5 per 1000 cft, the existing basin volume will support BOD_5 of 13,134 lbs/day. The projected 2050 average and maximum month influent BOD_5 loading are 13,600 lbs/day and 17,600 lbs/day, respectively. Assuming 35% BOD reduction in the primary clarifier, the projected BOD_5 loading to the aeration basins are 8,840 lbs/day and 11,440 lbs/day. Therefore, the existing aeration basin volume is adequate for the 2050 projected loading.

The three existing 250-hp blowers are at the end of their useful life. In 2001 the original coarse bubble aeration equipment in all three aeration basins was replaced with fine-bubble diffusers. This retrofit was performed to increase the transfer efficiency of the aeration system to reduce power costs. The aeration system was designed to meet an Actual Oxygen Requirement (AOR) of 16,327 lb. O_2/day. However, to meet the projected peak loading, approximately AOR of 25,000 lb. O_2/day will be needed. Therefore, new blowers will be needed. As part of the blower system rehabilitation, the cost benefit of turbo blowers in lieu of the centrifugal blowers and incorporation of automatic dissolved oxygen based control should be evaluated as part of the pre-design phase.

The underground air piping at the plant is corroded and leaks significantly. The buried piping should be replaced with new above-ground piping.

The firm capacity of the RAS pump station (one pump operating) is 5.4 MGD. Using the DEQ standard for standard rate (required RAS rate of 75 percent of average flow or 6.2 MGD), the RAS pumping does not have adequate capacity. Therefore, additional RAS pumping capacity will be required.

WAS is pumped to the WAS dissolved air floatation thickener by two vertical, non-clog pumps each with a capacity of 160 gpm. These pumps were installed in 1983. One pump can support influent flows up to 0.92 MGD per DEQ design criteria requiring pumping of 25% of average daily flow Therefore, the WAS pumping will be expanded to provide approximately 2 MGD firm capacity.

The following improvements are proposed for the aeration basins and the blower system:

- Modify the primary clarifier effluent channel and the inlets to the aeration basins to improve the flow split between the basins.
- Replace existing three 250-hp blowers and add two additional blowers to meet the peak oxygen requirements. A total of five 250-hp blowers will be needed. The existing blower building will need to be expanded to accommodate the new blowers.
- The existing basin volume is adequate. Add additional diffusers to the existing basins to meet the increased oxygen demand.
- Replace underground air piping between the blowers and the aeration basins with above-ground air piping.
- Add flow measurement on the basin effluent line to monitor each basin flow.
- Add additional RAS and WAS pumping capacity.
- Improve RAS flow split between the aeration basins.

2.1.6 Final Clarifiers

DEQ standards limit the surface overflow rate (SOR) for final clarifiers following conventional activated sludge systems with single-stage nitrification process (which is CWWTP) to 400 gal/ft^2/day at design average flows and 1,000 gal/ft^2/day at peak hourly flows. At these SORs, the secondary clarifiers have a rated average design capacity of 9.7 MGD and a rated peak capacity of 24.4 MGD. DEQ standards also limit the peak solids loading rate to 35 lb./ft^2/day for activated sludge processes. The peak solids loading rate would allow for a peak flow of approximately 20.9 MGD at a mixed liquor suspended solids concentration of 2,800 mg/l and a recycle rate of 75%. This also assumes that both types of clarifiers are considered equal. However, the capacity of the rectangular clarifiers to handle solids is somewhat limited due to the shallower depth and the limitation of the siphon sludge withdrawal mechanisms. Considering these limitations, the clarifier system will allow a peak (process) flow of approximately 16.0 MGD. Given the condition and efficiencies of the existing rectangular clarifiers, it is recommended that the existing rectangular clarifiers be replaced with two new circular clarifiers to match the existing circular clarifier. The following improvements are proposed for the final clarifier system:

- Add weir washers to improve algae removal for all clarifiers.

- Add two new 95-diameter circular clarifiers to match existing. Existing rectangular clarifiers can be used as sludge storage.
- Rehabilitate expansion joints in the rectangular clarifiers and the effluent channel of the clarifiers.

2.1.7 Effluent Filtration

Currently there is no effluent filtration and none is needed for the current discharge permit limits. However, with the increased projected design flow, and depending upon the outcome of the wasteload allocation study that will establish the viability of a second Caney River discharge to facilitate de facto reuse, more stringent effluent limits in terms of tighter BOD and TSS limits could be imposed by DEQ. The following improvements are proposed for the effluent filtration system.

- New effluent filtration system consisting of dual media filters complete with backwash system to handle a peak capacity of 16.4 MGD.

2.1.8 Effluent Disinfection

Currently chlorine and sulfur-dioxide gas are used for effluent disinfection. To mitigate the risk associated with gaseous systems, a new ultra-violet (UV) disinfection system is proposed. The UV system will be an open channel type system which could be installed in the existing chlorine contact basin. The following improvements are proposed:

- Convert existing chlorine contact basin into a new UV disinfection system.
- Repair existing slide gates on the chlorine contact basins, and include electric valve operators on the contact basin influent valves.

2.1.9 Effluent Sampler

The effluent sampler is a critical part of the process monitoring and compliance system. The existing unit experiences winter freezing problems and needs a new permanent enclosure.

2.1.10 Effluent Pumping

Effluent from the chlorine contact basins flows by gravity to the Caney River outfall under normal discharge conditions. However, during high flood events effluent pumping is required. The existing station consists of a wet well and two vertical turbine pumps, each with a capacity of 5,400 gpm. The effluent pumping capacity needs to be increased to provide the projected peak flow of 16.4 MGD. Two new pumps, each rated for 5,700 gpm, will be required to provide a firm capacity of 16.4 MGD.

It is noted that Bartlesville is pursuing a separate project to permit a second Caney River discharge approximately 5 to 7 river miles upstream of the existing Caney River raw water intake. The effluent pumping system should be consolidated and coordinated with the second Caney discharge pumps to maximize the overall benefits and minimize the capital cost.

2.1.11 Cascade Aeration

With the increased flow, the anticipated new discharge permit will likely have more stringent limits for oxygen. To accommodate the requirement, a cascade aerator will be included in this option. This will require the construction of a cascade aerator structure and some modifications to the discharge piping. To fit into the existing plant site, these improvements *will require that DEQ grant a waiver* for less than standard set-back distances. Such a waiver should be granted in this case since the unit is along the property line that is adjacent to the Caney River.

It is noted that this cascade aeration facility should be planned in conjunction with the second Caney River discharge that Bartlesville is pursuing (under a separate project) to consolidate the overall effluent oxygen discharge limits for both discharge locations.

2.1.12 Standby Generator Power Supply

The facility has a 750 kW generator that was installed in 1983. To add backup power for the existing blowers, as required by DEQ Standards, would require a 1000 kW generator or a second source of power from the electric utility provider. A new backup generator is recommended to support the plant process needs in accordance with DEQ requirements.

2.1.13 WAS Thickening

The existing dissolved air floatation (DAF) unit has the capacity to accommodate the waste activated sludge (WAS) produced by the proposed activated sludge system under maximum month conditions. However, there is only one DAF unit and DEQ standards require redundancy. The DAF unit shows signs of aging, and the existing building does not have space for a second unit. Another option is to consider alternative technology such as the rotating drum thickeners in a new building to replace the existing DAF units altogether. For this option, two rotating drum thickeners in a new building are assumed.

2.1.14 Anaerobic Digestion

Based on the current conditions, the existing anaerobic digesters can support an influent flow of 6.4 MGD. Therefore, to meet the projected 2050 flow of 8.21 MGD, additional digester volume will be needed. In addition, the secondary digester walls are leaking which needs to be repaired. The following improvements are proposed for the anaerobic digestion:

- Rehabilitate the mixing and heating in Digesters 1 and 2.
- Add mixing and heating for Digester 3.
- Rehabilitate floating digester covers and gas piping system.
- Rehabilitate sludge system and valves.
- Add a new Digester 4 with a volume of approximately 430,000 gallons to meet maximum loading conditions.

2.1.15 Digested Sludge Thickening

The existing gravity belt thickener building does not have adequate ventilation for year around operation and requires an improved heating and ventilation system to maintain the proper temperature in the unit during the winter months.

2.1.16 Sludge Dewatering Facility

Currently the plant produces its solid residuals in the form of liquid sludge. The liquid is stored on site and hauled to the land application sites for disposal. Storing, hauling, and disposing of liquid sludge is relatively expensive as compared to dewatered sludge, and liquid sludge does not allow for other reuse options such as composting.. a new sludge dewatering facility will be beneficial to reduce the cost of disposal. However, this will require modification of current sludge disposal practices and the constraints attached to the sludge application lands. Based on input from city staff, it was decided to not include sludge dewatering facility at this time. At some point in the future, it may be worthwhile to consider improving this system to generate a Class A type cake which allows us some flexibility in disposal. Therefore, new sludge dewatering facility is not included in this option.

2.1.17 Administration and Laboratory Building

The existing administration and laboratory building was constructed in 1983 and apparently was constructed without regard to poor subsurface conditions. The building has settled significantly and has numerous cracks and deflections. Repair has been considered and determined not to be feasible. Demolition of the existing building and the construction of a new building are included in this alternative. The new proposed building will have a footprint that is 150 ft^2 larger than the existing building to accommodate a larger laboratory space. Special foundation support in the form of piling will also be included for the new building since it will be constructed in the same vicinity as the existing building. New parking will also be provided.

2.1.18 Additional Land

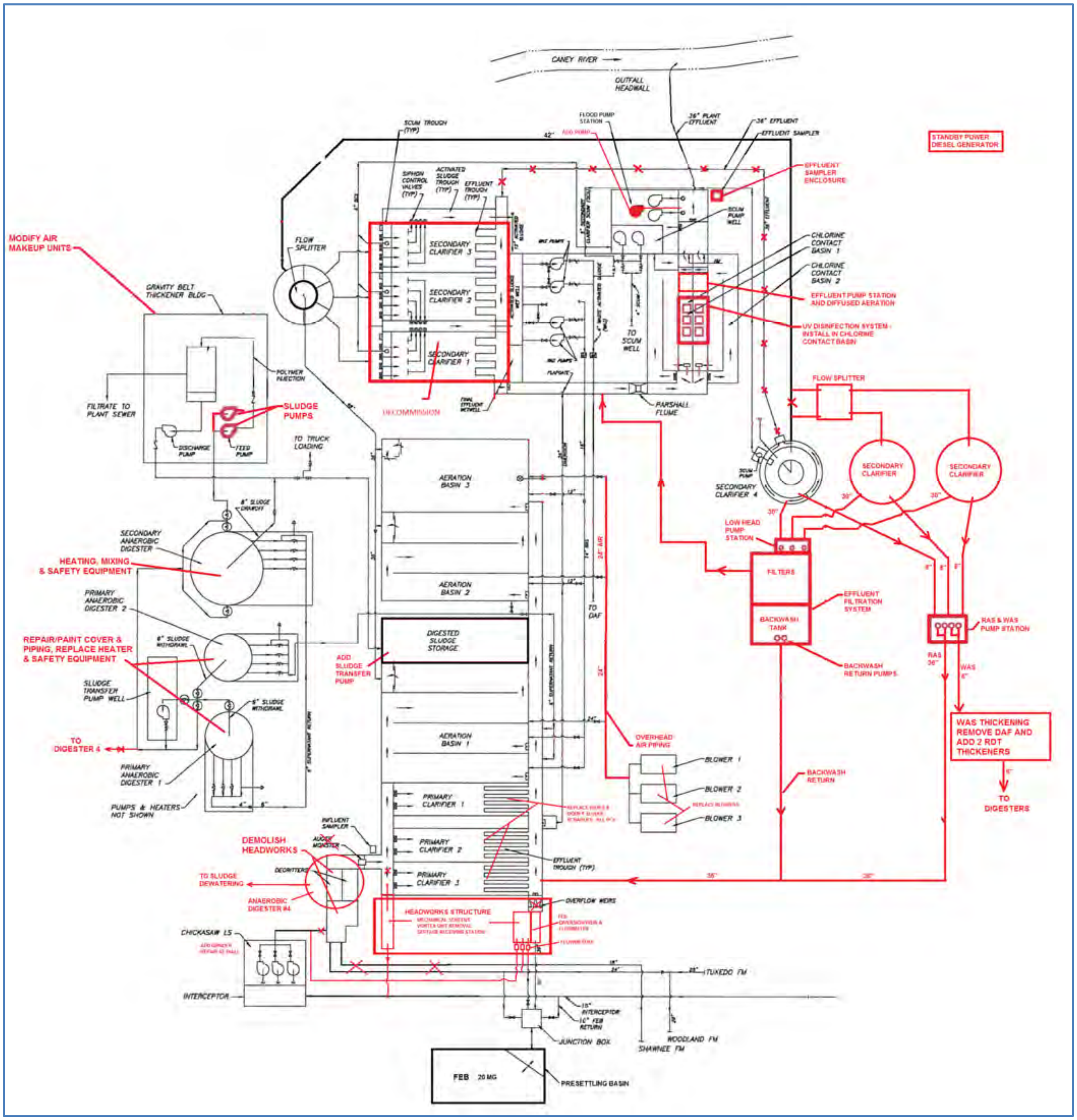
To accommodate the proposed new process units, additional land adjacent to the existing plant site will be needed. Approximately of 5 acres new land is adequate; however, since the existing site is in the floodway and floodplain limits, 5 acres of additional land is recommended to accommodate any flood study mitigation requirements.

Current DEQ regulations require protection of treatment works structures, electrical and mechanical equipment from damage during a 100-year flood. Access to the treatment plant must remain operational and be accessible during a 25-year flood. These requirements will be applicable to existing facilities undergoing major modifications. As part of the flood study, appropriate mitigation and flood protection measures should be identified. For the cost estimate $750,000 is included for constructing a berm around the plant.

Figure TM2.1-1 shows the process schematic for Option 1. The opinion of probable cost is provided on Table TM2.1-2

TABLE TM2.1-2 OPTION 1 PROBABLE COST			
ITEM	DESCRIPTION		ESTIMATE
1	Chickasaw Lift Station		$191,900
2	Flow Equalization Basin		$146,900
3	Headworks		$3,786,100
4	Primary Clarifiers		$985,800
5	Aeration Basins & Blowers		$3,014,700
6	Final Clarifiers		$2,808,500
7	Effluent Filtration		$2,204,500
8	Effluent Disinfection (UV) and Sampling		$1,070,100
9	Effluent Pumping		$197,600
10	Standby Power		$463,000
11	WAS Thickening		$648,500
12	Anaerobic Digestion		$2,730,300
13	Digested Sludge Thickening		$100,000
14	Administration and Laboratory Building		$1,245,700
15	Other (Mobilization, sitework, SCADA, site electrical)		$1,664,700
	SUBTOTAL		$21,258,300
	CONTRACTOR OH&P	16%	$3,401,400
	SUBTOTAL CONSTRUCTION		$24,659,700
	CONTINGENCY	20%	$4,932,000
	TOTAL ESTIMATED CONSTRUCTION COST		**$29,591,700**
	OTHER COSTS		
	PREDESIGN (Floodplain Analysis & Approval)	0.50%	$148,000
	DESIGN	7%	$2,071,500
	GEOTECHNICAL INVESTIGATION	0.15%	$44,400
	BIDDING	0.20%	$59,200
	CONSTRUCTION ADMINISTRATION	2%	$591,900
	RESIDENT INSPECTION		$468,300
	LAND		$250,000
	FLOOD PROTECTION BERM AROUND PLANT		$750,000
	PERMITS		$6,500
	TOTAL ESTIMATED PROJECT COST		**$33,981,500**

Figure TM2.1-1 OPTION 1 PROCESS SCHEMATIC – EXPAND EXISTING



2.2 OPTION 2 – EXTENDED AERATION AND AEROBIC DIGESTION

Option 2 uses extended aeration whereby the need for the primary clarification is eliminated. This option will also require the conversion of the existing anaerobic digesters to aerobic digesters since anaerobic digesters are conducive to process without primary sludge loading. For extended aeration mode of operation, a larger aeration basin is needed.

This option maintains as many of the existing treatment process as possible. It includes additional improvements that are required due to the system aging and by DEQ regulations that were implemented in 2015. Where applicable, the cost of improvements from the 2010 report was used and adjusted by ENR Construction Cost index.

Following is a summary of recommended and necessary improvements.

2.2.1 Chickasaw Lift Station

The improvements are the same as Option1 and repeated here.

The lift station pump capacity is adequate for this option. However, a second grinder unit is needed for redundancy and to protect the pumps. The southeast wall needs repairs to prevent groundwater seepage from migrating into the station. The following improvements are proposed:

- Add a second influent grinder for redundancy.
- Repair the southeast wall to mitigate groundwater seepage.
- Adjust/relocate air duct to above maximum water level.
- The valve separating the wet wells is inoperable and needs to be replaced.

2.2.2 Flow Equalization Basin (FEB)

The improvements are the same as Option 1and repeated here.

The Chickasaw FEB volume is adequate for this option. The synthetic liner was recently replaced by the plant operator. Such liners have a typical service life of 30 years which should last through the 30-year planning period. Currently the flow diversion to the FEB is manually achieved, and there is no flow measurement for process control. A new flow diversion structure with flow measurement and automated flow diversion control is recommended. The following improvements are proposed:

- New flow diversion and control structure with automatic diversion gate.
- Add flow measurement to measure flow into and out of FEB.

2.2.3 Headwork and Degritters

The improvements are the same as Option 1 and repeated here.

Currently there is a single augur grinder unit exposed to the weather. DEQ standards require redundancy. There is not enough space adjacent to the existing unit to accommodate a second unit. A new headwork structure is proposed adjacent to and upstream of the existing headwork facility. The structure will collect all of the influent flows (from three forcemains) and will have two parallel channels to accommodate redundant screens. One will be the existing augur screen relocated to the building, and the second will be the new screen. The structure will also have a flow measurement device.

The existing degritters were last rehabilitated in 1993. The degritters meet DEQ requirements for redundancy, but the screening device, consisting of a single auger monster, does not. There is no room in the existing structure for a second screen or the flow measurement devices. The headwork gate valves upstream of the grit units leak. The existing grit unit is much less efficient than newer technologies such as a vortex type grit removal system. A new vortex type grit system is recommended.

The existing septic truck receiving facility also needs rehabilitation to provide a holding volume for testing and verification and to screen the septic dump prior to discharge to the plant process. The following improvements are proposed:

- New headwork structure with dual screening facility.
- New vortex type dual grit removal facility.
- Add a second influent grinder for redundancy.
- Repair the southeast wall to mitigate groundwater seepage.
- Adjust/relocate air duct to above maximum water level.
- New septic truck receiving station.

2.2.4 Primary Clarifiers

Primary clarifiers are not needed under this option. The existing primary clarifier will be decommissioned.

2.2.5 Aeration Basin

The aeration basin volume must be expanded to meet DEQ's criteria for extended aeration process. The existing Basin 1 is a three-pass plug flow basin with a volume of 0.920 MG. Basin 2 is a three-pass plug flow basin with a process volume of 0.960 MG. Basin 3 is a complete-mix basin with a volume of 0.927 MG. Each basin has a side water depth of 13.5 ft.

For extended aeration mode, DEQ design standards require that the basin volume provide a minimum hydraulic retention time (HRT) of 18-24 hours and a BOD loading no more than 15 lbs BOD_5 per 1000 cft. The three existing basins together provide a process volume of 2.807 MG. This volume will provide approximately 8.2 hours HRT at the 2050 design flow of 8.21 MGD. On the basis of BOD loading, assuming DEQ loading criteria of 15 BOD_5 per 1000 cft, the existing basin volume will support BOD_5 of 5,629 lbs/day. The projected 2050 average and maximum month influent BOD_5 loading are 13,600 lbs/day and 17,600 lbs/day, respectively. Therefore, additional aeration volume will be needed as follows:

Aeration volume needed for BOD loading criteria	= (17600/15)*1000*7.48/1000000 = 8.78 MG
Aeration volume needed for 18-Hour HRT	= 11.65 * (18/24) = 8.73 MG

Together the three existing basins have a total volume of 2.807 MG; therefore, approximately 5.97 MG of additional aeration basin volume is needed. As discussed later, the existing rectangular final clarifiers are proposed to be replaced with new circular clarifiers. The three existing rectangular clarifiers have a total volume of approximately 1.56 MG that could be used for the aeration basin volume. The existing rectangular clarifiers have a side water depth of 12 feet, and the wall may need to be raised by approximately two feet to match the side water depth of the existing aeration basins. This still leaves approximately 4.41 MG in additional aeration volume, which will be provided with a new aeration Basin 5.

Aeration Basin 1 (existing)	= 0.920 MG
Aeration basin 2 (existing)	= 0.960 MG
Aeration basin 3 (existing)	= 0.927 MG

Aeration basin 4 (new)	= 1.560 MG (existing rectangular clarifier modified)
Aeration Basin 5 (new)	= 4.41 MG
Total	= 8.78 MG

Process technology such as Integrated Fixed Film Activated Sludge (IFAS) process was evaluated as an option to handle the increased loading conditions. IFAS integrates the activated sludge process with the attached growth process by incorporating floating media. IFAS can significantly increase the biological process capability of existing aeration basins, and to minimize or eliminate the need for additional basin volume. A conceptual analysis indicated that IFAS can be successfully incorporated within the existing aeration basin footprint to handle the projected BOD loadings. However, in order to meet the DEQ criteria for aerated hydraulic retention time, additional aeration volume will still be needed. Therefore, for this scenario, IFAS technology was not further discussed. However, it is recommended that as part of the pre-design efforts, further review and discussion with DEQ should be initiated for variance in which case the IFAS could be a beneficial option with potential for cost savings.

The three existing 250-hp blowers are at the end of their useful life. In 2001 the original coarse bubble aeration equipment in all three aeration basins was replaced with fine-bubble diffusers. This retrofit was performed to increase the transfer efficiency of the aeration system to reduce power costs. The aeration system was designed to meet an Actual Oxygen Requirement (AOR) of 16,327 lb. O_2/day. However, to meet the projected peak loading, an AOR of approximately 44,000 lb. O_2/day will be needed. Therefore, new blowers will be needed.

The underground air piping at the plant is corroded and leaks significantly. The buried piping should be replaced with new above-ground piping.

The firm capacity of the RAS pump station (one pump operating) is 5.4 MGD. Using the DEQ standard for standard rate (required RAS rate of 150 percent of average flow or 6.2 MGD), the RAS pumping does not have adequate capacity. Therefore, additional RAS pumping capacity will be required.

WAS is pumped to the WAS dissolved air floatation thickener by two vertical, non-clog pumps each with a capacity of 160 gpm. These pumps were installed in 1983. One pump can support influent flows up to 0.92 MGD per DEQ design criteria requiring pumping of 25% of average daily flow Therefore, the WAS pumping will be expanded to provide approximately 2 MGD firm capacity.

The following improvements are proposed for the aeration basins and the blower system:

- Modify the primary clarifier effluent channel and the inlets to the aeration basins to improve the flow split between the basins.
- Replace existing three 250-hp blowers and add two additional blowers to meet the peak oxygen requirements. A total of five 250-hp blowers will be needed. The existing blower building will need to be expanded to accommodate the new blowers.
- The existing basin volume is adequate. Add additional diffusers to the existing basins to meet the increased oxygen demand.
- Replace underground air piping between the blowers and the aeration basins with above-ground air piping.
- Add flow measurement on the basin effluent line to monitor each basin flow.
- Add additional RAS and WAS pumping capacity.

2.2.6 Final Clarifiers

DEQ standards limit the surface overflow rate (SOR) for final clarifiers following extended aeration processes to 400 gal/ft^2/day at design average flows and 1000 gal/ft^2/day at peak hourly flows. At these SORs, the secondary clarifiers have a rated average design capacity of 9.7 MGD and a rated peak capacity of 24.4 MGD. DEQ standards also limit the peak solids loading rate to 35 lb./ft^2/day for activated sludge processes. The peak solids loading rate would allow for a peak flow of approximately 20.9 MGD at a mixed liquor suspended solids concentration of 2,800 mg/l and a recycle rate of 75%. This also assumes that both types of clarifiers are considered equal. However, the capacity of the rectangular clarifiers to handle solids is somewhat limited due to the shallower depth and the limitation of the siphon sludge withdrawal mechanisms. Considering these limitations, the clarifier system will allow a peak (process) flow of approximately 16.0 MGD. Given the condition and efficiencies of the existing rectangular clarifiers, it is recommended that the existing rectangular clarifiers be replaced with two new circular clarifiers to match the existing circular clarifier. The following improvements are proposed for the final clarifier system:

- Add weir washers to improve algae removal for all clarifiers.
- Add two new 95-diameter circular clarifiers to match existing. Existing rectangular clarifiers can be modified to provide aeration basins.
- Rehabilitate expansion joints in the rectangular clarifiers and the effluent channel of the clarifiers

2.2.7 Effluent Filtration

The improvements are the same as Option 1 and repeated here.

Currently there is no effluent filtration and none is needed for the current discharge permit limits. However, with the increased projected design flow, and depending upon the outcome of the wasteload allocation study that will establish the viability of a second Caney River discharge to facilitate de facto reuse, more stringent effluent limits in terms of tighter BOD and TSS limits could be imposed by DEQ. The following improvements are proposed for the effluent filtration system.

- New effluent filtration system consisting of dual media filters complete with backwash system to handle a peak capacity of 16.4 MGD.

2.2.8 Effluent Disinfection

The improvements are the same as Option 1 and repeated here.

Currently chlorine and sulfur dioxide gas are used for effluent disinfection. To mitigate the risk associated with gaseous systems, a new ultra-violet (UV) disinfection system is proposed. The UV system will be an open channel type system which could be installed in the existing chlorine contact basin. The following improvements are proposed:

- Convert existing chlorine contact basin into a new UV disinfection system.
- Repair existing slide gates on the chlorine contact basins.

2.2.9 Effluent Sampler

The improvements are the same as Option 1 and repeated here.

The effluent sampler is a critical part of the process monitoring and compliance system. The existing unit experiences winter freezing problems and needs a new permanent enclosure.

2.2.10 Effluent Pumping

The improvements are the same as Option 1 and repeated here.

Effluent from the chlorine contact basins flows by gravity to the Caney River outfall under normal discharge conditions. However, during high flood events effluent pumping is required. The existing station consists of a wet well and two vertical turbine pumps, each with a capacity of 5,400 gpm. The effluent pumping capacity needs to be increased to provide the projected peak flow of 16.4 MGD. Two new pumps each rated for 5,700 gpm will be required to provide a firm capacity of 16.4 MGD.

It is noted that Bartlesville is pursuing a separate project to permit a second Caney River discharge approximately 5 to 7 river miles upstream of the existing Caney River raw water intake. The effluent pumping system should be consolidated and coordinated with the second Caney discharge pumps to maximize the overall benefits and minimize the capital cost.

2.2.11 Cascade Aeration

The improvements are the same as Option 1 and repeated here.

With the increased flow, the anticipated new discharge permit will likely have more stringent limits for oxygen. To accommodate the requirement, a cascade aerator will be included in this option. This will require the construction of a cascade aerator structure and some modifications to the discharge piping. To fit on the existing plant site, these improvements *will require that DEQ grant a waiver* for less than standard set-back distances. Such a waiver should be granted in this case since the unit is along the property line that is adjacent to the Caney River.

It is noted that this cascade aeration facility should be planned in conjunction with the second Caney River discharge that Bartlesville is pursuing (under a separate project) to consolidate the overall effluent oxygen discharge limits for both discharge locations.

2.2.12 Standby Generator Power Supply

The improvements are the same as Option 1 and repeated here.

The facility has a 750 kW generator that was installed in 1983. To add backup power for the existing blowers, as required by DEQ Standards, would require a 1000 kW generator or a second source of power from the electric utility provider. A new backup generator is recommended to support the plant process needs in accordance with DEQ requirements.

2.2.13 WAS Thickening

With the modification of the digestion process from aerobic to anaerobic, there is no separate primary and secondary waste sludge. The need for thickening is not necessary. However, to reduce the overall volume requirement for the aerobic digester, waste sludge thickening is assumed in this option. The existing dissolved air floatation (DAF) unit has the capacity to accommodate the waste activated sludge (WAS) produced by the proposed activated sludge system under maximum month conditions. However, there is only one DAF unit and DEQ standards require redundancy. The DAF unit shows signs of aging, and the existing building does not have space for a second unit. Another option is to consider alternative technology such as the rotating drum thickeners in a new building to replace the existing DAF units altogether. For this option, two rotating drum thickeners in a new building are assumed.

2.2.14 Aerobic Digestion

The existing anaerobic digesters will be converted into open-top aerobic digesters. The following improvements are proposed for the anaerobic digestion:

- Convert the three existing anaerobic digesters to aerobic digesters.
- Construct new aerobic digesters with a total volume of 1.15 MG.
- Provide new aerobic digester blower building with four 150-hp blowers.

2.2.15 Digested Sludge Thickening

The existing gravity belt thickener building requires an improved heating and ventilation system to maintain the proper temperature in the unit during the winter months.

2.2.16 Sludge Dewatering Facility

Currently the plant produces its solid residuals in the form of liquid sludge. The liquid is stored on site and hauled to the land application sites for disposal. Storing, hauling, and disposing of liquid sludge is relatively expensive as compared to dewatered sludge, and liquid sludge does not allow for other reuse options such as composting.. a new sludge dewatering facility will be beneficial to reduce the cost of disposal. However, this will require modification of current sludge disposal practices and the constraints attached to the sludge application lands. Based on input from city staff, it was decided to not include sludge dewatering facility at this time. At some point in the future, it may be worthwhile to consider improving this system to generate a Class A type cake which allows us some flexibility in disposal. Therefore, new sludge dewatering facility is not included in this option.

2.2.17 Administration and Laboratory Building

The existing administration and laboratory building was constructed in 1983 and apparently was constructed without regard to poor subsurface conditions. The building has settled significantly and has numerous cracks and deflections. Repair has been considered and determined not to be feasible. Demolition of the existing building and the construction of a new building are included in this alternative. The new proposed building will have a footprint that is 150 ft^2 larger than the existing building to accommodate a larger laboratory space. Special foundation support in the form of piling will also be included for the new building since it will be constructed in the same vicinity as the existing building. New parking will also be provided.

2.2.18 Additional Land

To accommodate the proposed new process units, additional land adjacent to the existing land site will be needed. A minimum of 5 acres is necessary; however, since the existing site is in the floodway and floodplain limits, 2 to 5 acres of additional land is recommended to accommodate any flood study mitigation requirements.

Current DEQ regulations require protection of treatment works structures, electrical and mechanical equipment from damage during a 100-year flood. Access to the treatment plant must remain operational and be accessible during a 25-year flood. These requirements will be applicable to existing facilities undergoing major modifications. As part of the flood study, appropriate mitigation and flood protection measures should be identified. For the cost estimate $750,000 is included for constructing a berm around the plant. Figure TM2.1-2.2 shows the process schematic for Option 2. The opinion of probable cost is provided on Table TM2.1-3.

Figure TM2.1-2 OPTION 2 PROCESS SCHEMATIC - EXTENDED AERATION AND AEROBIC DIGESTION

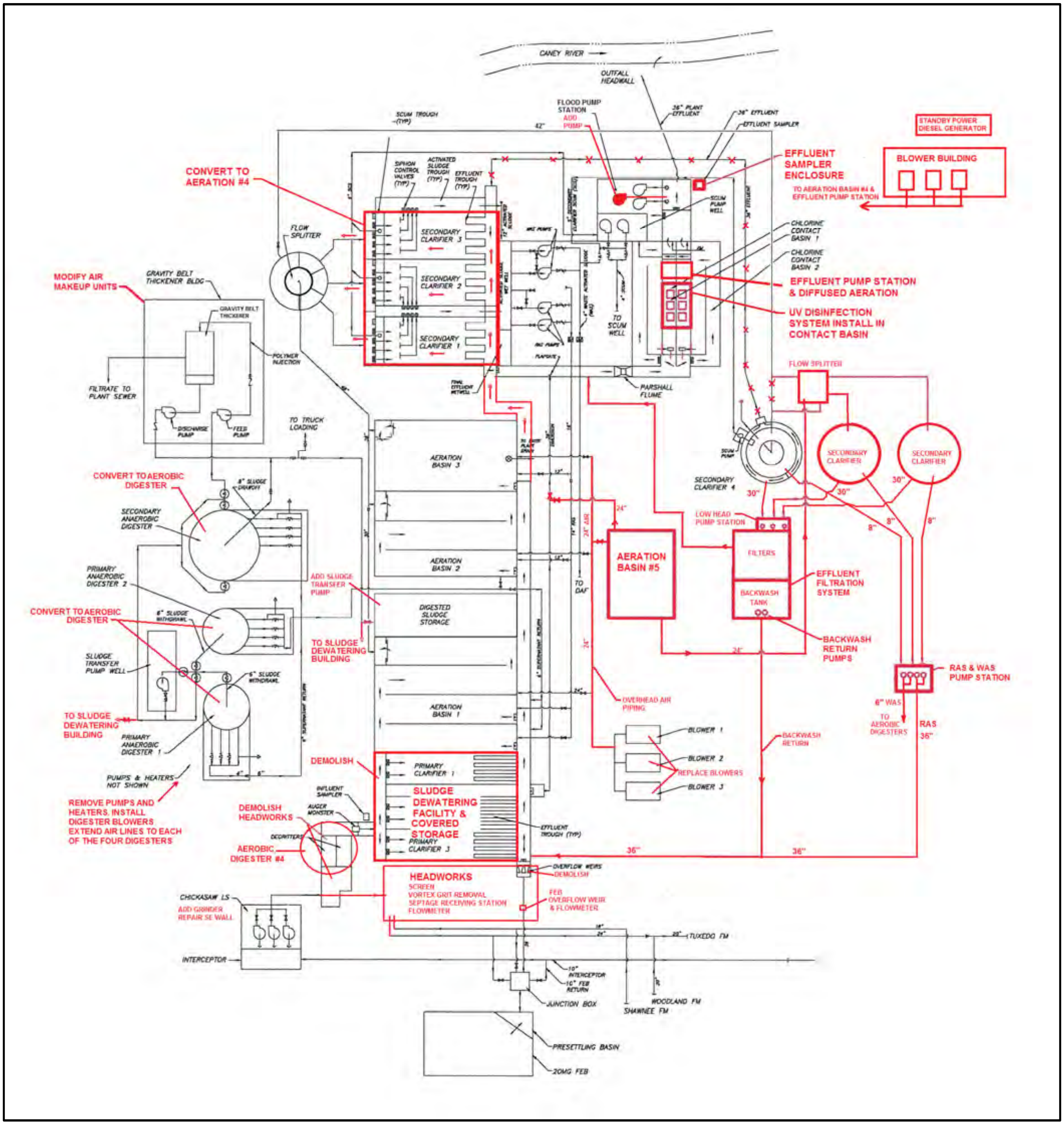


TABLE TM2.1-3 OPTION 2 PROBABLE COST			
ITEM	**DESCRIPTION**		**ESTIMATE**
1	Chickasaw Lift Station		$173,800
2	Flow Equalization Basin		$146,800
3	Headworks		$3,724,900
4	Primary Clarifiers		$50,000
5	Aeration Basins		$7,874,000
6	Final Clarifiers		$2,793,500
7	Effluent Filtration		$2,203,800
8	Effluent Disinfection and Sampling		$1,010,300
9	Effluent Pumping		$156,500
10	Effluent Aeration		$41,000
11	Standby Power		$630,800
12	WAS Thickening		$504,300
13	Aerobic Digestion		$3,323,000
14	Digested Sludge Thickening		$30,000
15	Administration and Laboratory Building		$1,260,300
16	Other (Mobilization, sitework, SCADA, site electrical)		$1,664,100
	SUBTOTAL		$25,587,100
	CONTRACTOR OH&P	16%	$4,094,000
	SUBTOTAL CONSTRUCTION		$29,681,100
	CONTINGENCY	20%	$5,936,300
	TOTAL ESTIMATED CONSTRUCTION COST		**$35,617,400**
	OTHER COSTS		
	PREDESIGN (Floodplain Analysis & Approval)	0.50%	$178,100
	DESIGN	7%	$2,493,300
	GEOTECHNICAL INVESTIGATION	0.15%	$53,500
	BIDDING	0.20%	$71,300
	CONSTRUCTION ADMINISTRATION	2%	$712,400
	RESIDENT INSPECTION		$468,300
	LAND		$250,000
	FLOOD PROTECTION BERM AROUND PLANT		$750,000
	PERMITS		$6,500
	TOTAL ESTIMATED PROJECT COST		**$40,600,800**

Amendment to

WWTP and Reuse Feasibility Study

Technical Memorandum No. 2.2

Alternative 2 Evaluation - Add Second Treatment Facility South of the City

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. 06/19

Tetra Tech Project No. 200-11458-16002

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REGISTERED PROFESSIONAL ENGINEER
SRINIVASAN
SUNDARAMOORTHY
16260
9.5.17
OKLAHOMA

ACRONYMS/ABBREVIATIONS

Acronyms/Abbreviations	Definition
AD	Anaerobic Digesters
AOR	Actual Oxygen Reduction
BOD	Biochemical Oxygen Demand
BOD	Biological Oxygen Demand
CCWWTP	Chickasaw Wastewater Treatment Plant
DAF	Dissolved Air Flotation
DEQ	Department of Environmental Quality
DS	Dry Solids
FC	Final Clarifier
FEB	Flow Equalization Basin
IFAS	Integrated Fixed Film Activated Sludge
GPCD	Gallons per Capita per Day
MGD	Million Gallons per Day
MBBR	Moving Bed Biofilm Reactor
MLSS	Mixed Liquor Suspended Solids
NH3-N	Ammonia Nitrogen
RAS	Return Activated Sludge
SCADA	Supervisory Control and Data Acquisition
SOR	Surface Overflow Rate
TBD	To be Determined
TM	Technical Memorandum
TSS	Total Suspended Solids
VFD	Variable Frequency Drive
WAS	Waste Activated Sludge
WW	Wastewater

1.0 INTRODUCTION

Technical Memorandum No.2.2 (TM-2.2) summarizes Alternative 2, which maintains the existing CCWWTP at reduced treatment capacity and constructs a new second treatment facility south of the city to handle the rest of the flow. Alternative 2 is updated to reflect current conditions, current construction costs, and the revised flow and plant loadings developed for the 2050 planning year (30-year planning period).

The 2010 Facility Plan assumed that up to 3.97 MGD will be treated at the CWWTP and the balance of 4.32 MGD will be treated at the new south facility. To distribute the flows, the previous amendment assumed that the CWWTP will handle only those flows from the Chickasaw, Tuxedo, and Woodland lift stations (basins), and the new treatment facility will handle the flows generated in the Shawnee and Rice Creek basins.

For this facility plan amendment, the concept of de-facto reuse has been incorporated in distributing the flows between the existing CWWTP and new facility. The following updates are noted:

- In discussion with City staff, it is assumed that up to 4 MGD of de-facto reuse could be utilized during the planning period. Therefore, for this alternative, a minimum 4 MGD flow diversion to the CWWTP is necessary. Projected 2050 average annual daily flows from the Chickasaw, Tuxedo, and Woodland basins contribute a total of 5.23 MGD that will be directed to the existing CWWTP. This will leave 2.98 MGD average annual daily flow that will be diverted to the new south treatment plant. Refer to Table TM3.7 included in TM3 for details regarding the 2050 projected total average and peak flows for the Bartlesville collection system basins.

- The population growth and projected flow from the Chickasaw, Tuxedo, and Woodland basins are updated based on current information and city staff input.

The process design loadings used to develop these options are provided in Table TM2.2-1.

Table TM2.2-1 SUMMARY OF DESIGN CRITERIA

Parameter	Peaking Factor	Planning Year 2050 CWWTP			Planning Year 2050 South Plant		
		Flow (MGD)	Conc. (mg/L)	Mass (lbs/day)	Flow (MGD)	Conc. (mg/L)	Mass (lbs/day)
Process Flow							
Average Annual Daily	--	5.23	--	--	2.98	--	--
Max Mo. Average Daily	1.37	7.165	--	--	4.416	--	--
Max. Day[1]	2	10.46	--	--	5.86	--	--
Influent BOD							
Average	--	--	199	8,763	--	199	4,837
Avg. Day of Max. Mo.	1.30	--	--	11,392	--	--	6,208
Max. Day.	3.03	--	--	26,552	--	--	14,248
Influent TSS							
Average	--	--	330	14,223	--	323	7,877
Avg. Day of Max. Mo.	1.41	--	337	18,490	--	320	12,610
Max. Day.	3.10	--	257	42,097	--	293	25,403
Influent NH_3-N							
Average	--	--	18	793	--	18	427
Avg. Day of Max. Mo.	1.46	--	--	1,157	--	--	623
Max. Day.	6.14	--	--	4,867	--	--	2,623
Influent Alkalinity, Min.[2]	--	--	--	228	--	228	--
Observed Yield - Raw Sludge:		0.80	lbs DS/lbs BOD				

[1]Assumes Flow above the 2050 Max. Day will diverted to an FEB.
[2]50th Percentile Influent alkalinity value from 2012-2017 plant data.

2.0 TREATMENT FACILITIES IMPROVEMENTS

2.1 CHICKASAW WWTP IMPROVEMENTS

This option is an update of the 2010 evaluation. It maintains as many of the existing treatment processes as possible. It includes additional improvements that are required due to the system aging and by DEQ regulations. Where applicable, the costs of improvements from the 2010 report were used and adjusted by the ENR Construction Cost index.

Following is a summary of recommended and necessary improvements.

2.1.1 Chickasaw Lift Station

The lift station pump capacity is adequate for this option. However, a second grinder unit is needed for redundancy and to protect the pumps. The southeast wall needs repair to prevent groundwater seepage from migrating into the station. The following improvements are proposed:

- Add a second influent grinder for redundancy.
- Repair the southeast wall to mitigate groundwater seepage.
- Adjust/relocate air duct to above maximum water level.
- The valve separating the wet wells is inoperable and needs to be replaced.

2.1.2 Flow Equalization Basin (FEB)

The Chickasaw FEB volume is adequate for this option. The synthetic liner was recently replaced by the plant operator. Such liners have a typical service life of 30 years which should last through the 30-year planning period. Currently the flow diversion to the FEB is manually achieved, and there is no flow measurement for process control. A new flow diversion structure with flow measurement and automated flow diversion control is recommended. The following improvements are proposed:

- New flow diversion and control structure with automatic diversion gate.
- Add flow measurement to measure flow into and out of the FEB.

2.1.3 Headwork and Degritters

Currently there is a single augur grinder unit exposed to the weather. DEQ standards require redundancy. There is not enough space adjacent to the existing unit to accommodate a second unit. A new headwork structure is proposed adjacent to and upstream of the existing headwork facility. The structure will collect all of the influent flows (from three force mains) and will have two parallel channels to accommodate redundant screens. One will be the existing augur screen relocated to the building, and the second will be the new screen. The structure will also have a flow measurement device.

The existing degritters were last rehabilitated in 1993. The degritters meet the DEQ requirements for redundancy, but the screening device, consisting of a single auger monster, does not. There is no room in the existing structure for a second screen or the flow measurement devices. The headwork gate valves upstream of the grit units leak. The existing grit unit is much less efficient than newer technologies such as a vortex-type grit removal system. A new vortex-type grit system is recommended.

The existing septic truck receiving facility also needs rehabilitation to provide a holding volume for testing and verification and to screen the septic dump prior to discharge to the plant process. The following improvements are proposed:

- New headwork structure with dual screening facility.
- New vortex-type dual grit removal facility.
- Add a second influent grinder for redundancy.
- Repair the southeast wall to mitigate groundwater seepage.
- Adjust/relocate air duct to above maximum water level.
- New septic truck receiving station.

2.1.4 Primary Clarifiers

Primary Clarifier 1 was constructed in 1934 and upgraded in 1983. Primary Clarifiers 2 and 3 were constructed in 1983. The concrete structure shows signs of aging but generally appears in satisfactory condition. However, the sludge removal mechanism, the overflow weir, and baffles need rehabilitation.

DEQ standards limit the hydraulic overflow rate to 1,000 gal/ft^2/day at design average flows and 1,500 gal/ft^2/day for peak hourly flows. At these overflow rates, the primary clarification system has a rated average design capacity of 7.8 MGD and a peak capacity of 11.7 MGD. The existing primary clarifier is adequate for this option; however, the following improvements are proposed to address certain existing deficiencies:

- Rehabilitate/replace sludge removal mechanisms on existing primary clarifiers.
- Replace weirs and baffles on the existing primary clarifiers.
- Investigate and correct hydraulic bottleneck within the primary clarifier bottom sludge draw-off sump.

2.1.5 Aeration Basin

Basin 1 is a three-pass plug flow basin with a volume of 0.920 MG. Basin 2 is a three-pass plug flow basin with a process volume of 0.960 MG. Basin 3 is a complete-mix basin with a volume of 0.927 MG. Each basin has a side water depth of 13.5 ft. Basin 1 was constructed in 1934 and modified in 1983. Basin 2 was constructed in 1983, and Basin 3 was constructed in 1993 to boost the nitrification capability of the process to a maximum month flow of 7.0 MGD. Proper flow splitting to the aeration basins continues to be a problem and needs to be remedied in the plan.

The existing CWWTP is a conventional suspended growth activated sludge process. Current DEQ design standards require that the basin volume provide a minimum hydraulic retention time (HRT) of 6-8 hours and a BOD loading of 30-40 lbs BOD_5 per 1000 cft. Three existing basins together provide a process volume of 2.807 MG. This volume will provide approximately 12.8 hours HRT at the 2050 design flow of 5.23 MGD. On the basis of BOD loading, assuming DEQ loading criteria of 35 BOD_5 per 1000 cft, the existing basin volume will support BOD_5 of 13,134 lbs/day. The projected 2050 average and maximum month influent BOD_5 loading are 8,763 lbs/day and 11,392 lbs/day, respectively. Assuming 35% BOD reduction in the primary clarifier, the projected BOD_5 loading to the aeration basins are 5,696 lbs/day and 7,405 lbs/day. Therefore, the existing aeration basin volume is adequate for the 2050 projected loading.

The three existing 250-hp blowers are at the end of their useful life. In 2001 the original coarse bubble aeration equipment in all three aeration basins was replaced with fine-bubble diffusers. This retrofit was performed to increase the transfer efficiency of the aeration system to reduce power costs. The aeration system was designed to meet an Actual Oxygen Requirement (AOR) of 16,327 lb. O_2/day. However, to meet the projected peak loading, an approximate AOR of 11,000 lb. O_2/day will be needed. Therefore, the existing blower capacity is adequate, but the blowers will be replaced with new.

The underground air piping at the plant is corroded and leaks significantly. The buried piping should be replaced with new above-ground piping.

The firm capacity of the RAS pump station (one pump operating) is 5.4 MGD. Using the DEQ standard for standard rate (required RAS rate of 75 percent of average flow or 5.23 MGD), the RAS pumping has adequate capacity.

WAS is pumped to the WAS dissolved air floatation thickener by two vertical, non-clog pumps each with a capacity of 160 gpm. These pumps were installed in 1983. One pump can support influent flows up to 0.92 MGD per DEQ design criteria requiring pumping of 25% of average daily flow Therefore, the WAS pumping will be expanded to provide approximately 2 MGD firm capacity.

The following improvements are proposed for the aeration basins and the blower system:

- Modify the primary clarifier effluent channel and the inlets to the aeration basins to improve the flow split between the basins.
- Replace the existing three 250-hp blowers with new blowers.
- The existing basin volume is adequate.
- Replace underground air piping between the blowers and the aeration basins with above-ground air piping.
- Add flow measurement on the basin effluent line to monitor each basin flow.
- Add additional WAS pumping capacity.
- Improve RAS flow split between basins.

2.1.6 Final Clarifiers

DEQ standards limit the surface overflow rate (SOR) for final clarifiers following conventional activated sludge systems with single-stage nitrification process (which is CWWTP) to 400 gal/ft^2/day at design average flows and 1,000 gal/ft^2/day at peak hourly flows. At these SORs, the secondary clarifiers have a rated average design capacity of 9.7 MGD and a rated peak capacity of 24.4 MGD. DEQ standards also limit the peak solids loading rate to 35 lb./ft^2/day for activated sludge processes. The peak solids loading rate would allow for a peak flow of approximately 20.9 MGD at a mixed liquor suspended solids concentration of 2,800 mg/l and a recycle rate of 75%. This also assumes that both types of clarifiers are considered equal. However, the capacity of the rectangular clarifiers to handle solids is somewhat limited due to the shallower depth and the limitation of the siphon sludge withdrawal mechanisms. Considering these limitations, the clarifier system will allow a peak (process) flow of approximately 16.0 MGD. Given the condition and efficiencies of the existing rectangular clarifiers, it is recommended that the existing rectangular clarifiers be replaced with two new circular clarifiers to match the existing circular clarifier. The following improvements are proposed for the final clarifier system:

- Add weir washers to improve algae removal for all clarifiers.
- Add two new 95-diameter circular clarifiers to match the existing. Existing rectangular clarifiers can be used as sludge storage.

2.1.7 Effluent Filtration

Currently there is no effluent filtration and none is needed for the current discharge permit limits. However, with the increased projected design flow, and depending upon the outcome of the wasteload allocation study that will establish the viability of a second Caney River discharge to facilitate de facto reuse, more stringent effluent limits in terms of tighter BOD and TSS limits could be imposed by DEQ. The following improvements are proposed for the effluent filtration system.

- New effluent filtration system consisting of dual media filters complete with backwash system to handle a peak capacity of 16.4 MGD.

2.1.8 Effluent Disinfection

Currently chlorine and sulfur-dioxide gas are used for effluent disinfection. To mitigate the risk associated with gaseous systems, a new ultra-violet (UV) disinfection system is proposed. The UV system will be an open channel type system which could be installed in the existing chlorine contact basin. The following improvements are proposed:

- Convert existing chlorine contact basin into a new UV disinfection system.
- Repair existing slide gates on the chlorine contact basins.

2.1.9 Effluent Sampler

The effluent sampler is a critical part of the process monitoring and compliance system. The existing unit experiences winter freezing problems and needs a new permanent enclosure.

2.1.10 Effluent Pumping

Effluent from the chlorine contact basins flows by gravity to the Caney River outfall under normal discharge conditions. However, during high flood events, effluent pumping is required. The existing station consists of a wet well and two vertical turbine pumps, each with a capacity of 5,400 gpm. The effluent pumping capacity needs to be increased to provide the projected peak flow of 16.4 MGD. Two new pumps, each rated for 5,700 gpm, will be required to provide a firm capacity of 16.4 MGD.

It is noted that Bartlesville is pursuing a separate project to permit a second Caney River discharge approximately 5 to 7 river miles upstream of the existing Caney River raw water intake. The effluent pumping system should be consolidated and coordinated with the second Caney discharge pumps to maximize the overall benefits and minimize the capital cost.

2.1.11 Cascade Aeration

With the increased flow, the anticipated new discharge permit will likely have more stringent limits for oxygen. To accommodate the requirement, a cascade aerator will be included in this option. This will require the construction of a cascade aerator structure and some modifications to the discharge piping. To fit into the existing plant site, these improvements *will require that DEQ grant a waiver* for less than standard set-back distances. Such a waiver should be granted in this case since the unit is along the property line that is adjacent to the Caney River.

It is noted that this cascade aeration facility should be planned in conjunction with the second Caney River discharge that Bartlesville is pursuing (under a separate project) to consolidate the overall effluent oxygen discharge limits for both discharge locations.

2.1.12 Standby Generator Power Supply

The facility has a 750 kW generator that was installed in 1983. To add backup power for the existing blowers as required by DEQ Standards would require a 1000 kW generator or a second source of power from the electric utility provider. A new backup generator is recommended to support the plant process needs in accordance with DEQ requirements.

2.1.13 WAS Thickening

The existing dissolved air floatation (DAF) unit has the capacity to accommodate the waste activated sludge (WAS) produced by the proposed activated sludge system under maximum month conditions. However, there is only one DAF unit and DEQ standards require redundancy. The DAF unit shows signs of aging, and the existing building does not have space for a second unit. Since the digested sludge gravity belt thickener will no longer be needed for that service (since the sludge dewatering system is being installed - see below), one option is to convert the existing belt thickener to a standby WAS thickener. The gravity belt thickener will perform well in this service with no facility modifications except for the inlet and outlet piping. However, given the age and condition of the existing DAF, a second gravity belt thickener will be needed sooner or later for redundancy purposes. Another option is to consider alternative technology such as the rotating drum thickeners in a new building to replace the existing DAF units altogether.

2.1.14 Anaerobic Digestion

Based on the current conditions, the existing anaerobic digesters can support an influent flow of 6.4 MGD. Therefore, additional digester volume is not needed. The following improvements are proposed for the anaerobic digestion:

- Rehabilitate the mixing and heating in Digesters 1 and 2.
- Add mixing and heating for Digester 3.
- Rehabilitate floating digester covers and gas piping system.
- Rehabilitate sludge system and valves.

2.1.15 Digested Sludge Thickening

The existing gravity belt thickener building does not have adequate ventilation for year around operation and requires an improved heating and ventilation system to maintain the proper temperature in the unit during the winter months.

2.1.16 Sludge Dewatering Facility

Currently the plant produces its solid residuals in the form of liquid sludge. The liquid is stored on site and hauled to the land application sites for disposal. Storing, hauling, and disposing of liquid sludge is relatively expensive as compared to dewatered sludge, and liquid sludge does not allow for other reuse options such as composting.. a new sludge dewatering facility will be beneficial to reduce the cost of disposal. However, this will require modification of current sludge disposal practices and the constraints attached to the sludge application lands. Based on input from city staff, it was decided to not include sludge dewatering facility at this time. At some point in the future, it may be worthwhile to consider improving this system to generate a Class A type cake which allows us some flexibility in disposal. Therefore, new sludge dewatering facility is not included in this option.

2.1.17 Administration and Laboratory Building

The existing administration and laboratory building was constructed in 1983 and apparently was constructed without regard to poor subsurface conditions. The building has settled significantly and has numerous cracks and deflections. Repair has been considered and determined not to be feasible. Demolition of the existing building and the construction of a new building are included in this alternative. The new proposed building will have a footprint that is 150 ft^2 larger than the existing building to accommodate a larger laboratory space. Special foundation support in the form of piling will also be included for the new building since it will be constructed in the same vicinity as the existing building. New parking will also be provided.

2.1.18 Additional Land

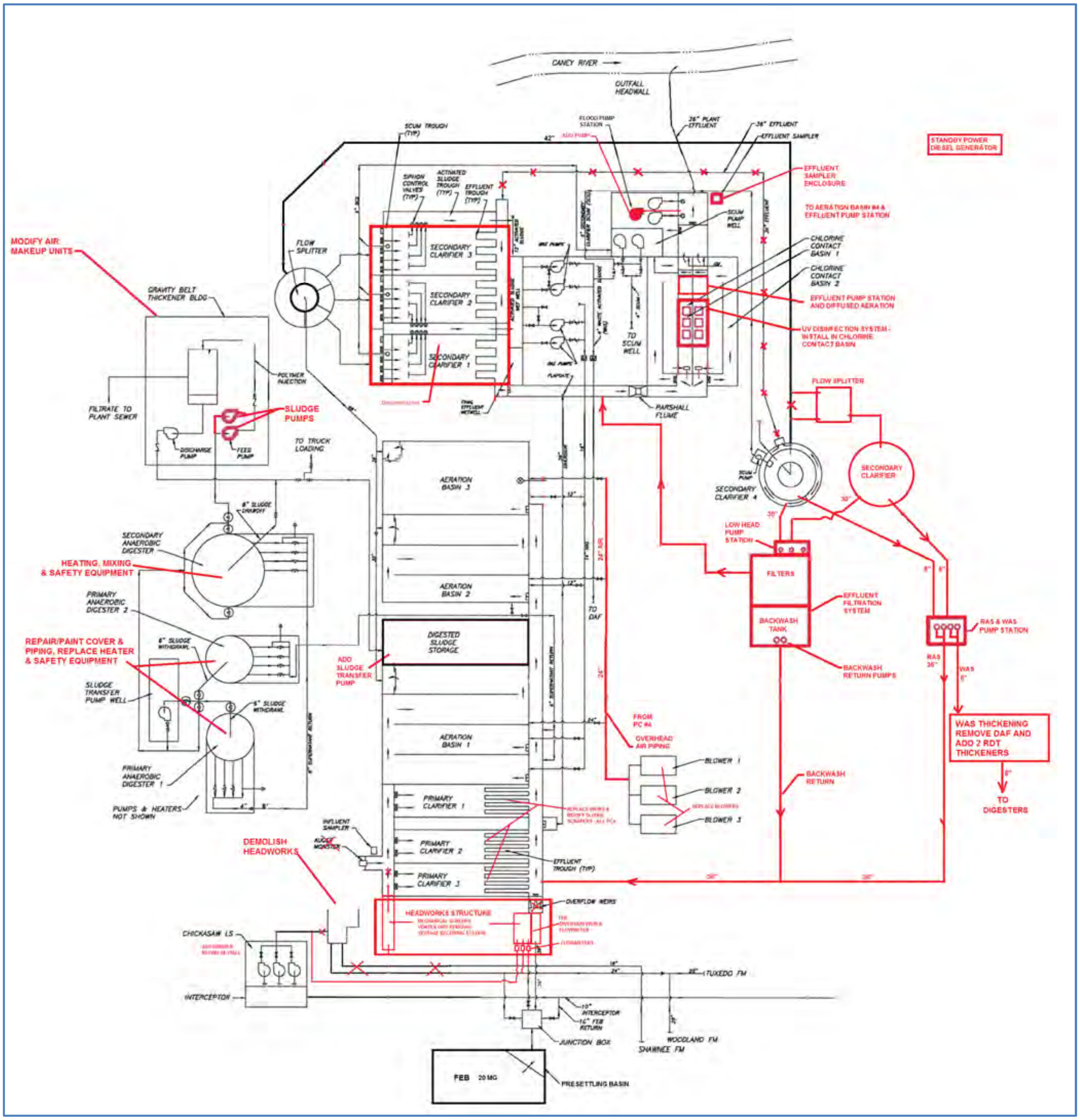
To accommodate the proposed new process units, additional land adjacent to the existing plant site will be needed. Approximately 5 acres of new land is adequate; however, since the existing site is in the floodway and floodplain limits, 2 to 5 acres of additional land is recommended to accommodate any flood study mitigation requirements.

Current DEQ regulations require protection of treatment works structures, electrical and mechanical equipment from damage during a 100-year flood. Access to the treatment plant must remain operational and be accessible during a 25-year flood. These requirements will be applicable to existing facilities undergoing major modifications. As part of the flood study, appropriate mitigation and flood protection measures should be identified. For the cost estimate $750,000 is included for constructing a berm around the plant.

Figure TM2.2-1 shows the process schematic for Option 2. The opinion of probable cost is provided on Table TM2.2-2.

TABLE TM2.2-2 ALTERNATIVE 2 - CWWTP IMPROVEMENTS PROBABLE COST			
ITEM	DESCRIPTION		ESTIMATE
1	Chickasaw Lift Station		$191,900
2	Flow Equalization Basin		$146,900
3	Headworks		$3,643,600
4	Primary Clarifiers		$965,900
5	Aeration Basins		$1,584,300
6	Final Clarifiers		$1,484,200
7	Effluent Filtration		$1,566,500
8	Effluent Disinfection and Sampling		$1,070,200
9	Effluent Pumping		$156,600
10	Effluent Aeration		$41,100
11	Standby Power		$463,100
12	WAS Thickening		$449,500
13	Anaerobic Digestion		$1,198,100
14	Digested Sludge Thickening		$100,000
15	Administration and Laboratory Building		$1,245,900
16	Other (Mobilization, sitework, SCADA, site electrical)		$1,533,500
	SUBTOTAL		$15,841,300
	CONTRACTOR OH&P	16%	$2,534,700
	SUBTOTAL CONSTRUCTION		$18,376,000
	CONTINGENCY	20%	$3,675,200
	TOTAL ESTIMATED CONSTRUCTION COST		**$22,051,200**
	OTHER COSTS		
	PREDESIGN (Floodplain Analysis & Approval)	0.50%	$110,300
	DESIGN	7%	$1,543,600
	GEOTECHNICAL INVESTIGATION	0.15%	$33,100
	BIDDING	0.20%	$44,200
	CONSTRUCTION ADMINISTRATION	2%	$441,100
	RESIDENT INSPECTION		$468,300
	LAND		$250,000
	FLOOD PROTECTION BERM AROUND PLANT		$750,000
	PERMITS		$6,500
	TOTAL ESTIMATED PROJECT COST		**$25,698,300**

Figure TM2.2-1 PROCESS SCHEMATIC – EXPAND EXISTING



2.2 SECOND TREATMENT PLANT

Alternative 2 splits the total wastewater flow between the CWWTP and a new treatment plant to be located south of the city. The concept for the new south plant will be established under a separate report, but the following attributes will be used to establish estimated capital and operation costs.

- The design flows for this plant will be an average of 2.98 MGD, an average of 4.416 MGD during the maximum month, and a peak of 5.86 MGD.
- The biological treatment process will be capable of providing advanced secondary treatment including nitrification. The process will also be selected such that it can initially (or be easily modified in the future) provide biological nutrient removal to address potential limits on total nitrogen and phosphorous which may be imposed in the future.
- The liquid and solids treatment systems will be selected and designed for expansion in a modular fashion to facilitate phasing, if needed, to accommodate funding.
- Separate solids processing facilities are assumed; include consolidated dewatered sludge disposal facilities for both the CWWTP and the new south plant.
- The solids processing system will include a composting system to allow a portion of the dewatered sludge to be composted for reuse. Initially, this unit would be of a demonstration size with area reserved for future expansion.
- The laboratory facility at the plant would be coordinated in conjunction with the CWWTP facility so as not to unnecessarily duplicate facilities and accommodate consolidation of tasks between the two facilities.
- The facility will not be located in the floodplain, and all process units (except for the cascade aerator) will be maintained above the 500-year flood level.
- Approximately 30 acres of land will be acquired for the plant and expected future expansions.

This option maintains as many of the existing treatment process as possible. It includes additional improvements that are required due to the system aging and by DEQ regulations that were implemented in 2015. Where applicable, the costs of improvements from the 2010 report were used and adjusted by the ENR Construction Cost index.

Amendment to

WWTP Facility Plan and Reuse Feasibility Study

Technical Memorandum No. 3

Existing Collection, Transport and Storage Facilities Condition Assessment

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. 06/19

Tetra Tech Project No. 200-11458-16002

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REGISTERED PROFESSIONAL ENGINEER
SRINIVASAN
SUNDARAMOORTHY
16260
9-5-17
OKLAHOMA

ACRONYMS/ABBREVIATIONS

Name	Abbreviation
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
FEB	Flow Equalization Basin
FOA	Funding Opportunity Announcement
Fps	Feet Per Second
FS	Feasibility Study
FY	Fiscal Year
GPM	Gallon Per Minute
IFAS	Integrated Fixed Film Activated Sludge
I/I	Infiltration and Inflow
IPR	Indirect Potable Reuse
kW	Kilowatt
LS	Lift Station
MBBR	Moving-Bed Bioreactor
MG	Million gallons
MGD	Million gallons per Day
NRCS	National Resources Conservation Services
O&M	Operation & Maintenance
ODEQ	Oklahoma Department of Environmental Quality
OWRB	Oklahoma Water Resources Board
PAS	Planning Assistance to States
QA/QC	Quality Assurance / Quality Control
RWD	Rural Water District
SSO	Sanitary Sewer Overflow
TL-WTP	Ted D Lockin Water Treatment Plant (City of Bartlesville)
TM	Technical Memorandum

Name	Abbreviation
USACE	US Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USCOE	US Corps of Engineers
USEPA	U.S. Environmental Protection Agency
VFD	Variable Frequency Drive
WQ	Water Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

1.0 INTRODUCTION

The purpose of this Technical Memorandum No. 3 (TM-3) is to update the 2010 Facility Plan Study which encompasses the existing collection, transport, and storage facilities along the conveyance corridor extending from the Limestone lift station to the Chickasaw wastewater treatment plant (CWWTP). This conveyance corridor (termed the Limestone-Chickasaw Corridor) includes the Limestone lift station and its forcemain, Limestone FEB, Golf Course lift station and its forcemain, Hillcrest lift station and forcemain, and Shawnee lift station and its forcemain along with its connection at the Chickasaw WWTP (CWWTP) and FEB.

Currently the Limestone lift station is only used to lift excess flow (wet weather flow) to the adjacent Limestone flow equalization basin (FEB). Normal and dry weather flows directly bypass the Limestone lift station and flow by gravity to the Golf Course lift station. They are pumped by the Golf Course lift station and then to the Hillcrest lift station before being pumped a third time to the CWWTP by the Shawnee lift station.

1.1 GENERAL DESCRIPTION

The entire Bartlesville wastewater collection system discharges to a total of 20 lift stations (plus one gravity force main), ranging in capacity from less than 10 gallons per minute (gpm) to as high as 14,000 gpm. The system includes three FEBs with capacities of 20.0 MG at the CWWTP, 5.9 MG at the Tuxedo lift station, and 6.0 MG at the Limestone lift station for a total of 31.9 MG of system storage. Table TM3.1 is a summary of the system wide FEBs, and Table TM3.2 is a summary of the key lift stations located along the Limestone-Chickasaw corridor. Also, the other lift stations that pump directly to the CWWTP are also included. This TM, however, only focuses on the conveyance corridor from the Limestone lift station to the Chickasaw wastewater treatment plant.

Figure TM3.1 is a map of Bartlesville that shows the layouts of the sewer basins, the locations of the main lift stations, and the CWWTP. Figure TM3.2 is a schematic of the existing collection system.

Table TM3.1 Flow Equalization Basins

Name	Year Constructed	Existing Capacity (MG)	Liner Type	Pre-sedimentation Basin Exists? (yes/no)	Liner
Tuxedo	1993	5.9	Synthetic	Yes	Synthetic
Limestone	1996	6.0	Synthetic	No	--
Chickasaw	1986	20.0	Synthetic	Yes	Concrete
Total Available Volume:		31.9			

Table TM3.2 Lift Stations and Force Mains

No.[1]	Name	Year Constructed[2]	Sub-Basins Served	Lift Station Firm Capacity		Force Main	
						Size	Length
				(gpm)	(MGD)	(in)	(ft)
7	Shawnee	1983	S01 - S15	2,999	4.32	18	10,900
8	Hillcrest	1983	S07 - S15	2,363	3.40	16	5,800
9	Golf Course	1983	S09 - S15	2,080	3.00	14	4,000
10	Limestone[3]	1999	Excess from S09 - S15	9,500	13.70	24	3,200
13	Woodland	2003	T07-T10	3,194	4.60	20	8,800
16	Tuxedo[4]	1983/1993/2003	T01 - T06	4,410	6.35	20	4,200
19	Chickasaw	1983/2003	C02-C07	11,200	16.10	18	80

Notes:
[1]Lift station numbers shown correspond to the numbering system utilized by the City. The remaining smaller pump stations found within the system can be seen in the 2013 “Final Report of the Collection System Analysis.”
[2]First year shown is year of original construction. Other years represent substantial improvements.
[3]Currently, the pump station only lifts excess wet weather flows to the FEB.
[4]Firm capacity includes the two main lift pumps that lift flow to the CWWTP. The firm capacity of the three storm flood pumps (12.9 MGD) that lift excess wet weather flows to the FEBs has not been included. Total station firm capacity (main lift plus storm = 19.25 MGD)

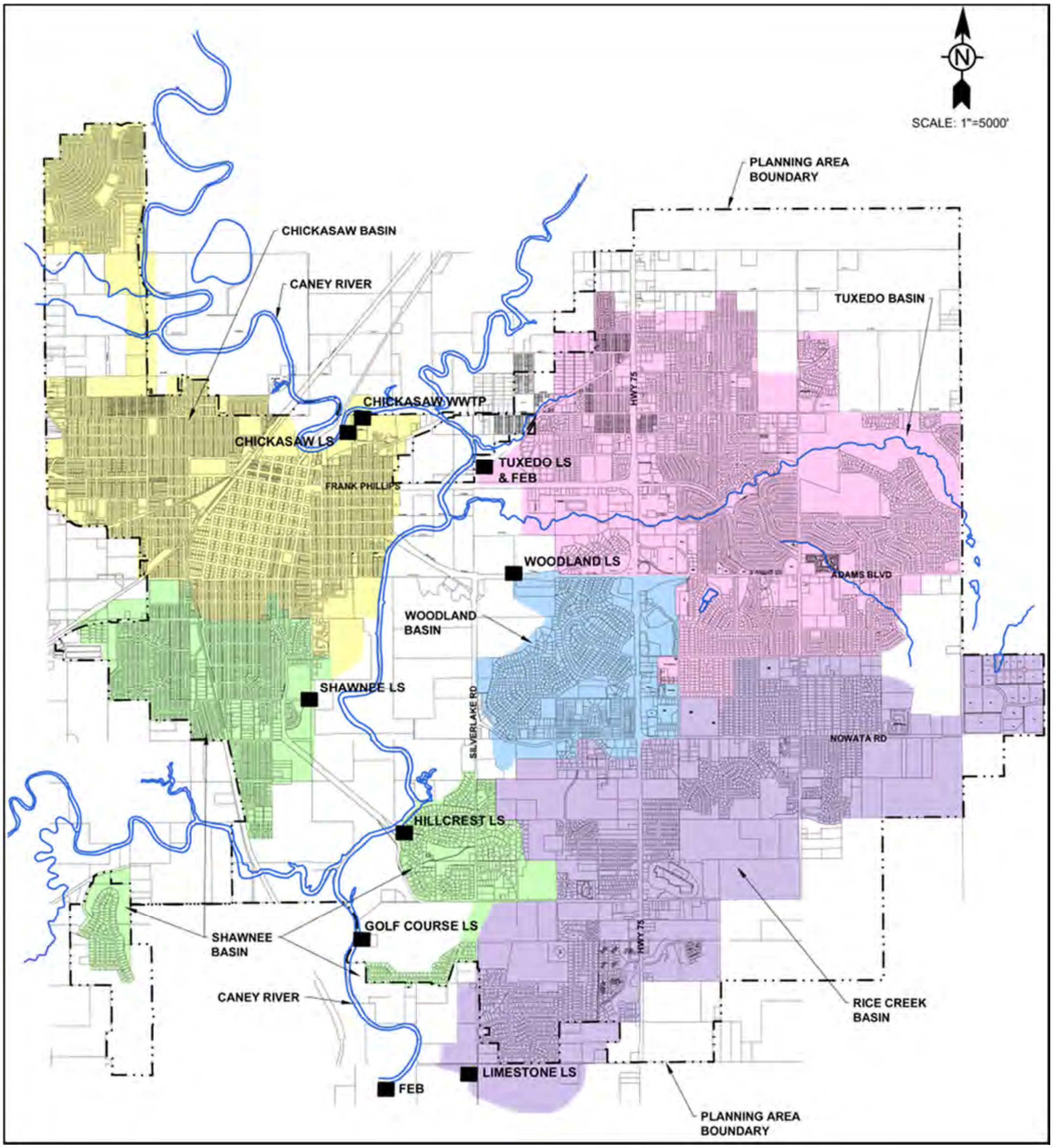


Figure TM3.1 - Main Lift Station and Sewer Basin Layout

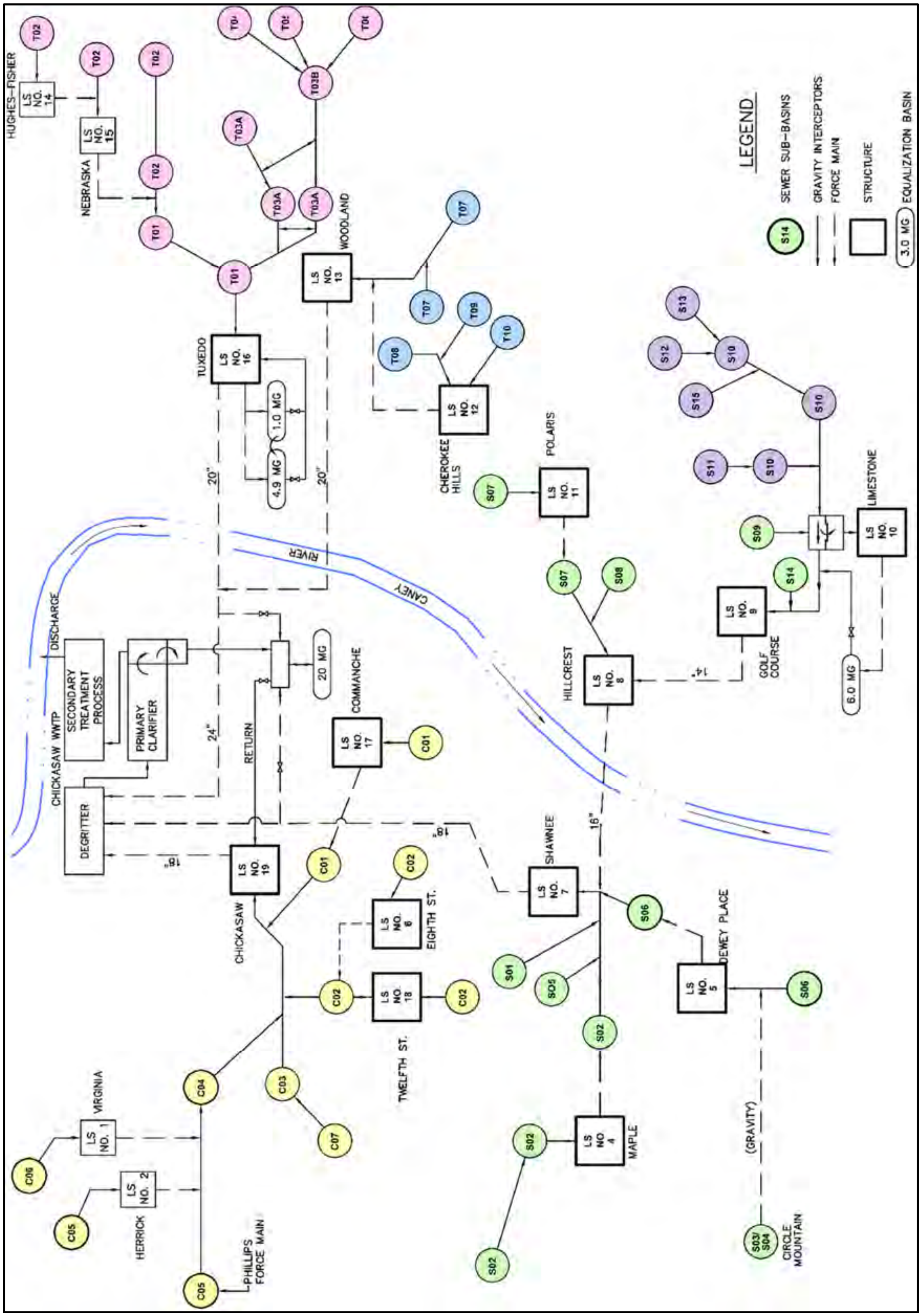


Figure TM3.2 - Wastewater System Schematic

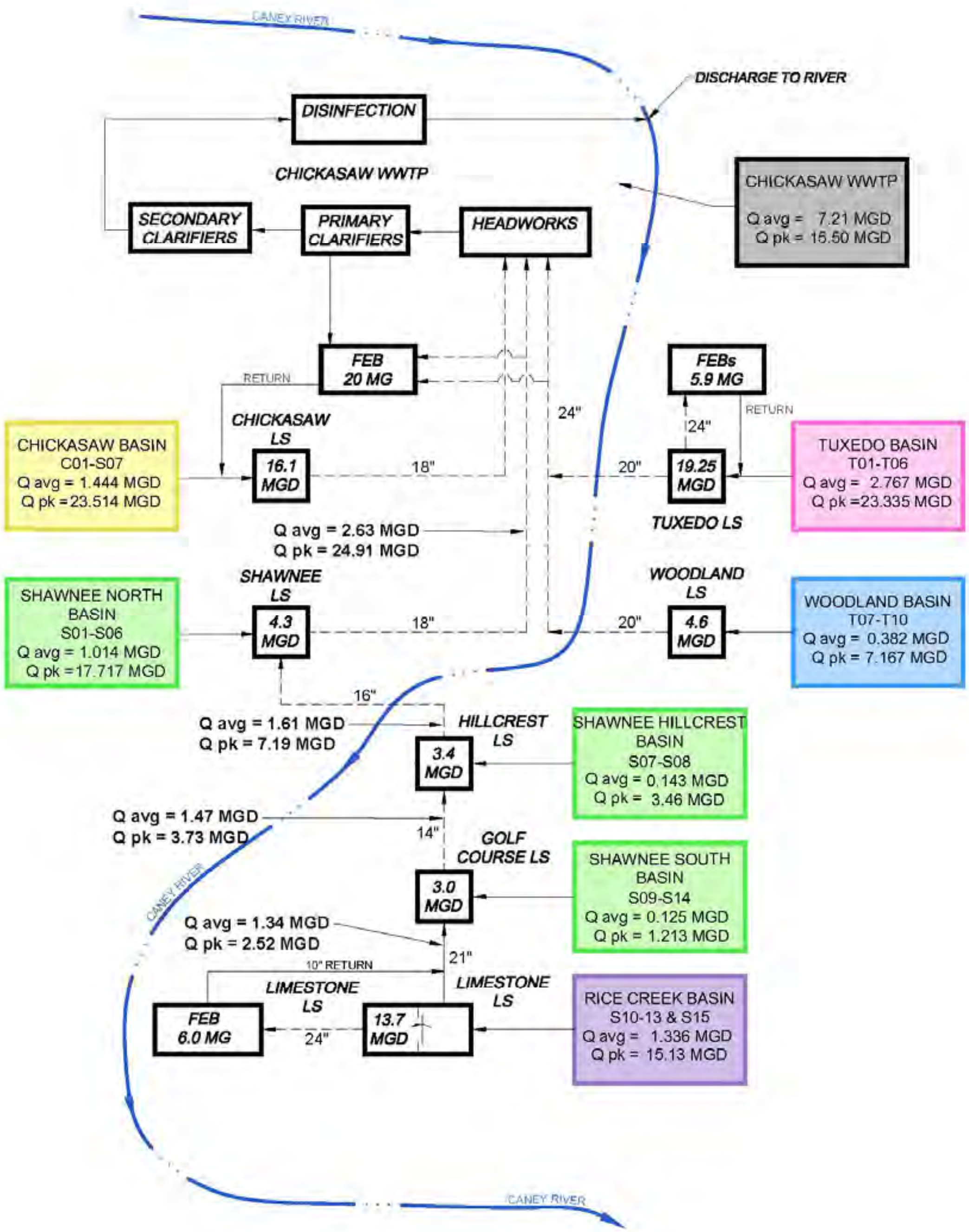
1.2 KEY ELEMENTS

The collection system elements that are of focus to this study are the Chickasaw, Shawnee, Golf Course, Hillcrest, and Limestone lift stations and their force mains. The required improvements to these facilities vary depending on the selected treatment alternatives. The Limestone and Chickasaw FEBs are also of key importance since the storage requirements vary based on the treatment alternative. The Tuxedo, Woodland and Chickasaw lift stations were also analyzed but are not part of the Limestone-Chickasaw corridor.

Figure TM3.3 is a schematic of the system which includes the current average and peak flow rates. On Figure TM3.3 the numbers inside the lift station boxes represent the current firm capacity of the station based on lift station assessments completed between 2012 and 2013 as indicated by Table 5.4 of "Final Report of the Update of the Collection System Analysis" dated 2013 (2012 analysis).

The peak flows as shown in Figure TM3.3 at each basin, represent peak flows developed from the 2012 collection system hydraulic model and analysis, and are considered as existing (current) flows based on the reasoning that the collection system conditions and flow have not changed substantially since the 2012 analysis. It is noted that the peak inflow component shown at each lift station (attenuated by the model) does not take into account any future I/I reduction due to the City's ongoing I/I mitigation efforts. The average flows as shown in Figure TM3.3 at each basin represent average annual daily flows based on the latest population distribution and recent flow data. Further information in regards to the current and projected flows can be found in TM1.

The model developed for the 2012 analysis was used to incorporate the design storm event based on recent historical data and estimate peak flow at each lift station contributed from its respective sub basins. A design event (5 yr-1 hr) was used in the model analysis to extract peak flows from the respective basins for the design storm event.



Q avg = Average Annual Daily Flow (2012)
Q pk = Unrestricted Model Flow with no I/I Reduction (2012) 5yr-1hr event.

Figure TM3.3 - Existing Main Transport and Treatment Facilities Schematic

2.0 EXISTING TRANSPORT FACILITIES

2.1 LIFT STATIONS ALONG THE LIMESTONE TO CWWTP CONVEYANCE

The collection system elements that are along the Limestone-Chickasaw corridor are the Shawnee, Golf Course, Hillcrest, and Limestone lift stations and their forcemains. The required improvements to these facilities vary depending on the selected treatment plant alternatives.

The Limestone lift station is only used to lift excess flow (wet weather flow) to the adjacent Limestone FEB. Normal and dry weather flows bypass the Limestone lift station and flow by gravity to the Golf Course lift station and are lifted in turn by the Golf Course lift station and the Hillcrest lift station before being pumped a third time to the CWWTP by the Shawnee lift station. Figure TM3.3 shows a schematic of these transport facilities. A detailed description of the Chickasaw lift station has been included in TM2. The Tuxedo and Woodland Lift Stations are also addressed separately in Appendix A.

The following sections describe the current condition and capacity of the lift stations and along the Limestone-Chickasaw corridor.

2.1.1 Limestone Lift Station

Figure TM3.4 shows the Limestone Lift Station which at this time is only used to lift excess flow (wet weather flow) to an adjacent FEB. Normal flows (dry weather flows) pass by the Limestone lift station by gravity to the Golf Course lift station.

The lift station was constructed in 1997/1998 and is equipped with 3 submersible pumps with a firm capacity of 13.7 MGD. The station is served by a standby power generator which automatically activates upon loss of power.

The FEB has a synthetic liner; however, it does not have a pre-sedimentation basin. The FEB is located in the floodway of the Caney River, and expansion may involve considerable regulatory review and possibly considerable mitigation measures for regulatory approval. The size of FEB expansion at this location will definitely be limited.

The lift station itself is located in the flood plain, and expansion outside the existing building footprint may involve additional regulatory review as is the case for FEB expansion.

Figure TM3.4 - Limestone Lift Station

The existing pumps are performing satisfactorily and the station piping is still in serviceable condition. The synthetic FEB liner still has useable life. Overall the lift station and FEB are in working condition.

The key characteristics and condition of the lift station are summarized in Table TM3.3.

Table TM3.3 Limestone Lift Station Summary	
Process Pumps (Pumping to FEB)	
Condition (1997)	Acceptable
Pump Type	Submersible
Number	3
Horsepower, Each	60
Capacity, Each (gpm)	6,350
Capacity, Firm (MGD)	13.7
Capacity, Maximum (MGD)	15.1
Force Main Size (inch. in diameter)	24
Screening	None
FEB (1997)	
Type	Earthen Synthetic Lined
Aeration	None
Capacity (MG)	6.0
Standby Power	
Type	Diesel
Number	1
Generator	199.8 kW

2.1.2 Golf Course Lift Station

The Golf Course lift station receives flow from gravity interceptors which serve the Rice Creek Basin (basins S10 through S13 & S15) and Shawnee South Basin (basins S09-S14), as well as return flow from the 6 MG Limestone FEB located near the Limestone lift station.

The lift station is equipped with 3 submersible pumps with a firm capacity of 3.0 MGD. The station is served by a standby power generator which automatically activates upon loss of line power. The standby generator is believed to be in acceptable working condition.

The wet well piping and isolation valves need to be replaced.

The key characteristics and condition of the lift station are summarized in Table TM3.4.

Table TM3.4 Golf Course Lift Station Summary	
Process Pumps	
Condition	Acceptable
Pump Type	Submersible
Number	3
Horsepower, Each	25
Capacity, Each (gpm)	1,300
Capacity, Firm (MGD)	3.0
Capacity, Maximum (MGD)	3.3
Force Main Size (inches in diameter)	14
Screening	None
Standby Power	
Type	Diesel
Number	1
Generator	60 kW

2.1.3 Hillcrest Lift Station

Hillcrest lift station receives flow from gravity interceptors which serve the Shawnee Basin (S07-S08) and discharge from the Golf Course lift station. The Hillcrest lift station discharges to the Shawnee lift station.

The station is equipped with 3 submersible pumps with a firm capacity of 3.4 MGD. That station is served by a standby power generator which automatically activates upon loss of power. The standby generator is believed to be in acceptable working condition.

The wet well piping and isolation valves need to be replaced.

The key characteristics of the lift station are summarized in Table TM3.5.

Table TM3.5 Hillcrest Lift Station Summary	
Process Pumps	
Condition	Acceptable
Pump Type	Submersible
Number	3
Horsepower, Each	25
Capacity, Each (gpm)	1,472
Capacity, Firm (MGD)	3.4
Capacity, Maximum (MGD)	3.6
Force Main Size (inches in diameter)	16
Screening	None
Standby Power	
Type	Diesel
Number	1
Generator	60 kW

2.1.4 Shawnee Lift Station

The Shawnee lift station (Figure TM3.5) is located approximately 1.5 miles south of the CWWTP. The lift station receives flow from gravity interceptors which serve basins S01 through S06 and the discharge from the Hillcrest lift station which handles flow from sewer basins S07 through S15. Flow from the station is discharged through an 18-inch force main to the headworks structure at the CWWTP. Flow in the force main can be diverted directly to the FEB at the CWWTP via a valved (manual) connection. The lift station is equipped with two vertical, dry-pit centrifugal pumps. There are connections and space left for a third pump to be installed. The station is served by a standby power generator which automatically activates upon loss of line power. The standby generator is believed to be in acceptable working condition. The station structure, pumps, and generator were all constructed in 1983. VFDs were installed for the pump drives in 2009.

Figure TM3.5 - Shawnee Lift Station

Table TM3.6 Shawnee Lift Station Summary

Process Pumps	
Condition	Acceptable
Pump Type	Centrifugal, Dry-Pit (1983)
Number	2
Horsepower, Each	150
Capacity, Each (gpm)	2,999
Capacity, Firm (MGD)	4.32
Capacity, Maximum (MGD)	5.53
Force Main Size (inches in diameter)	18
Control	VFDs (2009)
Screening	None
Standby Power	
Type	Diesel
Number	1
Generator	450 kW (1983)

The key characteristics of the lift station are summarized in Table TM3.6.

Grinders are in place for pump protection at the station. If the existing lift station is to be retained under the selected alternative, the pump isolation valves need to be replaced. A scrubber was added to address odors at the lift station. It is a leased unit.

The existing 18" Shawnee force main is suspected to have internal corrosion especially at the location of air relief valves. A complete condition assessment of the forcemain and the lift station are beyond the scope of this study. Based on its age and discussion with City staff, a new parallel forcemain will be included in the recommendation with the requirement that a more detailed conditional assessment be performed in the future to determine whether a complete or target segments of the forcemain need to be replaced. As a minimum, all high points in the force main should be replaced with non-metallic piping. All new piping should be non-metallic due to the corrosive conditions in the pipeline.

As an alternate to upgrading the Shawnee LS, constructing a new lift station south and east of the current location will be evaluated. For the purpose of this report, a new lift station and force main will be assumed for both improvement alternatives. The size of the force main and required pumping capacity of the Lift Station varies depending on which alternative is selected. Refer to Figure TM3.8 and Figure TM3.15 for details. An aerial view of the proposed Shawnee LS location can be seen in Figure TM3.7. An aerial view of the proposed forcemain extending from the Shawnee LS to the CWWTP can be seen in Figure TM3.6.

2.2 INTERCEPTOR SEWERS

The main gravity interceptor of the Limestone-Chickasaw conveyance corridor is the line beginning at the Limestone Lift Station and ending at Golf Course Lift Station. The line is approximately 5,500 LF of 21 inch gravity sewer that drains the Rice Creek Basins and Limestone FEB. This interceptor is believed to be in serviceable condition and is rated to deliver the maximum flow that Golf Course Lift Station can be expected to currently handle. Should greater flow to Golf Course Lift Station from Limestone Lift Station be desired, the line will need to be upsized.

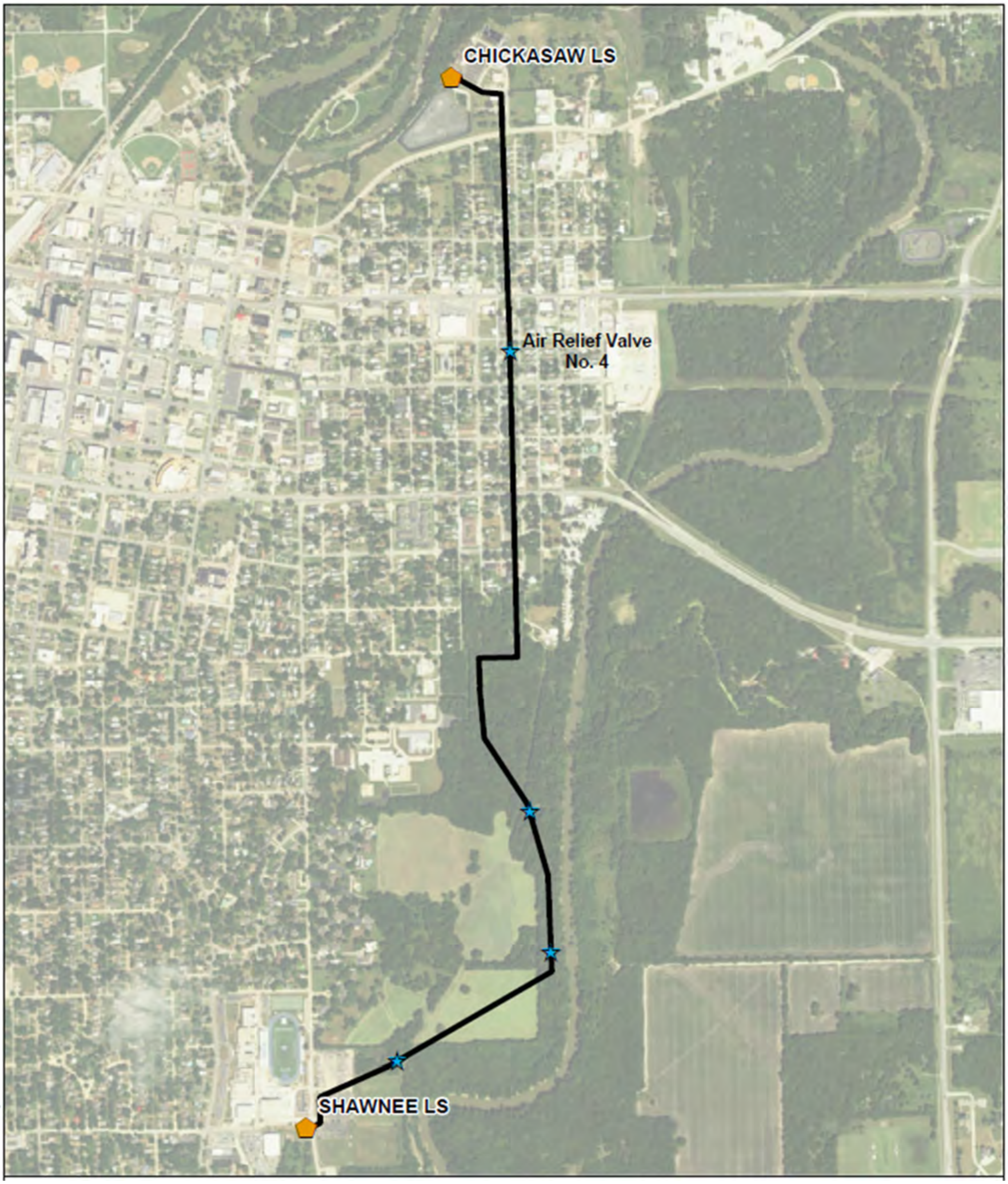


Figure TM3.6 - Proposed layout of the new forcemain from Shawnee LS to CWWTP

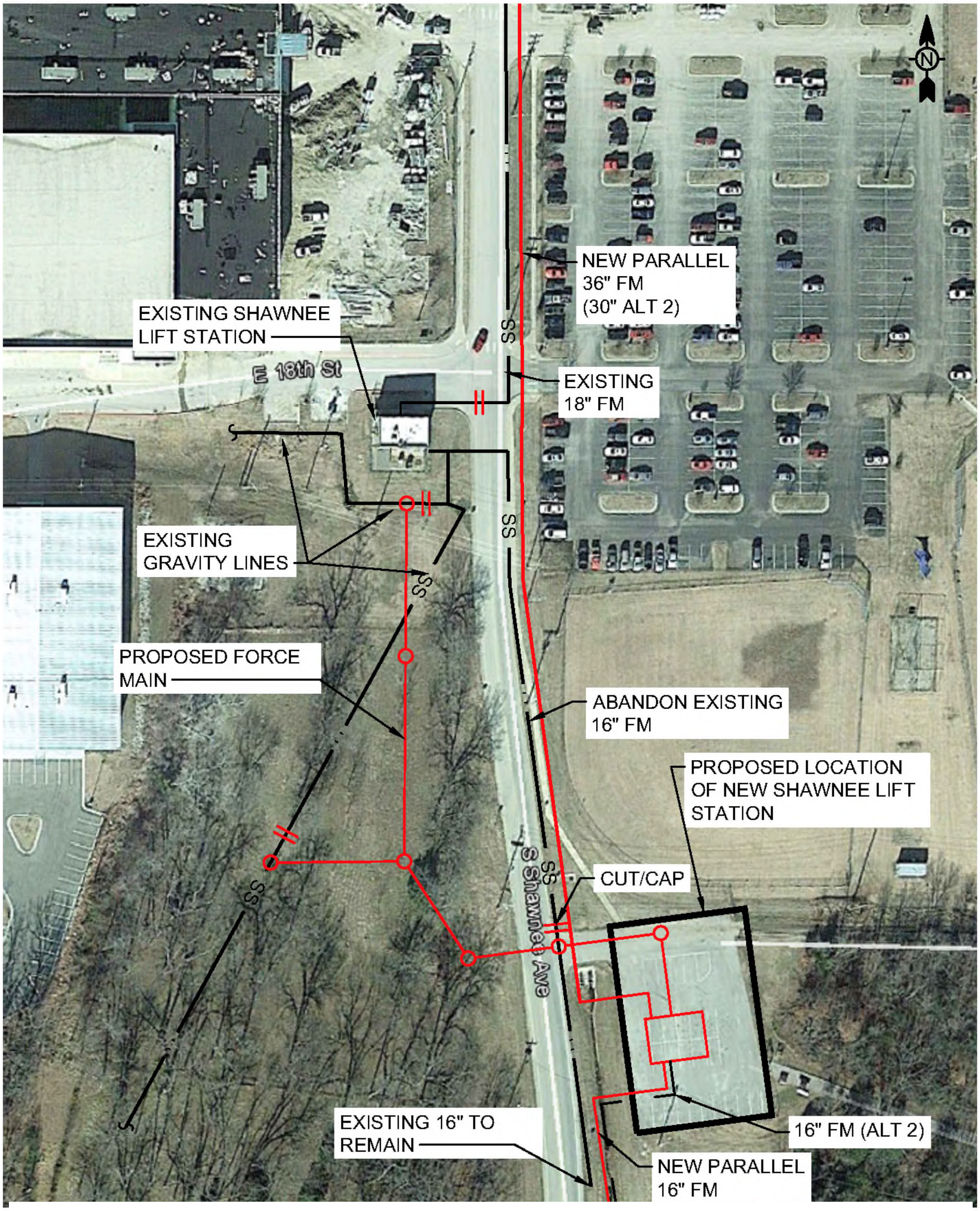


Figure TM3.7 - Proposed location of new Shawnee lift station

2.3 SANITARY SEWER OVERFLOWS

Bartlesville has aggressively addressed sanitary sewer overflows (SSOs) and has seen significant reduction in their occurrence. The Limestone-Chickasaw corridor is the last area that still has significant SSOs in the city. These shall be mitigated either through capacity enhancements along this corridor, and/or through flow reduction by construction of a new wastewater treatment plant in the south that will divert some of the flow away from this corridor.

3.0 DESIGN FLOWS

Three definitions of flows were utilized in this report: average daily flow on an annual basis, average daily dry weather flow, and peak flows. Average flows were based on the data as reported in the TM1 – “Population, Flow, and Wasteload Updates” to reflect flow data from the CWWTP and the new population projections. It was determined that there was very minimal difference in population estimates from 2012 to 2015, therefore an update to average (base) flows was determined to be unnecessary. Table TM3.7 is a breakdown of the average annual flow, average daily flow and peak flow rates from the 2012-2013 analysis and projected 2050 flows.

Annual average daily flows were calculated based on the monthly operating reports. The average annual daily flows include wet weather days in the calculation.

The average dry weather flows were based on 2012 flow monitoring and hydraulic modeling completed by Tetra Tech after the 2010 Facility Report was prepared. This is the most recent information available for this study.

Peak flows are used to size the transport and storage facilities. To establish the peak influent flows in the system under the current and future average base flow conditions, the following steps were taken.

For the 2012 and the 2050 model simulations:

- The hydraulic model was run for a 5-year, 1-hour storm event to determine the peak inflow from each collection system basin.
- Base flows were distributed using the population distribution shown in Table TM1.4 of Tech Memorandum TM1 – “Population, Flow, and Wasteload Updates” and the current and future projected populations.

For the 2050 model, the following adjustments were made to model output:

- The inflow components (attenuated by the model) from the collection system were reduced by 20 percent to account for future inflow removal rates as a result of the City’s continued effort to reduce I/I in the system. The resulting rates were then increased by 5 percent to account for potential inflow from future new construction.
- The adjusted inflow rates and the adjusted base flows (average annual) were added together to establish a new peak design rate for each basin.

Table TM3.7 Average Daily and Peak Flows

	2012 Analysis Design Flows				2050 Projected Flows			
Collection System Basins	Average Dry Weather Flow (MGD)	Average Annual Daily Flow (MGD)	Peak Inflow (MGD, 2012 Calibrated Model 5-yr, 1-hr Storm)	Total Peak Flow (MGD)	Average Dry Weather Flow (MGD)	Average Annual Daily Flow (MGD)	Peak Inflow (MGD, Model 5-yr, 1-hr Storm)	Total Peak Flow (MGD)
Chickasaw	1.209	1.444	23.514	24.958	1.393	1.643	19.752	21.395
Tuxedo	1.611	2.767	23.335	26.102	1.826	3.149	19.601	22.750
Woodland	0.228	0.382	7.167	7.549	0.285	0.435	6.020	6.455
Shawnee								
North	0.600	1.014	17.717	18.731	0.702	1.153	14.882	16.036
Hillcrest	0.092	0.143	3.460	3.603	0.112	0.162	2.906	3.069
South	0.066	0.125	1.213	1.338	0.085	0.142	1.019	1.161
Rice Creek	1.162	1.336	15.130	16.466	1.375	1.521	12.709	14.230
Totals:	4.968	7.211	--	--	5.778	8.205	--	--

4.0 ALTERNATIVES ANALYSIS

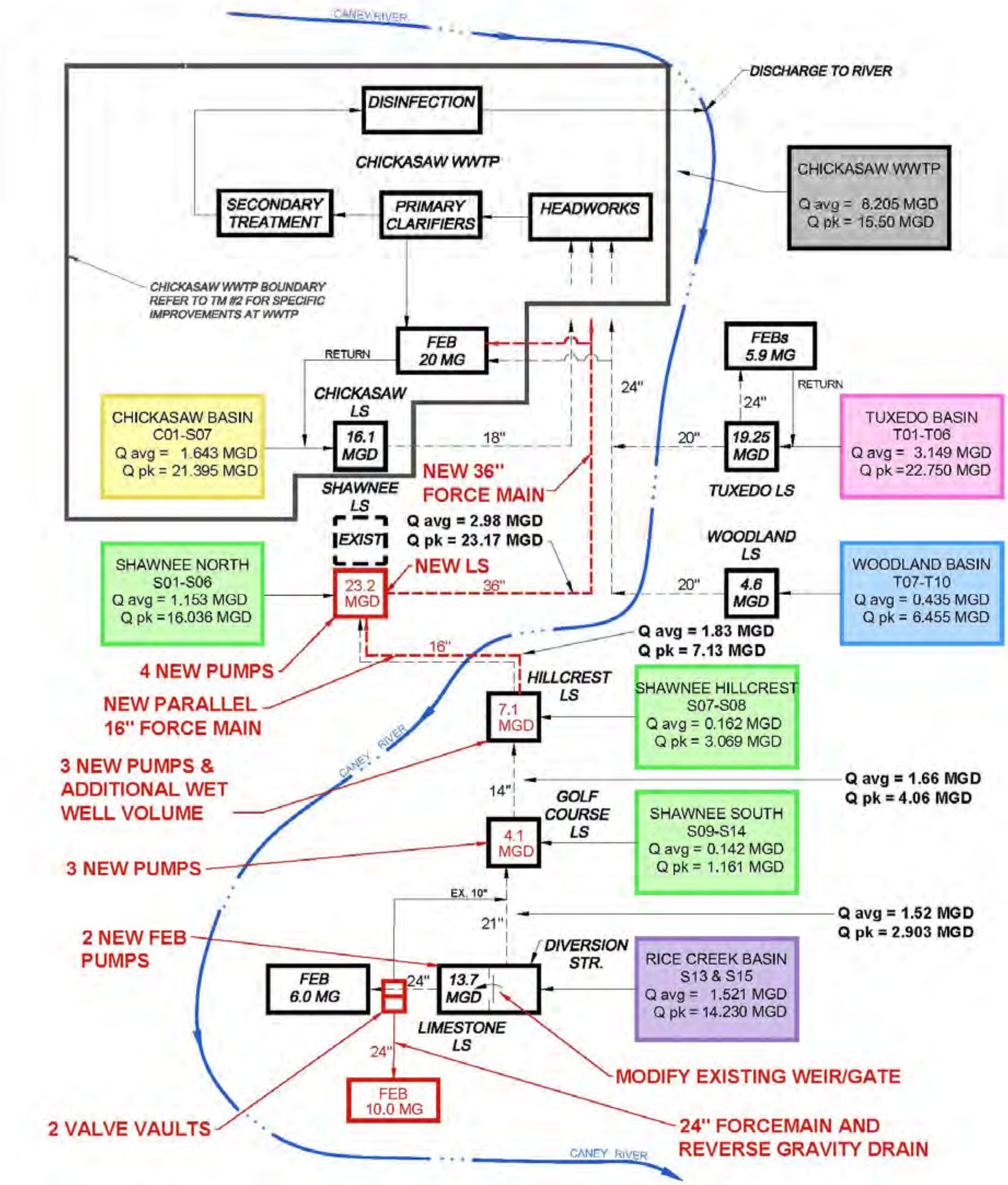
The 2010 Facility Plan developed two alternatives to address future treatment requirements and wasteloads in the Bartlesville collection system. Alternative 1 maintained the current practice of transporting all flows to and treating all flows at the CWWTP. Alternative 2 added a second treatment facility at a location south of the City. These two alternatives have been re-evaluated based on the current conditions of the existing transport facilities, current construction costs, and the revised flow and plant loadings developed for the 2050 planning year.

Both alternatives include updating the condition and capacity evaluations of each unit process at the CWWTP, updating the CWWTP components requiring improvement or replacement (as discussed in Technical Memorandum 2), and updating the Limestone-Chickasaw corridor transport facilities condition and capacity evaluations (TM3).

In each alternative, the concept of reuse will be incorporated as part of the analysis. The two alternatives are discussed in the following sections.

4.1 ALTERNATIVE 1- ALL FLOWS TO EXISTING CWWTP

Alternative 1 maintains the current practice of transporting all flows to and treating all flows at the CWWTP as shown in Figure TM3.8. In addition to facility improvements at the CWWTP (discussed in TM2), this alternative considers the complete replacement of the Shawnee force main, the replacement of the Shawnee lift station, and the addition of storage volume to the Limestone FEB at the recently acquired Archambo site.



Q avg = Projected Average Annual Daily Flow (2050)
Q pk = 2050 Model Projected Peak Flow

Figure TM3.8 - Proposed layout of the transport facilities required along the Limestone - Shawnee corridor for Alternative 1

4.1.1 Gravity Collection System Improvements

Under Alternative 1, no gravity interceptors will require modification. Under this alternative, the existing 21" gravity sewer line was evaluated at the existing designed slope (.08%) and it was determined that 2.903 MGD is the maximum pipe capacity the 21" can carry and is the assumed 2050 projected peak flow that will bypass the Limestone LS.

It is noted that the focus of this study is the conveyance corridor from the Limestone lift station to the Chickasaw wastewater treatment plant. Other interceptors in the various basins around the collection system (which are not included in this study) may also need to be upgraded for capacity, but those improvements are not covered by this report. See the "Final Report of the Update of the Collection System Analysis" dated 2013 for details of basin-wide improvements.

4.1.2 Lift Stations and Force Mains Improvements

Alternative 1 will require improvements to three main lift stations and force mains as summarized below. An overview of the alignment and lift station improvements can be seen in Figure TM3.9.

4.1.2.1 Golf Course Lift Station and Force Main

The Golf Course lift station will require new pumps and discharge piping. Three new 50-horsepower submersible pumps will be installed in the existing wet well each with a capacity of 2,200 gpm yielding a firm capacity (2 pumps) of 2,850 gpm (4.1 MGD). Each pump will be equipped with a VFD. The existing wet well should be adequate. A new 250 kW generator will be required along with new controls. At the design firm capacity, the velocity in the existing 14-inch force main would approach 5.9 ft/sec which is acceptable. The station piping must be increased from 8 inches to 12 inches.

4.1.2.2 Hillcrest Lift Station and Force Main

The Hillcrest lift station will require new pumps and discharge piping. Three new 25-horsepower submersible pumps will be installed in the existing wet well each with a capacity of 2,600 gpm yielding a firm capacity (2 pumps) of 4,950 gpm (7.1 MGD). Each pump will be equipped with a VFD. The existing wet well will need to be expanded to provide another 4,000 gallons of usable volume. A new 120 kW generator will be required along with new controls. At the design firm capacity, the velocity in the existing 16-inch forcemain would approach 8 ft/sec. The existing 16-inch forcemain will need to be increased in capacity by the addition of a second parallel 16-inch forcemain extending from the lift station to the Shawnee lift station (with a crossing of the Caney River) in order to reduce line velocities to approximately 4 ft/sec. The station piping must be increased from 8 inches to 12 inches.

4.1.2.3 Shawnee Lift Station and Force Main

The 2010 Wastewater Treatment Facility Plan concluded that the existing Shawnee lift station should have new pump replacement at a minimum, but after further evaluation and discussion with the City, it has been determined that a new relocated lift station will be necessary. Space for additional pumps is limited at the LS, and due to the large increase in needed pumping capacity, it is unlikely 3 additional new large pumps would fit within the building footprint. The building and wet well would have to be extended to accommodate the additional pumps. Due to the deteriorating conditions of the LS, constructing a new LS is believed to be a more viable option when considering the existing LS is approaching the end of its design life. In addition, odor issues due to its close proximity to the high school necessitates its relocation.

The new Shawnee lift station will be approximately 60 ft x 60 ft with a 30 ft deep wet well and will have 4 new 125-horsepower submersible pumps, each with a capacity of 6,250 gpm yielding a firm capacity (3 pumps) of 16,110 gpm (23.2 MGD). Each new pump will be controlled by a variable frequency drive (VFD). A new 600 kW generator is required for the new loads, and new pump controls will be required. At the design firm capacity, the velocity in the existing 18-inch forcemain would be well over 20 ft/sec. The existing 18-inch force main will need to be increased in capacity to a new 36” force main from the lift station to the CWWTP in order to reduce line velocities to approximately 5.1 ft/sec. The station piping (the header) will be 30 inches in diameter.

If the existing 18-inch force main is to remain in service, and used as a standby in conjunction with the proposed 36-inch line, several improvements would be necessary. The existing 18” Shawnee force main is suspected to have internal corrosion especially at the location of air relief valves. As a minimum, all high points in the force main should be replaced with non-metallic piping. All new piping should be non-metallic due to known corrosive soil conditions along the pipeline alignment. A complete condition assessment of the forcemain and the lift station are beyond the scope of this study, therefore, the costs associated with these improvements have not been included in this study. Costs for the rehabilitation of this line are not included in this memorandum.

4.1.2.4 Limestone Lift Station

Under this alternative, the Limestone lift station will be modified to lift a portion of peak flow from the Shawnee and Rice Creek basins (sub-basins S01 – S15) to a new south FEB. The existing pumps are sufficient to fill the existing 6 MG FEB, but because the proposed 10 MG FEB will be over a mile away, higher head conditions will necessitate two new pumps. The existing Limestone station was constructed with space and piping for the addition of these pumps. The two new main lift pumps will handle flows up to 10 MGD that will be pumped to the new FEB. Each pump will have a capacity of 4,800 gpm (6.9 MGD) yielding a combined capacity (2 pumps) of 10 MGD. Each pump will be 100-horsepower. Each pump will be equipped with a VFD. A second 200 kW standby power generator will be added. A proposed 24” force main that will also serves as a gravity drain from the proposed FEB will connect the Limestone lift station to the new south FEB.

A flow diversion structure (valve box) will be installed next to the existing Limestone FEB concrete inlet/outlet structure. A connection to the structure will be installed as well as a 24” bypass line and motor-operated valve. Once the existing 6 MG FEB becomes full, excessive flows under storm events will be diverted to the proposed 10 MG FEB via the new 24” FM using motor operated valves. The new FEB will be designed to utilize the 24” FM as a dual gravity drain that will empty back to the existing 10” gravity sewer located near the existing FEB once peak flows have subsided. Combination air/vacuum valves will be installed in order to mitigate the effects of any air pockets that would otherwise form. A motor-operated valve will allow the user to modulate the flowrate at which the new FEB will empty. Figure TM 3.10 shows the alignment and profile of the proposed 24” line. Figure TM 3.11 shows a schematic of the valve box locations and proposed 24” line.

The City has since acquired property for new wastewater facilities not shown in the 2010 Facility Report. The new proposed location of the property can be seen in Figure TM3.16. The existing three 6,350 gpm pumps currently at the Limestone lift station which lift excess (storm) flow to the Limestone FEB will remain in service and do not require modification. The firm capacity of these pumps is 13.7 MGD. Figure TM3.9 shows the proposed alignment for the 24” FM/ Gravity line in Alternative 1.

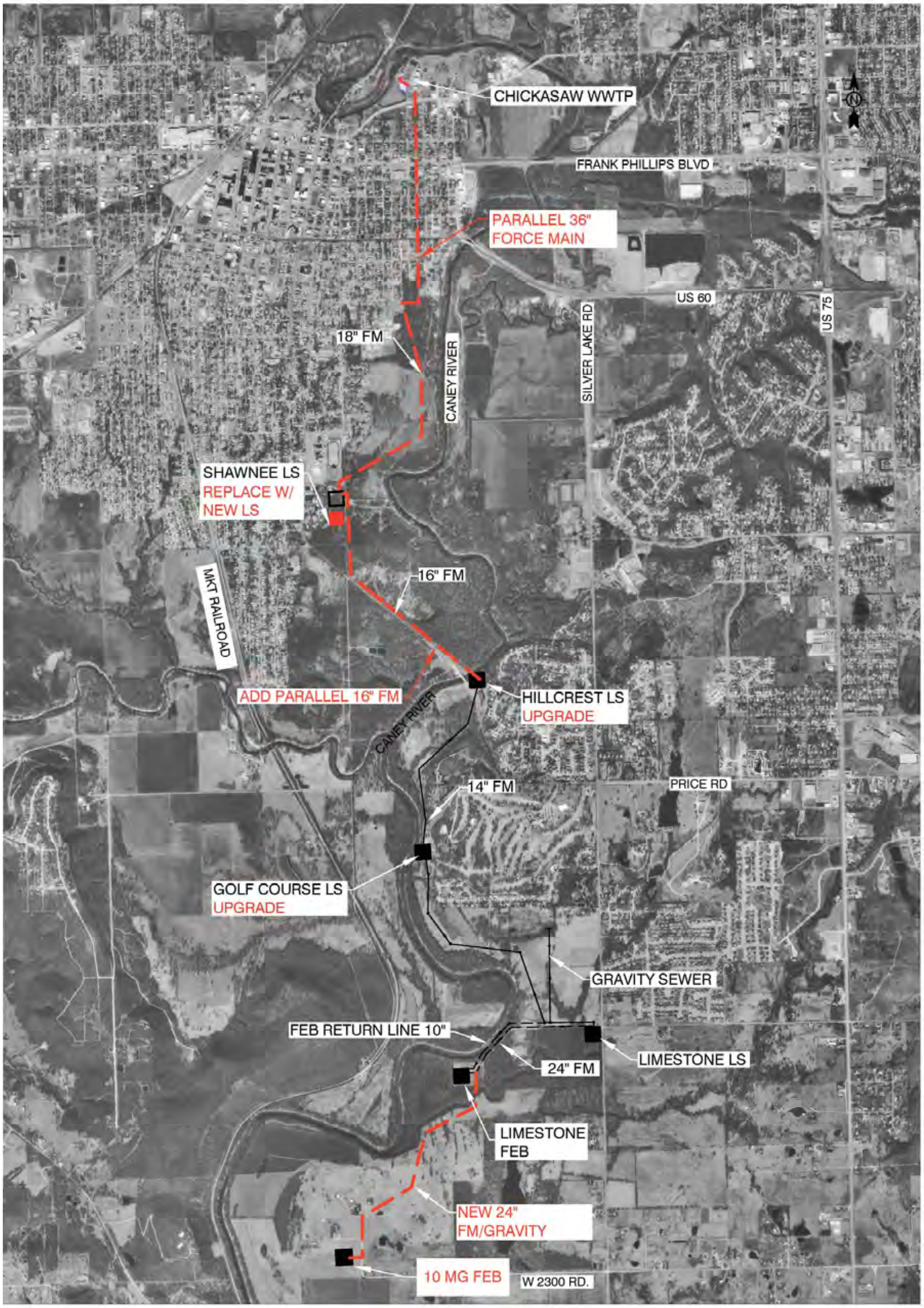


Figure TM3.9 - Alternative 1 Alignment

4.1.3 FEBs Improvements

Alternative 1 will require the following FEB improvements.

4.1.3.1 Limestone FEB

In order to evaluate storage capacity of the Limestone FEB, the 2012 model was loaded with 2008 and 2015 wet weather months, and a hydrograph was created at the outfall of the contributing basins near the Limestone LS. The wet weather months of May 2008 and May 2015 were selected because they contained the worst 7-day events in 10 years as required by DEQ standards. The dry weather flows were adjusted to account for 2050 population projections based on Table TM1.4 of the TM1 Report - “Population, Flow, and Waste Load”. The inflow component was extracted from the wet weather month hydrograph. A reduction of 20% was removed from the inflow component to reflect future I&I reductions (as a result of the continued I/I reduction efforts by the City) and 5% was added back for new construction to be consistent with the modeled flows as shown in Table TM3.7 of this report.

The modeling performed as part of the “Final Report of the Update of the Collection System Analysis” completed in January 2013, determined that under the conditions of Alternate 1, the South Service Area near Limestone FEB will require 15.1 MG of total storage (assuming I&I abatement is performed). Subtracting the existing Limestone FEB storage volume of 6 MG, the additional storage needed is 9.1 MG. More recently and as described above, the model was loaded with a 2015 wet weather month because it contained one of the worst 7 day events in 10 years. After further analysis of this storm event, the storage requirements in the south service area will require 15.4 MG of total storage. Subtracting the existing 6 MG volume, the additional storage needed is 9.4 MG. For the purpose of this report, a 10 MG FEB will be used for cost estimating purposes. Due to floodway restriction concerns, the additional storage must be placed at the new site south of the Limestone FEB and Lift Station.

This evaluation is based on capping the flow in the existing 21-inch gravity sewer, which carries flow from the Limestone LS to the Golf Course LS to 2.9 MGD, which is believed to be its estimated capacity. The proposed schematic of the Limestone Lift Station and FEB piping is shown in Figure TM 3.10.

Figure TM3.10--New FEB and Pipeline Plan and Profile

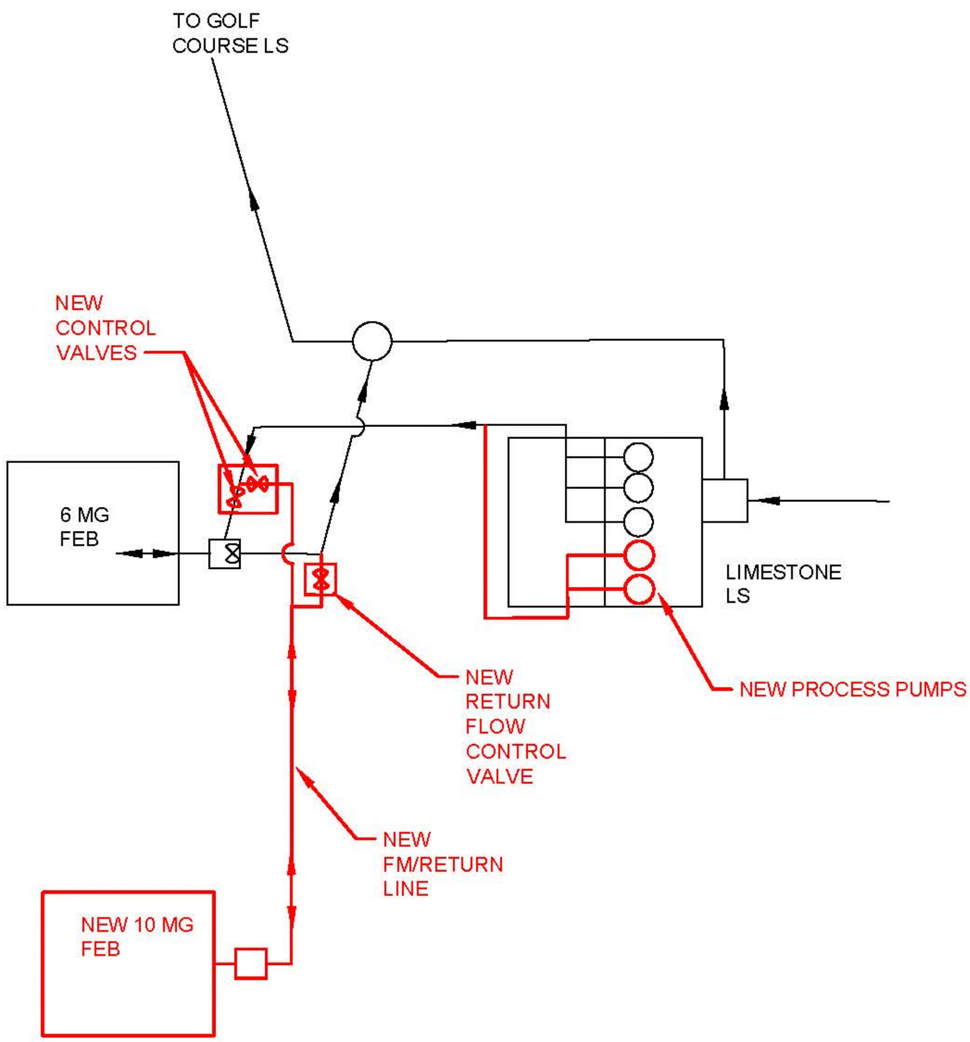


Figure TM3.11—Alternative 1 Limestone LS and FEB Schematic

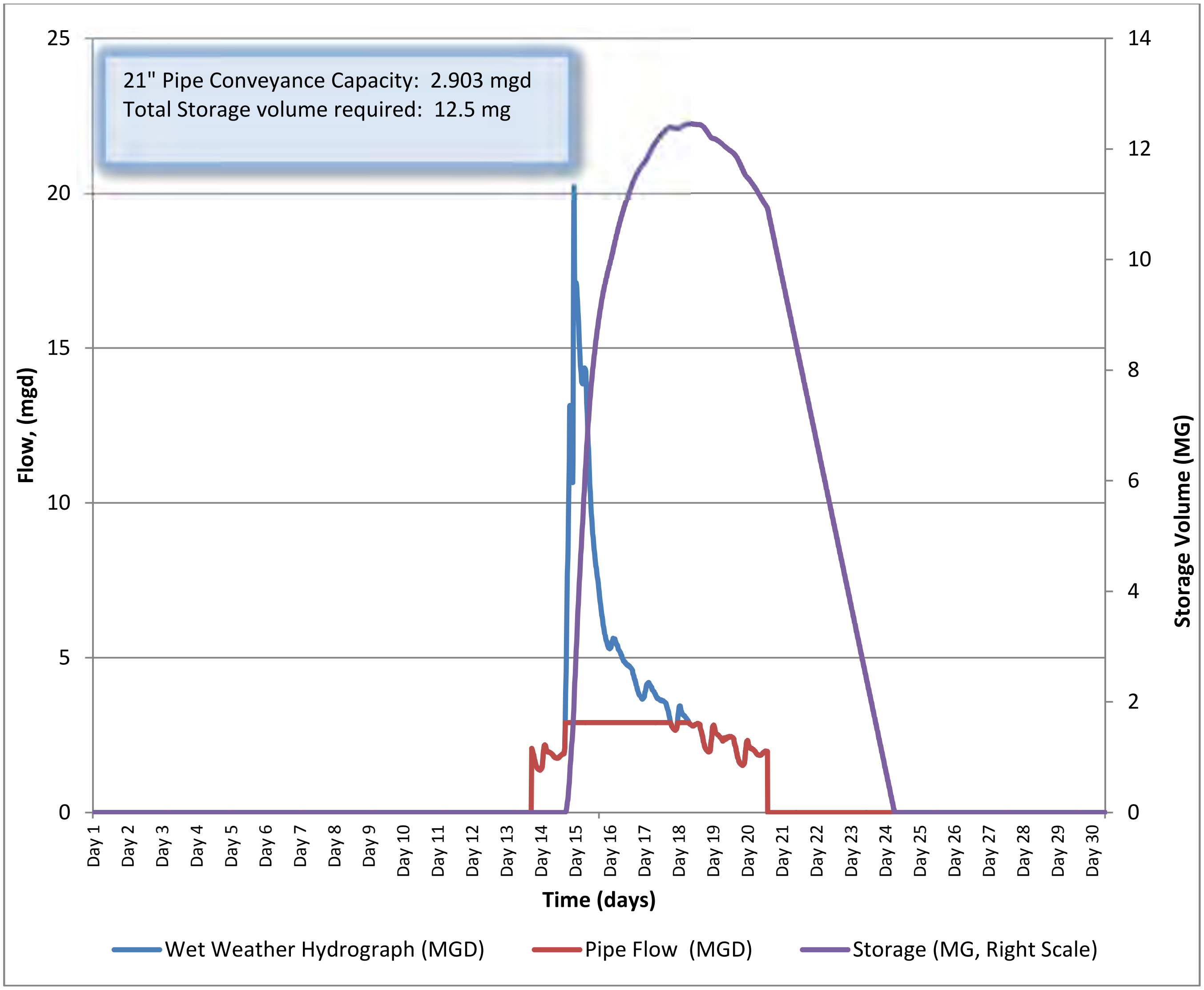


Figure TM3.12 – Alternative 1-Limestone FEB Volume Need Using May 2008 Wet Weather Month

As an additional design consideration for sizing the required Limestone storage volume under Alternative 1, events of May 2008 and May 2015 were also evaluated. A hydrograph was created using the same steps as specified for the May 2008 event. These events were selected because they produced the largest peak flows recorded in the last 10 years.

After further evaluation, it was determined that the May 2015, required a greater storage volume than the 2008 event. Because of this, the May 2015 event was used to size the south service area FEB. Refer to Figure TM3.13 below for further information.

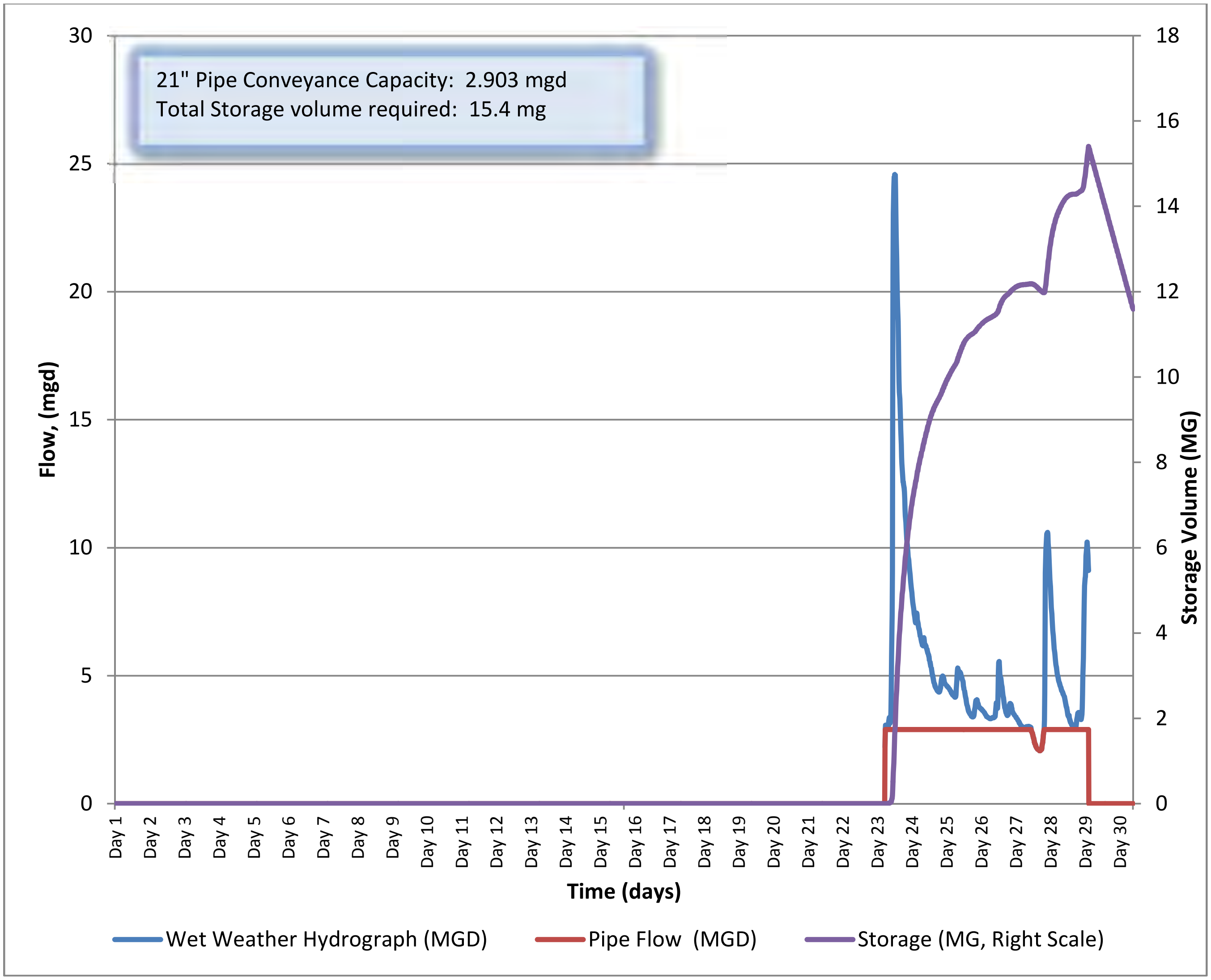


Figure TM3.13 - Alternative 1-Limestone FEB Volume Need Using May 2015 Wet Weather Month

4.1.3.2 Chickasaw FEB

As demonstrated by the "Update of the Collection System Analysis" completed in 2013, no capacity improvements to the 20 MG Chickasaw FEB will be required under this alternative. As an additional check, the same 2008 wet weather event (and 2012 calibrated model) that was used to evaluate the Limestone FEB storage capacity, was also applied to the north service area (refer to Figure TM3.14 below) in order to confirm there was adequate storage capacity at the existing Chickasaw FEB. After further evaluation, the conclusions were the same. No capacity improvements to the Chickasaw FEB are required for this alternative.

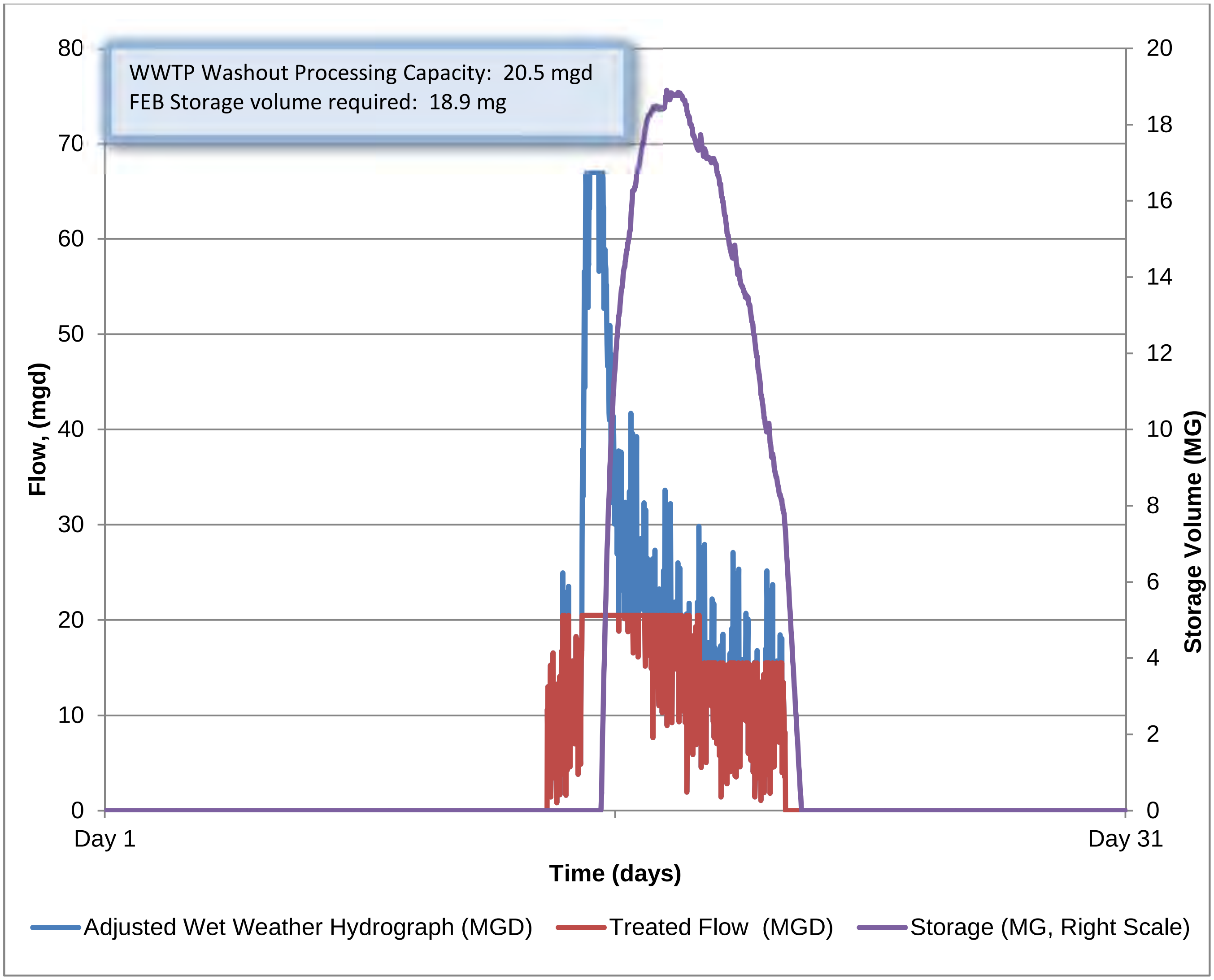
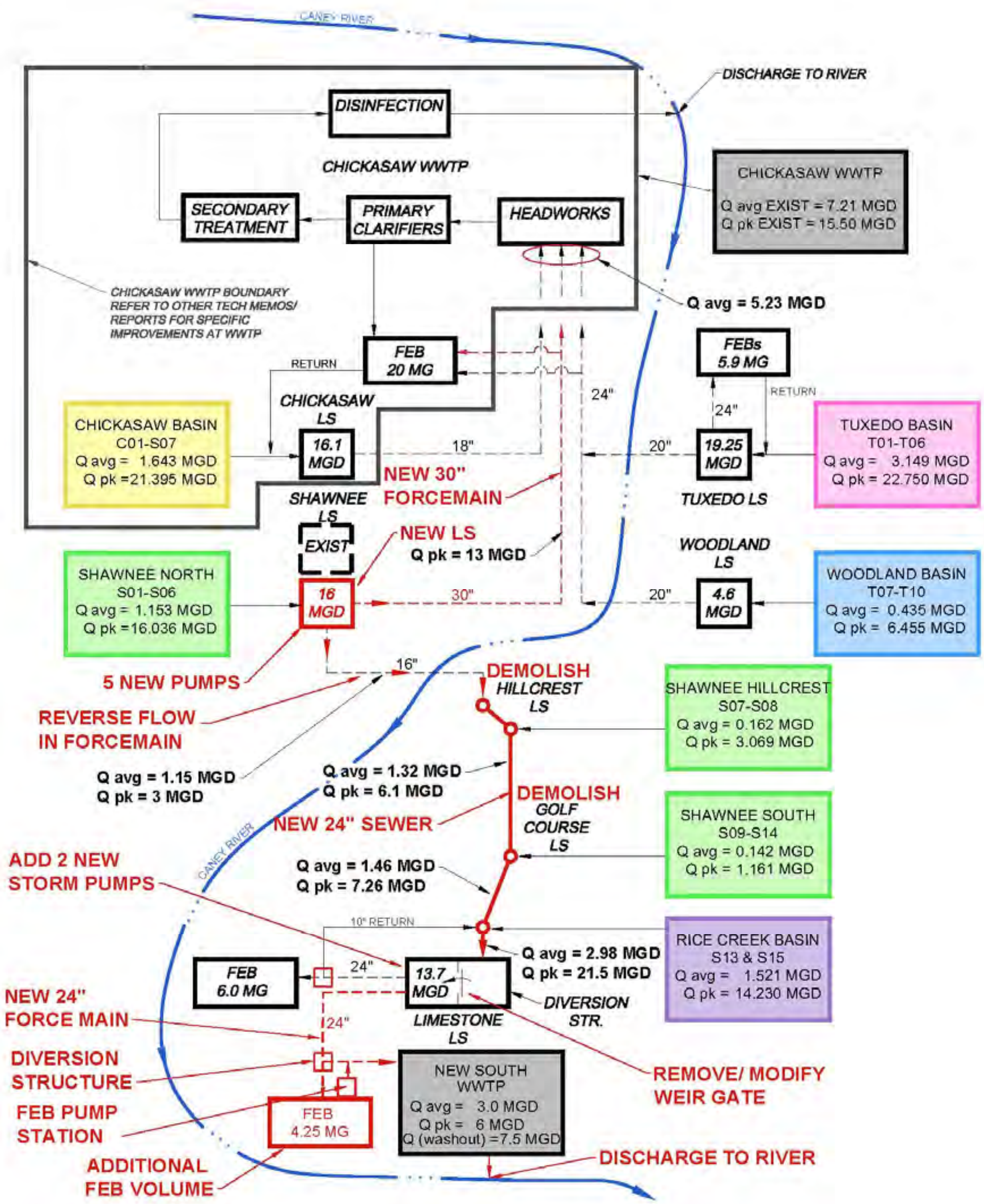


Figure TM3.14 - Alternative 1-Chickasaw FEB Volume Need Using May 2008 Wet Weather Month

4.2 ALTERNATIVE 2 – CONSTRUCT NEW SOUTH PLANT

Alternative 2 adds a second treatment facility at a location south of the City. This alternative also incorporates the concept of reuse, including the potential for a second discharge from the CWWTP upstream in the Caney River discharge.

In discussion with City staff it is assumed that up to 4 MGD of reuse could be utilized during the planning period. Therefore, for this alternative, a minimum 4 MGD flow diversion to the CWWTP is necessary. Projected 2050 average annual daily flows are 8.21 MGD with flows from Chickasaw, Tuxedo, and Woodland basins contributing a total of 5.23 MGD that will be directed to the existing CWWTP. This will leave 2.98 MGD average annual daily flow that will be diverted to the new south treatment plant. Refer to Table TM3.7 for details regarding 2050 projected total average and peak flows for the Bartlesville collection system basins. Figure TM3.15 shows the proposed layout of the transport facilities required along the Limestone - Shawnee corridor for Alternative 2.



Q avg = Projected Average Annual Daily Flow (2050)
Q pk = 2050 Model Projected Peak Flow

Figure TM3.15 - Proposed Layout of the Transport Facilities Required Along the Limestone - Shawnee Corridor for Alternative 2

4.2.1 Gravity Collection System Improvements

Under this alternative, a new 24" gravity interceptor would be installed along the east side of the Caney River. The line size was selected because it has the capacity to convey the 2050 projected peak flows. This pipe size would also provide more than 2 ft/sec of scour velocity based on the anticipated slope of the line that can be expected after review of the existing terrain and existing parallel 21" gravity sewer alignment. It will take flow from the Shawnee lift station (by reversing the flow in the existing Hillcrest force main) and sub-basins S07 and S08 at the existing location of the Hillcrest lift station and take the combined flow south to the Limestone lift station (for pumping to the new south treatment facility). The line would also collect flows from sub-basins S09 and S14 and the return line from the Limestone FEB. A portion of this line near the golf course lift station will have to be directional drilled as the existing terrain limits the possibility of open-cut just east of the river near the bluffs. As with Alternative 1, some interceptors in the various basins around the system may also need upgraded for capacity, but those improvements are not covered by this report. See the "Final Report of the Update of the Collection System Analysis" dated 2013 for this improvement.

4.2.2 Lift Stations and Force Main Improvements

Alternative 2 will require improvements to two main lift stations and force mains and will include the demolition of two lift stations (and abandonment of one force main) as summarized below.

4.2.2.1 Golf Course lift Station and Force Main

The Golf Course lift station will be removed from service and demolished. The existing 14" force main will be abandoned. Wastewater flow that gravity feeds to the Golf Course lift station (from sub-basins S09 and S14) will be handled by the new 24" gravity interceptor and be routed south to the Limestone lift station.

4.2.2.2 Hillcrest Lift Station and Force Main

The Hillcrest lift station will be removed from service and demolished. However, the existing 16" force main will be reused and its flow will be reversed. The Shawnee lift station will be connected to this force main (see below) and send up to 3 MGD of flow back towards the Hillcrest lift station under wet weather conditions, discharging to the upper end of the new interceptor sewer discussed above. Wastewater flow that gravity feeds to the Hillcrest lift station (from sub-basins S07 and S08) will be handled by the new 24" gravity interceptor and be routed south to the Limestone lift station.

4.2.2.3 Shawnee Lift Station and Force Main

As mentioned in Alternative 1, the 2010 Wastewater Treatment Facility Plan concluded that the existing Shawnee lift station should have new pump replacement at a minimum, but after further evaluation and discussion with the City, it has been determined that a new relocated lift station will be necessary. The new Shawnee lift station will have three new 100-horsepower submersible pumps, each with a capacity of approximately 5,850 gpm yielding a firm capacity (2 pumps) of 9,000 gpm (13 MGD) as well as two new 25-horsepower submersible pumps, each with a capacity of approximately 2,100 gpm (3 MGD). This will result in a total firm capacity of 16 MGD for the Shawnee LS. Each pump will be equipped with a VFD. The two 25-horsepower pumps will pump weather flows up to 3.0 MGD south to the new WWTP. The three 100-horsepower pumps will pump wet weather flows in excess of 3.0 MGD towards the north to the CWWTP. An isolation valve will be installed on the discharge header between the pumps in order to direct the flow properly. A new 450 kW generator is required for the new loads, and new pump controls will be required. At the design firm capacity, the velocity in the existing 18-inch forcemain would exceed 11 ft/sec (for 13 MGD going north to CWWTP). A new 30" force main will be provided to carry the flow (from the lift station to the CWWTP).The station piping (the header) will be 24 inches in diameter.

As mentioned in Alternative 1, if the existing 18-inch force main is to remain in service, and used as a standby in conjunction with the proposed 30-inch line, several improvements would be necessary. The existing 18" Shawnee force main is suspected to have internal corrosion especially at the location of air relief valves. As a minimum, all high points in the force main should be replaced with non-metallic piping. A complete condition assessment of the forcemain and the lift station are beyond the scope of this study. Costs for the rehabilitation of this line are not included in this memorandum.

4.2.2.4 Limestone Lift Station

The Limestone lift station will be modified to lift flow from the Shawnee and Rice Creek basins (sub-basins S01 – S15) to the new south treatment plant. The existing pumps are sufficient to fill the existing 6 MG FEB. However, two new pumps will be required to pump process flor to the new treatment plant and to pump excess flow to the new FEB. These flows are 7.5 MGD and 4.3 MGD respectively, for a total firm capacity for the new process pumps of 11.8 MGD (8,200 gpm). The existing Limestone station was constructed with space and piping for the addition of these pumps. Each pump will have a capacity of 5,350 gpm (7.67 MGD). Each pump will be 100-horsepower. Each pump will be equipped with a VFD. A second 200 kW standby power generator will be added. A proposed 24" force main will connect the Limestone lift station to the new south treatment plant. A flow diversion structure (splitter box) will be installed on the proposed 24" FM to the new south plant that will allow excessive flows under storm events to be diverted to the new FEB via a new 24" gravity line. A small pump station located at the new FEB will return the contents of the FEB to the new treatment plant once high flows have subsided.

The City has since acquired property for the new plant at a location not shown in the 2010 Facility Report. The new proposed location of the south treatment plant can be seen in Figure TM3.16 below. Figure TM3.17 shows the proposed alignment for Alternative 2.

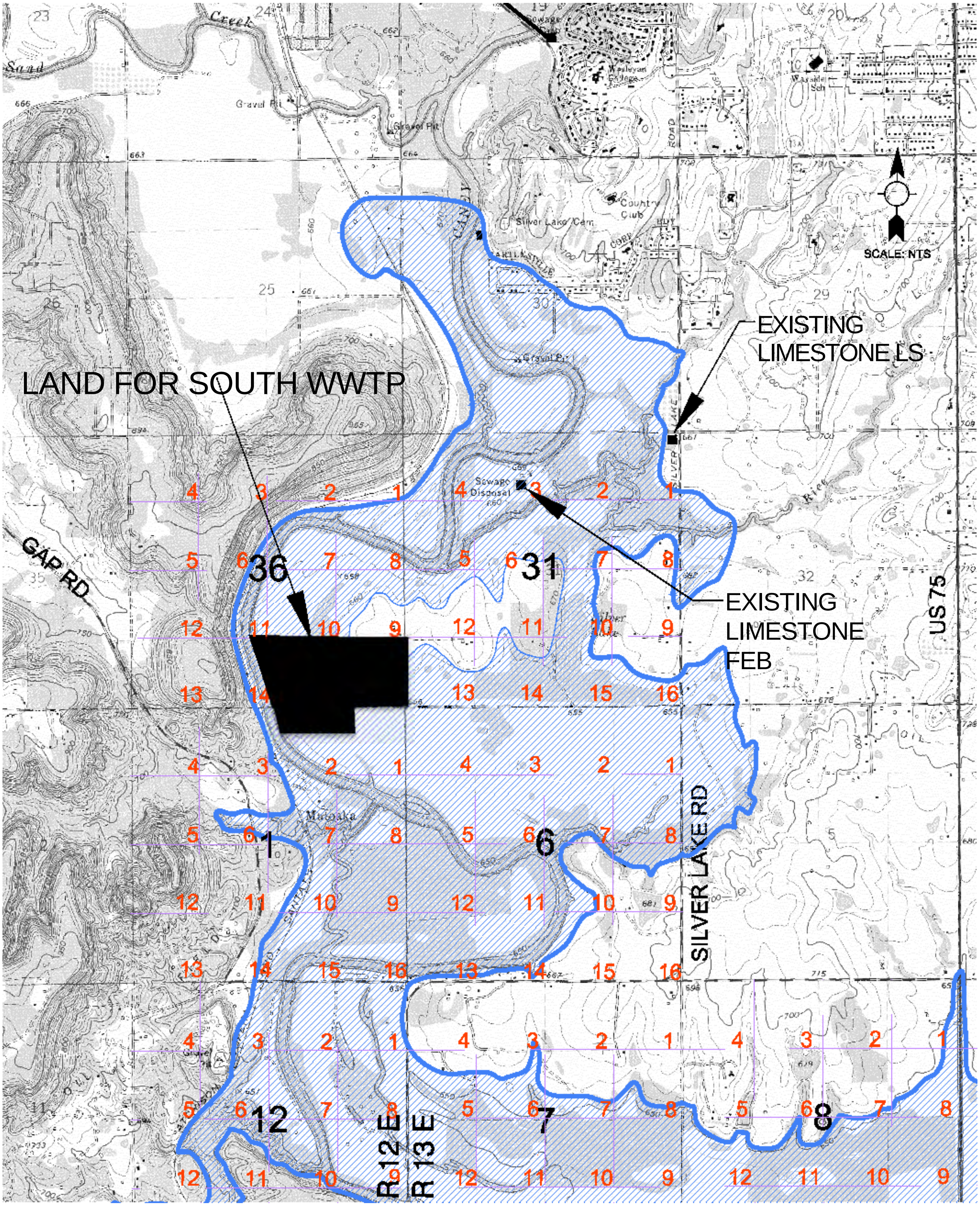


Figure TM3.16 – New South WWTP Location

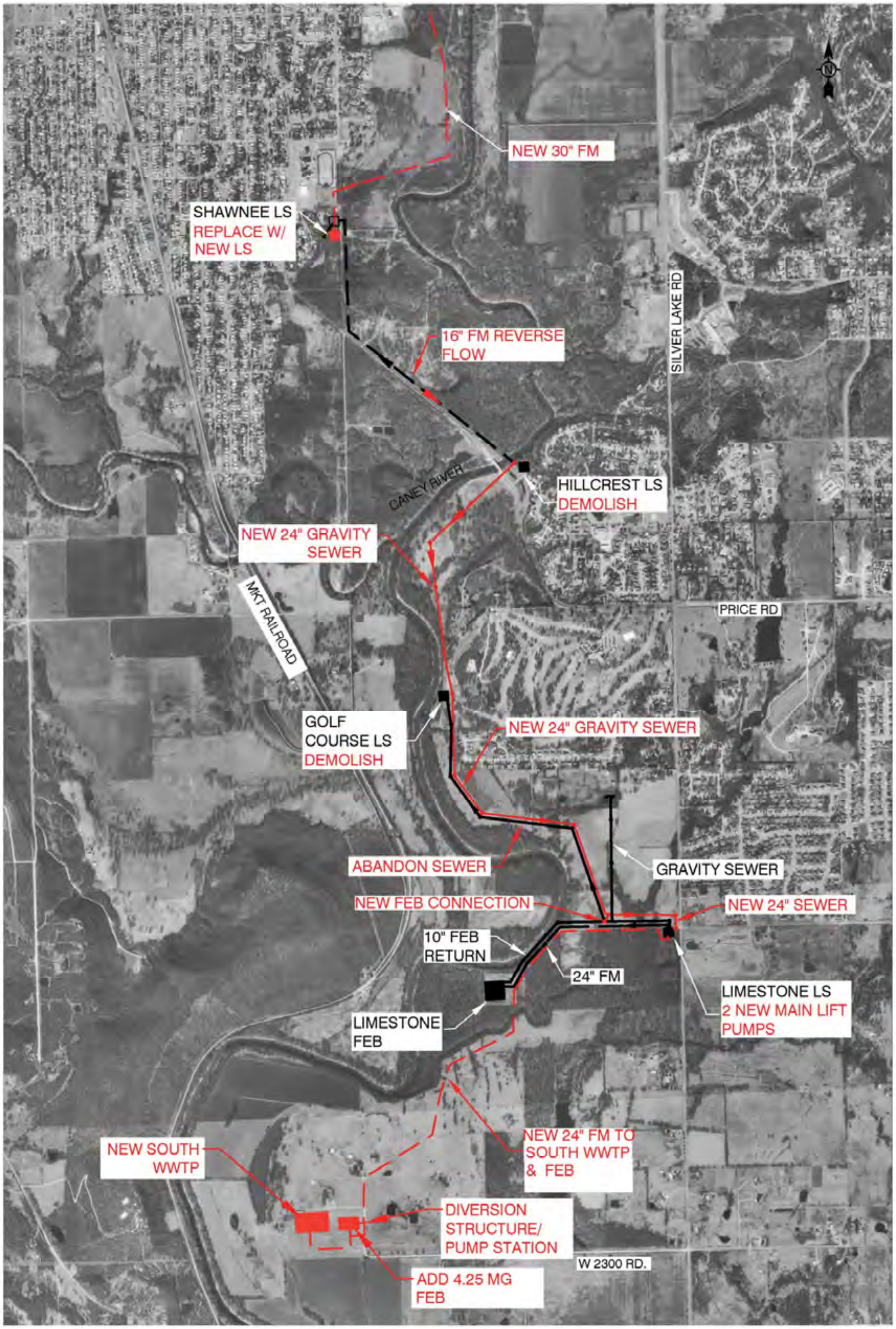


Figure TM3.17 Alternative 2 – New South WWTP and Pipeline Alignment from Limestone LS

4.2.3 FEBs Improvements

Alternative 2 will require the following FEB improvements:

4.2.3.1 Limestone FEB

In order to evaluate storage capacity in the south WWTP service area, the model calibration from the 2012 analysis was loaded with two extreme wet weather months, and hydrographs were created. The wet weather months of May 2008 and May 2015 were selected because they contained the worst 7-day event in 10 years as specified by DEQ requirements. These events created the largest required storage volume in the south service area. Flows were adjusted to account for 2050 population projections based on Table TM1.4 of the TM1 Report - "Population, Flow, and Waste Load." The inflow component was extracted from the wet weather month hydrograph. A reduction of 20% was removed from the inflow component to reflect I&I reductions and 5% was added back for new construction to be consistent with the modeled flows as shown in Table TM3.7 of this report.

The existing 16" FM between Hillcrest and Shawnee lift stations that is proposed to be reused under this alternative will limit the amount of flow that can be sent south from the Shawnee lift station. As a result, storage requirements in the south service area were determined assuming that only 3 MGD of peak flow from the Shawnee North basin area (from Shawnee LS) could be pumped towards the south. The remaining flow (13 MGD peak) would be sent north to the existing CWWTP.

As demonstrated by the graph below, Figure TM3.19 shows that approximately 10.1 MG of storage is required (assuming I&I abatement is performed) in the south WWTP service area. Because the existing Limestone FEB currently has 6 MG of storage available, this equates to an additional 4.1 MG of increased storage that is needed. Limestone FEB is located in a floodway and is not suitable for expansion. Therefore, it is assumed that a new FEB will be constructed at the new south treatment facility. The new FEB will be designed at 4.25 MG.

The new pumps in the Limestone Lift Station will pipe directly to the new wastewater treatment plant where excess flows will be diverted to the new FEB by means of a diversion structure and 30-inch gravity line. A 10-horsepower pump station (2 pumps, 500 gpm each) will be constructed at the new FEB to return the contents of the FEB to the new south treatment plant via a new 8-inch force main once storm flows have subsided. Average and peak process flows will bypass the new FEB and flow directly to the new south WWTP via a 30-inch gravity line. The new FEB will have a concrete-lined pre-sedimentation basin. Figure TM3.18 below shows the schematic of the Limestone LS, diversion structure, new pump station, and the two FEBs with piping to the new WWTP.

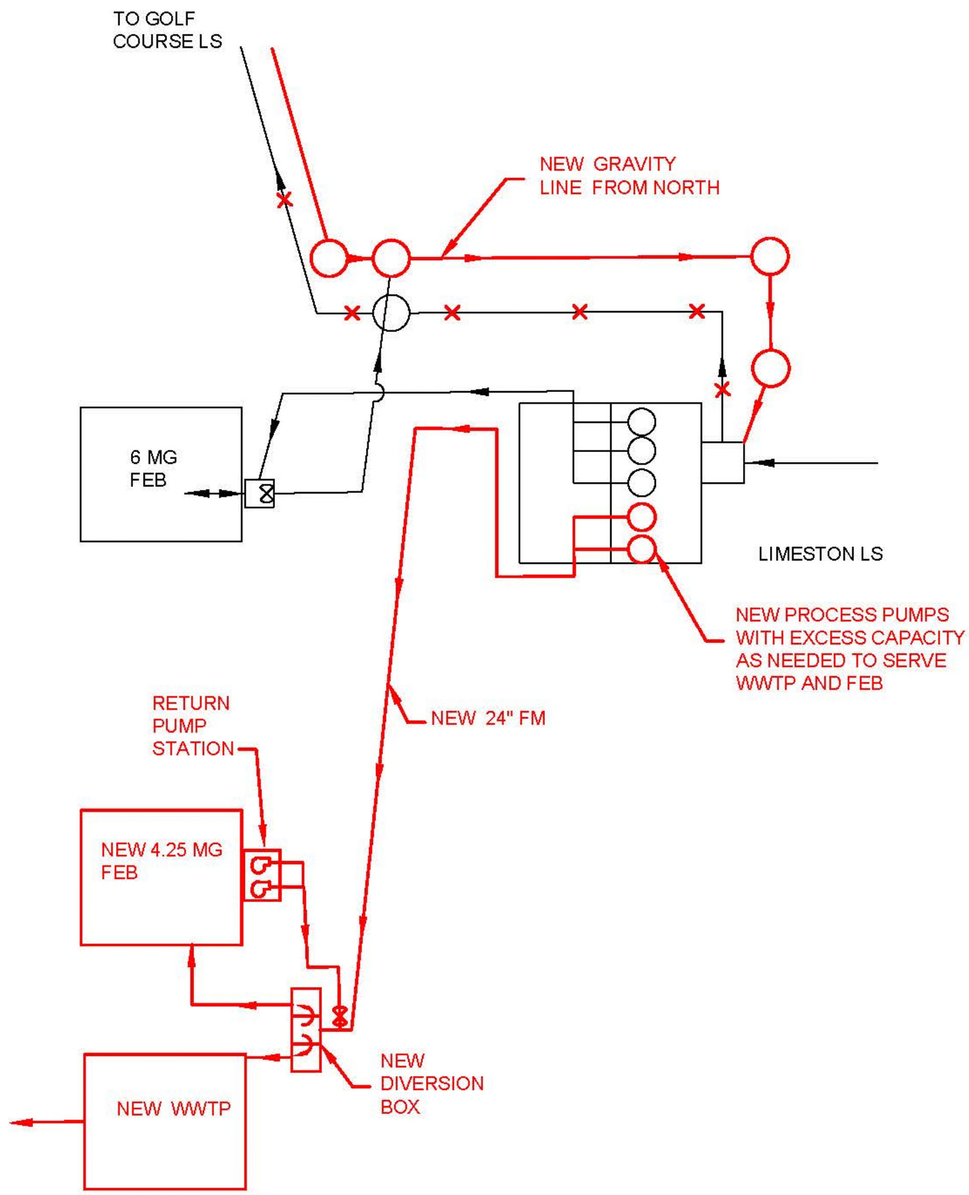


Figure TM3.18—Alternative 2 Limestone LS, FEB and Piping to New WWTP Schematic

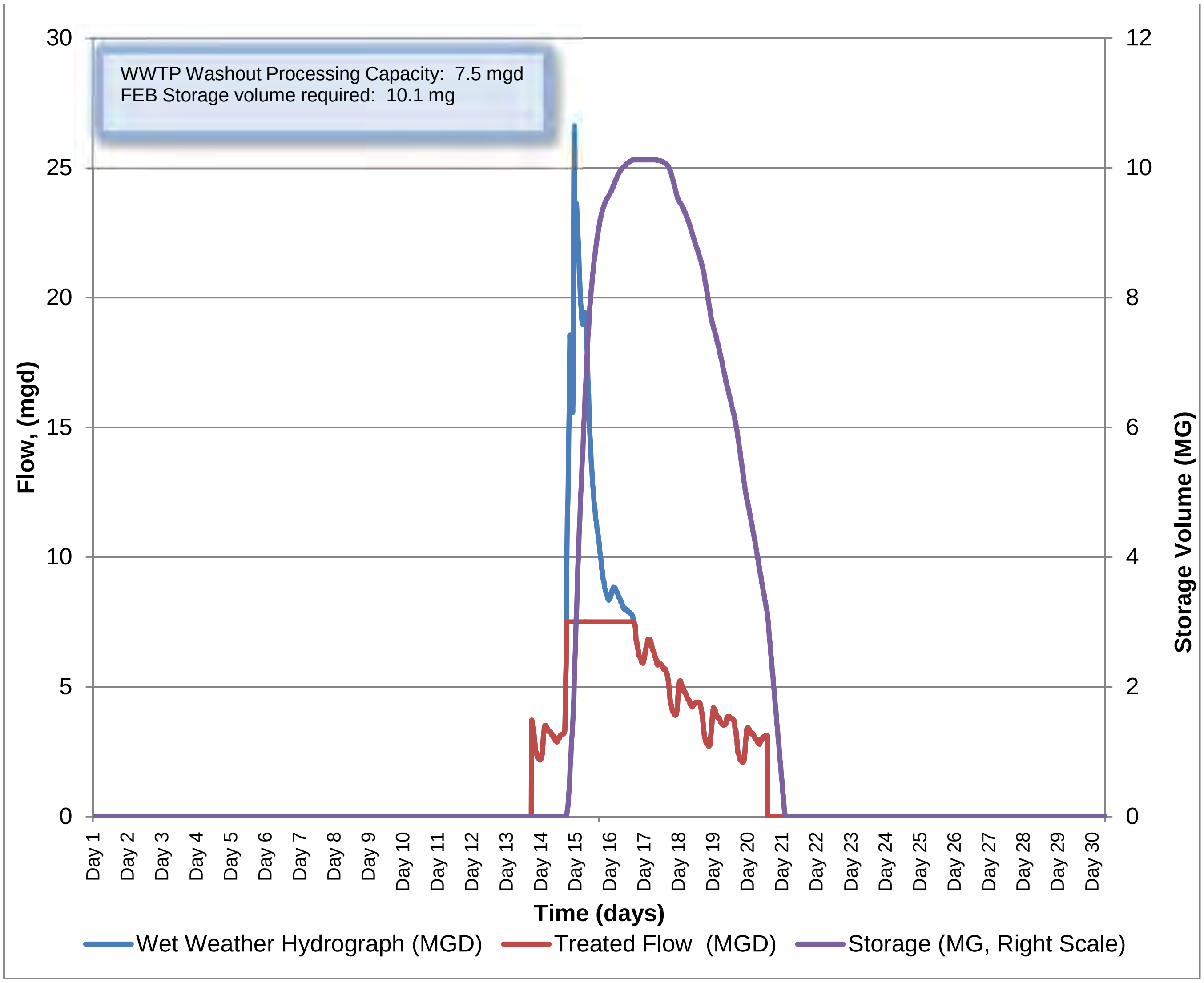


Figure TM3.19 Alternative 2- Limestone FEB Volume Need Using May 2008 Wet Weather Month

To size the required storage volume for the south WWTP service area, two storm events were used from May 2008 and May 2015. A hydrograph was created for each event using the same steps as discussed above under Alternative 1. These events were selected because they produced the largest peak flows recorded in the last 10 years.

After further evaluation, it was determined that even though the May 2015 event had a larger peak compared to that of the May 2008 event, the required storage volume was actually less (9.5 MG) due to the fact that the event had a smaller peak storm duration and created less volume that needed to be stored. Because of this, the May 2008 event was used to size the south service area FEB. Refer to Figures TM3.19 and TM3.20 for further information.

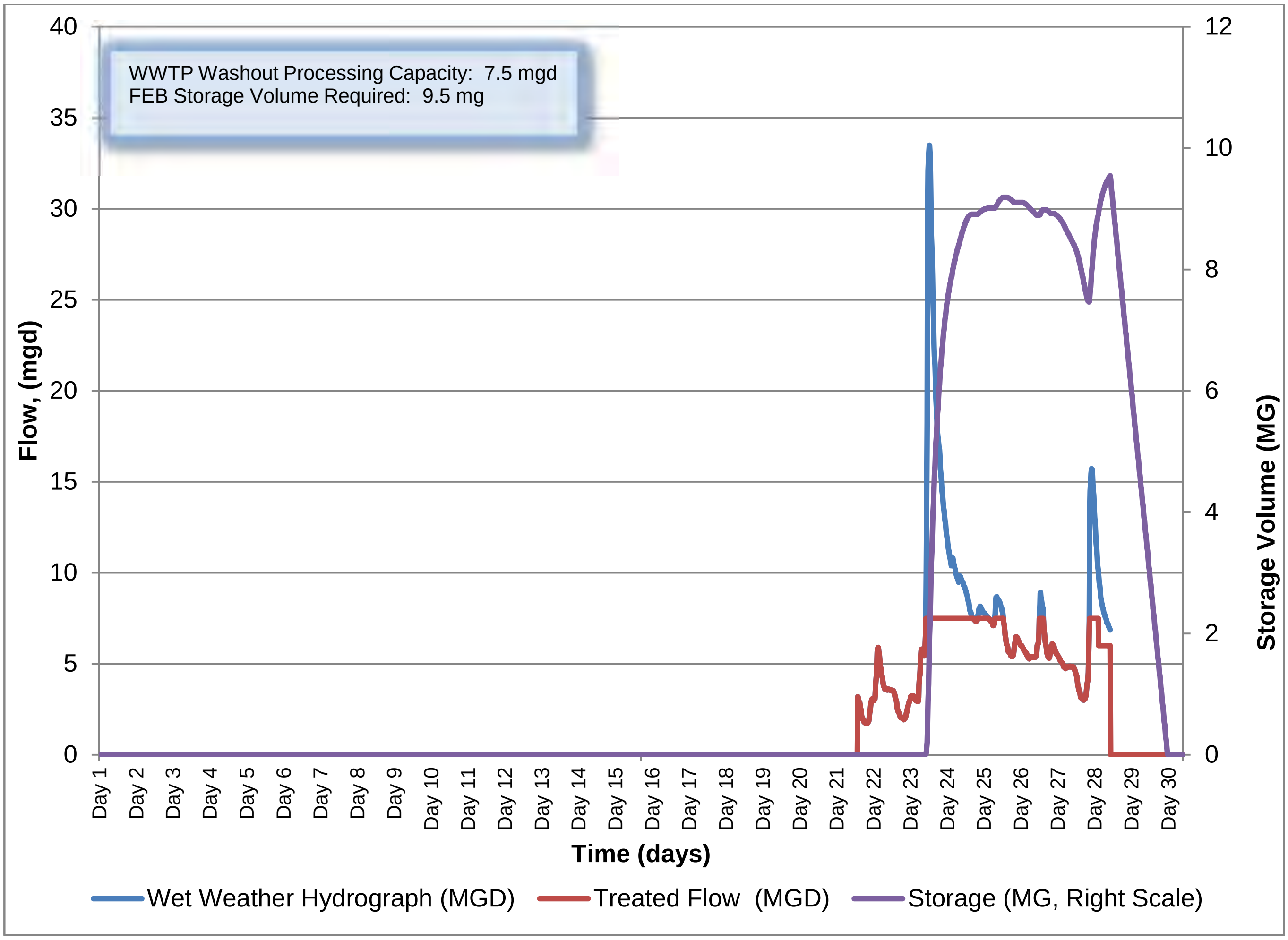


Figure TM3.20 – Alternative 2- FEB Volume Need at the New South WWTP Using May 2015 Wet Weather

4.2.3.2 Chickasaw FEB

As demonstrated by the “Update of the Collection System Analysis” completed in April 2006 and affirmed in the “Final Report of the Update of the Collection System Analysis” dated 2013, no capacity improvements to the Chickasaw FEB will be required under Alternative 2. As an additional check, the same 2008 wet weather event (and 2012 calibrated model) that was used to evaluate the Limestone FEB storage capacity, was also applied to the north service area (refer to Figure TM3.21 below) in order to confirm there was adequate storage capacity at the existing Chickasaw FEB. After further evaluation, the conclusions were the same. No capacity improvements to the Chickasaw FEB are required for this alternative.

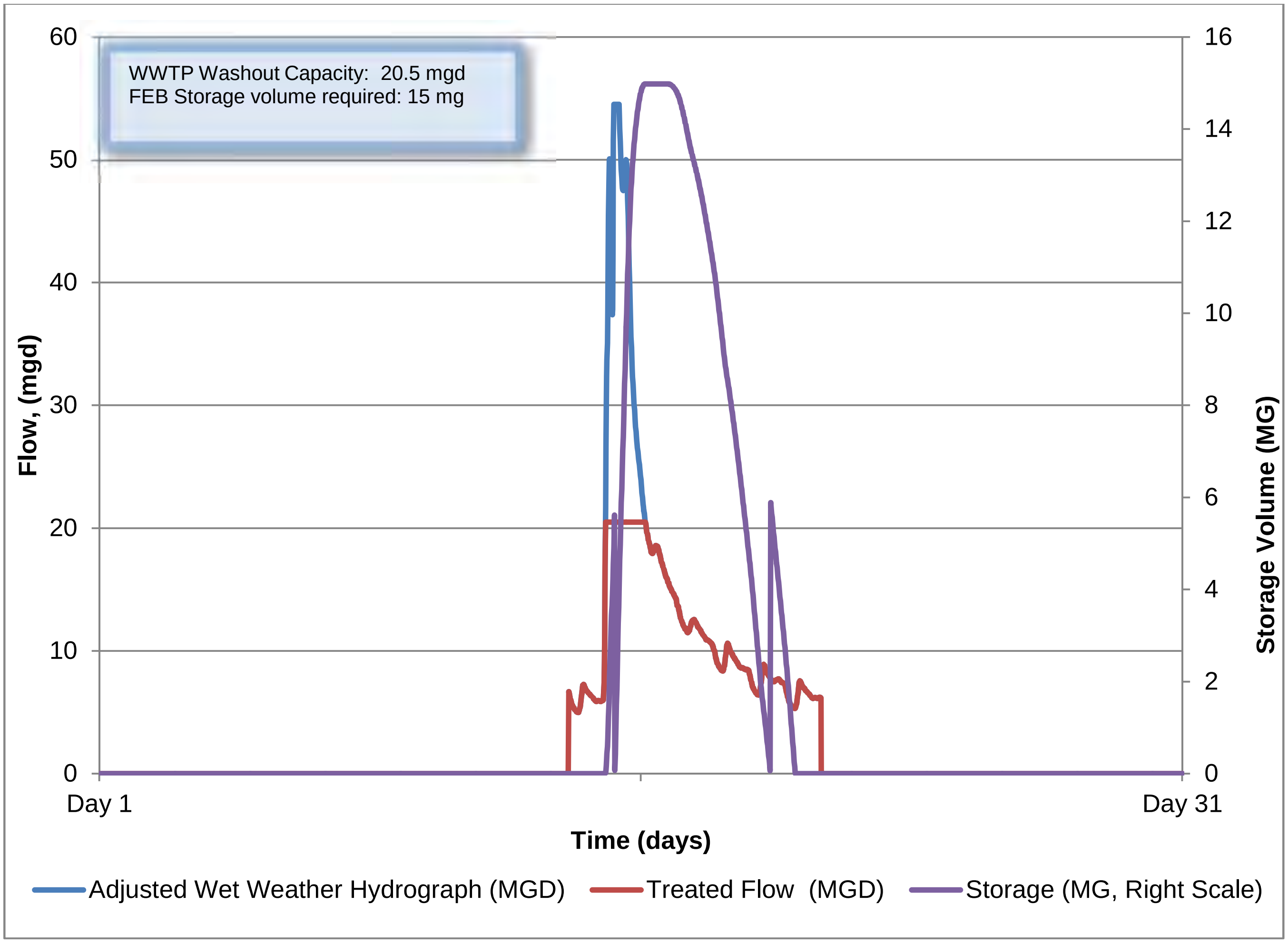


Figure TM3.21 - May 2008 Wet Weather Event for Chickasaw FEB Storage Design

Table TM3.8 Summary of Improvements—Alternative 1 (All Flows to Existing CWWTP)

Facility	Lift Station	Force Main	Gravity Main	Flow Equalization Basin	Estimated Cost
Shawnee LS	New Lift Station 4 - 125 HP Pumps with VFDs Isolation Valve 600 kW Generator SCADA for New LS 30" Station Piping	New 36" Force Main	24" Gravity Collection to LS		$ 9,627,700
Golf Course LS	3 - 50 HP Pumps with VFDs 250 kW Generator 12" Station Piping				$ 780,800
Hillcrest LS	3 - 25 HP Pumps with VFDs 4,000 Gal Wet Well Expansion 120 kW Generator 12" Station Piping	16" Parallel Force Main			$ 1,293,400
Limestone LS	2 - 100 HP Pumps with VFDs 200 kW Generator	New 24" Force Main and Return Line to New FEB w/ control valves and vaults			$ 2,506,000
New South FEB				New 10 MG FEB	$ 1,226,800
Chickasaw FEB*				New Synthetic Liner*	
Total					$ 15,434,700

*Already completed by earlier project.

Table TM3.9 Summary of Improvements—Alternative 2 (Construct New South Plant)

Facility	Lift Station	Force Main	Gravity Main	Flow Equalization Basin	Estimated Cost
Gravity Interceptor			New 24" Line from Hillcrest LS to Limestone LS		$ 3,896,000
Shawnee LS	New Lift Station 3 - 100 HP Pumps with VFDs 2 - 25 HP Pumps Isolation Valve 450 kW Generator SCADA for New LS 24" Station Piping	New 30" Force Main	24" Gravity Collection to LS		$ 8,225,000
Golf Course LS	Demolition	Abandon 14" Force Main			$ 26,000
Hillcrest LS	Demolition	Reverse flow of 16" Force Main			$ 26,000
Limestone LS	2 - 100 HP Pumps with VFDs 200 kW Generator	New 24" Force Main			$ 3,131,000
South WWTP FEB	New Lift Station 2 - 10 HP Pumps Splitter Box/Diversion Structure	8" Force Main from New FEB to South WWTP	30" Gravity Line from Diversion Structure to New FEB/ WWTP	New 4.25 MG FEB with Concrete Lined Pre-Sedimentation Basin	$ 1,398,000
Chickasaw FEB				New Synthetic Liner*	
Total					$ 16,702,000

*Already completed by earlier project.