DRAFT

EL DORADO LAKE WESTERN WATER SUPPLY STUDY

B&V PROJECT NO. 174532

PREPARED FOR

City of El Dorado, Kansas

28 MAY 2013

Table of Contents

LIST OF TABLES

Executive Summary

The City of Eldorado, in partnership with the United States Army Corps of Engineers (USACE), constructed El Dorado Lake for the purposes of flood control and Municipal and Industrial (M&I) water storage. The lake has provided and will remain a reliable source of supply for the City and its customers. In addition to supplying the City's current and future customers, excess yield is available in the lake that could be utilized for neighboring communities, including the City of Wichita, which would help relieve the strain on available water supply throughout the region. This study evaluated the feasibility of supplying finished water to Wichita. The major components of the study included:

- Lake yield analysis at various return intervals to identify excess yield available
- Review of the current status of Wichita's Aquifer Storage and Recovery System
- Water Treatment Plant Process Design
- Delivery Options
- Capital Cost Development

Lake Yield Analysis

A number of model simulations were performed to evaluate the potential yield for existing and future conditions at various draw downs in the lake. Although no lake storage is specifically set aside for recreation, fish, or wildlife purposes, reasonable limits of drawdown were chosen for drought years for which could be delivered to future customers. All model conditions considered continuous withdrawal for current customers, while water deliveries to potential future customers are shut off if the lake drops to below the minimum allowable elevation.

After review of various withdrawal rates, a maximum lake withdrawal rate of 30 mgd for treatment and delivery to Wichita was selected as the initial basis of this evaluation. Based on current lake conditions, 57 year historical rainfall records, including the drought years of the 1950s, with a maximum lake drawdown of 5 ft below top of normal pool and existing customer demand of 9 mgd, 30 mgd is available to be withdrawn from the lake approximately 78 percent of the time. This results in an average annual supply of 23.4 mgd available for future customers.

Wichita may desire to have treated water from El Dorado Lake provided during peak demands to reduce or defer the need for additional treatment capacity. Therefore, the City should consider potential operational schemes as part of the negotiations that may lower the lake level below the five foot drawdown during peak demands, or varying the withdrawal rate to optimize the available of supply to Wichita.

Wichita's Aquifer Storage and Recovery Plan

The City of Wichita has embarked on a multi-phased aquifer storage and recovery (ASR) project to replenish water supply for the city to meet future demands. The concept of the ASR project is to divert flow from the Little Arkansas River, treat to drinking water standards, and inject into the aquifer to recover groundwater levels that have declined over time due to well production exceeding natural recharge.

This analysis of ASR program was limited only to review of historical stream gage data and information included in the ASR Program Review Water Supply Plan, prepared by HDR Engineering, May 2010. Discussion with the City of Wichita on current operation, updates to the yield predictions, and changes since the report are not included in this evaluation. Based on the information available, it appears Phase II facilities could operate a full capacity approximately 33% of the time with annual raw water supply injected into the aquifer of about 11.2 mgd. If Phase III were implemented, an additional 6.9 mgd of flow would be diverted to the aquifer for a total recharge supply of about 18.1 mgd. These values reflect the potential available yield if every drop of available water was captured and sent to the aquifer. These flows do not account for effectiveness of the system to capture all the available supply. Also, these values do not relate directly to the available of water supply to the City as they do not account for losses within the aquifer or water loss associated with finished water treatment before entering the distribution system.

The primary goal of the ASR program is to recharge the aquifer to provide sufficient water storage to allow Wichita to continue to meet future demands during extreme drought conditions. If during non-drought periods the water from El Dorado is used to serve the customers while the majority of flow from the ASR is only used to replenish the aquifer, the goals of the ASR could still be achieved, much quicker and more reliably.

Treatment

The City of El Dorado new water treatment plant would have a capacity of 30 mgd. Raw water would supply the treatment plant from the two existing 36-inch pipes exiting the dam outfall wetwell. The location of the new treatment plant should be downstream of the dam to allow for gravity flow from the lake to the treatment plant. The treatment process would consist of conventional coagulation-flocculation-sedimentation, followed by membrane filtration and chloramination before delivery to Wichita. The estimated construction cost, in 2012 dollars, for the 30 mgd treatment facility is $62.1 million dollars.

Delivery

Finished water would be delivered via a 48-inch pipeline to the Webb Road Pumping Station and Reservoir located in the northeast portion of Wichita's distribution system. Several routes appear to be viable for the transmission main. A detailed alignment study should be conducted to confirm the route of the new pipeline. The estimated cost for the finished water pipeline is $59.5 million dollars.

Schedule

The overall schedule is dependent on response from Wichita and other stakeholders as to the viability of this supply option and how it fits into the overall regional supply needs. The entire project could be complete and ready to deliver finished water to Wichita by 2015.

1.0 Introduction

The City of El Dorado is investigating the feasibility of supplying finish water to potential regional customers including the City of Wichita. Water would be obtained from excess yield available in El Dorado Lake. Currently, the average annual water demand to serve existing customers from the lake is approximately 9 million gallons per day (mgd). Based on a yield analysis of El Dorado Lake conducted by Black & Veatch, 30 mgd is available for the new treatment plant 78 percent of time, in addition to 9 mgd for existing customer's average annual demands which is available 100 percent of the time.

Utilizing the excess yield by selling to wholesale customers will create a new revenue stream that can be used to reinvest in the lake, community, and the community's infrastructure. Selling wholesale water to Wichita can also potentially supply a cost-effective way for Wichita to have an additional, reliable water source. This study provides the framework for facilities required to deliver finished water to the City of Wichita. Additionally, customers along the route will be identified and considered should this project move forward.

1.1 BACKGROUND

The City, in partnership with the United States Army Corps of Engineers (USACE), constructed El Dorado Lake for the purpose of flood control and Municipal and Industrial (M&I) water storage. The lake was completed in 1981 and is operated by USACE with all M&I water storage owned by the City.

The City is currently paying for half of the lake's storage and accumulating the funds necessary to pay for the remaining storage in a sinking fund.

1.2 PURPOSE AND SCOPE

The City retained Black & Veatch to update and confirm the potential yield of the lake for various return intervals. After establishment of the lake yield, the scope included review of raw and finish water quality to address compatibility of the lake water with Wichita's distribution. The scope also included development of preliminary sizing of treatment components, identification of potential transmission main alignment, and pumping requirements. The estimated capital and operation and maintenance (O&M) costs were developed for components required to deliver finished water in order to establish a business plan.

1.3 SUMMARY OF FINDINGS

The initial findings of the water supply study can be briefly summarized as follows:

- Current El Dorado average annual demands for existing customers are 9 mgd with a peak of 10 mgd.
- Future El Dorado average annual demands for existing customers are expected to be 13.4 mgd by year 2050.
- It is anticipated that the new treatment plant for delivering water to new future customers will operate at 30 mgd when the lake elevation is higher than five (5) feet below normal pool, elevation 1334. When the lake level drops below elevation 1334, a conservation rate would be implemented reducing withdrawls such that the remaining lake yield will be sufficient to meet current El Dorado customer demands of 9 mgd even through drought years.
- Using 57 years of historical record and the plant operation described above, El Dorado Lake will be able to provide 30 mgd in addition to the City's current average demand of 9 mgd 78 percent of the time, and continue to provide the City's current demands of 9 mgd the remainder of the time.
- The 57-year historical record included the 1950s drought-of-record for Kansas. For climate conditions similar to the time period since 1994, El Dorado Lake will be able to provide 30 mgd in addition to the City's current average demand of 9 mgd 88.5 percent of the time, and continue to provide the City's current average demand of 9 mgd the remainder of the time.
- Wichita has two primary sources of drinking water, the Cheney Reservoir constructed in 1965 by the US Bureau of Reclamation and the Equus Beds Aquifer wells.
- Wichita's Aquifer Storage Recovery (ASR) program was initiated primarily by the City of Wichita to divert flow from the Little Arkansas River, treat, and then inject into an aquifer to recharge the supply. The total capacity envisioned for the ASR program is 100 mgd.
- Water rights require a minimum flow in the Little Arkansas River of 30 cubic feet per second(cfs). Configuration of the river intake requires a minimum of 50 cfs in addition to the flow being diverted to the aquifer before withdrawals can be made. Therefore, to utilize the full capacity of Phase II treatment components, the minimum river flow must be 105 cfs.
- Phase III requires a minimum stream flow of 160 cfs to be operable.
- Phase IV consists of bank infiltration wells.
- Based on historical records, the total average annual flow injected into the aquifer for both Phase II and Phase III combined will be about 18.1 mgd when available.

- Not selling water in the future will result in accumulation of interest to the remaining debt on the lake, until the storage is activated with the Corps and annual payments begin.
- A new revenue stream created from selling the excess yield can be used support payments to the USACE and to reinvest in the lake, the community, and the community's infrastructure.
- There is excess water supply storage in El Dorado Lake. Conceptually, integrating three sources of supply for Wichita (Cheney Reservoir, Equus Beds, and El Dorado Lake) will provide more operational flexibility and will optimize the use of available supplies in the region. El Dorado Lake water could be utilized in lieu of costly infrastructure for implementing ASR Phase III and the Northwest Treatment Facility. Water from El Dorado Lake would be used to supplement the distribution system and allow reduced withdrawals out of the Equus Bed Aquifer, allowing for more rapid filling of the aquifer at a lower overall water resource cost. An integrated plan with both utilities working together would enable the systems to meet maximum demands without detriments to the long term yield.
- A finished water supply system provides redundancy for the City of El Dorado.

2.0 Lake Yield Analysis

This section describes the preliminary findings for the frequency and amount of water supply that can be obtained from El Dorado Lake for a variety of lake and climate conditions.

Although no lake storage is specifically set aside for recreation, fish, or wildlife purposes, there would likely be resistance to the idea of drawing the lake down to very low levels every year for water supply, specifically in non-drought years. For this evaluation, reasonable limits are chosen for the allowable annual lake drawdown for which water can be delivered to potential future customers. Normal lake operating elevation is 1339 feet. Computer model simulations were performed for El Dorado Lake to determine the percentage of months that the lake level remains above elevations 1338, 1337, 1336, 1335, and 1334 feet (less than 1, 2, 3, 4, and 5 feet of drawdown below normal pool) at various water supply withdrawal rates for current and potential future customers when considering the historical climate record. The model was set up such that withdrawals are made from the lake continuously for current customers, while water deliveries to potential future customers are modified if the lake level drops below the minimum allowable elevation.

A number of model simulations were performed, and the results are provided in the Table 2-2 for current conditions and Table 2-3 for future conditions. Each row of the tables represents an individual model simulation, and can be described with the following example shown in Table 2-1. A description of this example is as follows:

- The model simulation provided in Table 2-1 considers existing lake volume and current average day water demands for the City's existing customers.
- Column #1 indicates that deliveries to potential future customers are made if the lake is less than 5 feet below normal pool.
- Column #2 shows today's average day water demands for the current customers is approximated at 9 MGD. The water demands of existing customers are met continuously in all model simulations.
- Column #3 shows that the model delivers 30 mgd to potential future customers during times that the lake is less than 5 feet below normal pool. If the lake drops more than 5 feet below normal pool, the deliveries to potential future customers is set to zero (0) mgd in the model. Deliveries below 5 feet are anticipated at a conservation rate, but are not credited in the calculations towards overall yield.

- Column #4 shows the average long-term delivery to future customers is 23.4 MGD, when considering months that the deliveries are made at 30 mgd and months that deliveries are discontinued.
- Column #5 shows that the lake provides 30 mgd to future customers 78% of the time.
- Columns #6 shows the longest period where a continuous flow of 30 mgd is delivered to future customers is 66 months, which will occur during the wettest climate conditions expected.
- Column #7 shows the longest continuous period where no flow will be delivered to future custormers is 54 months, which will occur during the driest climate conditions expected.
- Column #8 shows that the average long-term lake drawdown below normal pool is 2.9 feet.

Table 2-1 - Example of One of the Model Simulations for El Dorado Lake

ALLOWABLE LAKE DRAWDOWN (FEET) (NO WATER IS DELIVERED TO NEW FUTURE CUSTOMERS IF THE LAKE IS AT OR BELOW THIS ELEV.)	*AVERAGE DAY RAW WATER SUPPLY FOR CURRENT CUSTOMERS (MGD)*	*WATER SUPPLY DELIVERED TO FUTURE CUSTOMERS IF LAKE IS ABOVE MINIMUM ALLOWABLE ELEVATION (MGD)*	*EFFECTIVE LONG-TERM AVERAGE DELIVERY TO FUTURE CUSTOMERS (MGD) (AVERAGING MONTHS WITH NO DELIVERIES)*	*PERCENT OF MONTHS THE LAKE WILL PROVIDE WATER TO FUTURE CUSTOMERS AT THIS ALLOWABLE LAKE DRAWDOWN*	*MAXIMUM NUMBER OF CONSECUTIVE MONTHS THAT THIS AMOUNT OF WATER IS DELIVERED TO FUTURE CUSTOMERS*	*MAXIMUM NUMBER OF CONSECUTIVE MONTHS THAT NO WATER IS DELIVERED TO FUTURE CUSTOMERS*	*AVERAGE LONG-TERM MONTHLY LAKE DRAWDOWN FROM NORMAL POOL EL 1339 (FEET)*
5 (El. 1334’)	9	30	23.4	78%	66	54	2.9

Table 2-2 provides the results for today’s lake volume conditions based on a recent lake survey performed by the Kansas Biological Survey (KBS) and for current average day water demands for the City based on 57 year historical record of precipitation.

Table 2-3 includes an evaluation of the effect on the lake’s water supply if the USACE would modify their control plan and hold back more water in the bottom six (6) inches of the flood control pool for water supply withdrawals to be made. These simulations show a slight increase of several percent in the frequency and long-term average amount of water delivered to potential future customers. Discussions with the USACE are required to determine the feasibility of reallocating and utilizing the bottom of the flood pool for water supply.

Table 2-2 – Frequency of Obtaining Various Water Supply Quantities with Allowable Drawdowns for Current Conditions with Top of Usable Water Supply Pool Set at 1339 feet [1]

ALLOWABLE LAKE DRAWDOWN (FEET) (NO WATER IS DELIVERED TO NEW FUTURE CUSTOMERS IF THE LAKE IS AT OR BELOW THIS ELEV.)	AVERAGE DAY RAW WATER SUPPLY FOR CURRENT CUSTOMERS (MGD)	WATER SUPPLY DELIVERED TO FUTURE CUSTOMERS IF LAKE IS ABOVE MINIMUM ALLOWABLE ELEVATION (MGD)	EFFECTIVE LONG-TERM AVERAGE DELIVERY TO FUTURE CUSTOMERS (MGD) (AVERAGING MONTHS WITH NO DELIVERIES)	PERCENT OF MONTHS THE LAKE WILL PROVIDE WATER TO FUTURE CUSTOMERS AT THIS ALLOWABLE LAKE DRAWDOWN	MAXIMUM NUMBER OF CONSECUTIVE MONTHS THAT THIS AMOUNT OF WATER IS DELIVERED TO FUTURE CUSTOMERS	MAXIMUM NUMBER OF CONSECUTIVE MONTHS THAT NO WATER IS DELIVERED TO FUTURE CUSTOMERS	AVERAGE LONG-TERM MONTHLY LAKE DRAWDOWN FROM NORMAL POOL EL 1339 (FEET)
1 (El. 1338')	9	20	11.0	55%	36	56	1.7
1 (El. 1338')	9	30	15.3	51%	34	57	1.8
1 (El. 1338')	9	40	19.2	48%	22	57	1.9
1 (El. 1338')	9	50	22.2	44%	19	57	2.0
2 (El. 1337')	9	20	13.2	66%	48	55	1.9
2 (El. 1337')	9	30	18.4	61%	47	56	2.1
2 (El. 1337')	9	40	22.6	57%	43	56	2.3
2 (El. 1337')	9	50	26.6	53%	35	56	2.4
3 (El. 1336')	9	20	14.8	74%	65	54	2.1
3 (El. 1336')	9	30	20.4	68%	49	55	2.4
3 (El. 1336')	9	40	25.3	63%	47	55	2.7
3 (El. 1336')	9	50	29.3	59%	44	56	2.8
4 (El. 1335')	9	20	16.0	80%	104	54	2.2
4 (El. 1335')	9	30	22.1	74%	65	54	2.6
4 (El. 1335')	9	40	27.2	68%	50	55	2.9
4 (El. 1335')	9	50	31.6	63%	46	55	3.3
5 (El. 1334')	9	20	16.8	84%	117	54	2.4
5 (El. 1334')	9	30	23.4	78%	66	54	2.9
5 (El. 1334')	9	40	28.9	72%	62	55	3.3
5 (El. 1334')	9	50	33.4	67%	55	55	3.7

Table 2-3 Frequency of Obtaining Various Water Supply Quantities with Allowable Drawdowns for2050 Lake Conditions with Top of Usable Water Supply Pool Set at 1339.5 feet (six inches into the Flood Control Pool) [1]

ALLOWABLE LAKE DRAWDOWN (FEET) (NO WATER IS DELIVERED TO NEW FUTURE CUSTOMERS IF THE LAKE IS AT OR BELOW THIS ELEV.)	AVERAGE DAY RAW WATER SUPPLY FOR CURRENT CUSTOMERS (MGD)	WATER SUPPLY DELIVERED TO FUTURE CUSTOMERS IF LAKE IS ABOVE MINIMUM ALLOWABLE ELEVATION (MGD)	EFFECTIVE LONG-TERM AVERAGE DELIVERY TO FUTURE CUSTOMERS (MGD) (AVERAGING MONTHS WITH NO DELIVERIES)	PERCENT OF MONTHS THE LAKE WILL PROVIDE WATER TO FUTURE CUSTOMERS AT THIS ALLOWABLE LAKE DRAWDOWN	MAXIMUM NUMBER OF CONSECUTIVE MONTHS THAT THIS AMOUNT OF WATER IS DELIVERED TO FUTURE CUSTOMERS	MAXIMUM NUMBER OF CONSECUTIVE MONTHS THAT NO WATER IS DELIVERED TO FUTURE CUSTOMERS	AVERAGE LONG-TERM MONTHLY LAKE DRAWDOWN FROM NORMAL POOL EL 1339 (FEET)
1 (El. 1338’)	13.4	20	11.9	60%	47	56	1.5
1 (El. 1338’)	13.4	30	16.8	56%	45	56	1.7
1 (El. 1338’)	13.4	40	20.5	51%	34	57	1.9
1 (El. 1338’)	13.4	50	23.6	47%	22	57	2.0
2 (El. 1337’)	13.4	20	13.7	68%	49	55	1.8
2 (El. 1337’)	13.4	30	18.7	62%	47	55	2.0
2 (El. 1337’)	13.4	40	23.1	58%	44	56	2.3
2 (El. 1337’)	13.4	50	26.7	53%	39	56	2.5
3 (El. 1336’)	13.4	20	14.9	74%	65	54	1.9
3 (El. 1336’)	13.4	30	20.3	68%	50	55	2.3
3 (El. 1336’)	13.4	40	25.2	63%	46	55	2.7
3 (El. 1336’)	13.4	50	29.2	58%	45	56	2.9
4 (El. 1335’)	13.4	20	16.0	80%	104	54	2.1
4 (El. 1335’)	13.4	30	21.9	73%	62	54	2.6
4 (El. 1335’)	13.4	40	26.7	67%	50	55	3.0
4 (El. 1335’)	13.4	50	31.2	62%	46	55	3.4
5 (El. 1334’)	13.4	20	16.7	83%	116	54	2.3
5 (El. 1334’)	13.4	30	23.0	77%	63	54	2.9
5 (El. 1334’)	13.4	40	28.1	70%	59	55	3.4
5 (El. 1334’)	13.4	50	32.6	65%	55	55	3.8

Conclusions

These simulations provide estimates for the frequency of achieving various lake withdrawal rates under various lake conditions when considering the historical period of record. Additional water supply simulations should be performed for specific climate conditions to determine the variability in the future water deliveries that will (1) assist the City in determining the most appropriate capacity for the facilities, (2) determine the amount of additional storage and water rights the City may need to acquire for the selected alternative, (3) provide a better understanding of operational considerations for the proposed project when considering the discontinuities in the deliveries depending on future potential customers demand fluctuations and changes in El Dorado Lake conditions, and (4) assist in future discussions with the USACE and potential customers. The following conditions were used for the basis of this evaluation:

- Current Lake Conditions estimated by draft results of 2010 Kansas Biological Survey (KBS) lake study.
- Existing customer average annual demands of 9 mgd.
- Maximum Lake Drawdown of 5 ft for delivering water to new customers (water deliveries between lake elevations 1334 -1339 ft).
- 30 mgd maximum lake withdrawal rate for new treatment plant and delivery to future customers.
- 57 year historical record for precipitation

Based on a yield analysis of El Dorado Lake using these conditions, 30 mgd is available for the new treatment plant 78 percent of time, in addition to 9 mgd for existing customer's average annual demands which is available 100 percent of the time. Assuming 30 mgd is available 78 percent of the time, while 0 mgd available 22 percent of the time, the average yearly raw water excess supply available is 23.4 mgd.

Wichita may also desire to have treated water from El Dorado Lake provided during peak demands to reduce or defer the need for additional treatment capacity , including the construction of the Northwest Treatment Facility. The City should consider potential operational schemes as part of the negotiations that may lower the lake level below the five foot drawdown during peak demands.

3.0 Raw versus Treated Water Supply

Two delivery options are available to serve Wichita with El Dorado Lake water. The water could either be treated by El Dorado near the lake source, resulting in finished water directly supplied to Wichita's distribution system, or raw water sent to Wichita for treatment and injection into the ASR aquifer. Both options were evaluated regarding feasible connection points, solids handling, system capacity, water quality, and regional water needs. The following summarizes the results of the evaluation.

Point of Connection

The 21st and Webb Road Pumping Station provides a viable location for connection of finished water supply to the distribution system. Although the existing pumping station does not have sufficient capacity to handle the full delivered quantity, the excess supply could be conveyed through the Northeast Transmission Main back to the Hess Pumping Station for redistribution to other locations. This approach would alleviate costly and disruptive improvements associated with extension of the supply main all the way back to Hess Pumping Station. Delivery of raw water would have to extend approximately 10 additional miles to the ASR treatment facility.

Solids Handling

A challenge with raw water delivery is solids handling. Phase I ASR includes solids handling, but has since been abandoned and now solids are discharged into the Little Arkansas River. Disposing the solids from El Dorado Lake directly into the Little Arkansas River would most likely not be permitted by KDHE. Therefore, solids handling facilities would have to be added to the ASR Facility. It is anticipated that the facilities would at a minimum include two gravity thickeners, three belt filter presses (2 online, 1 standby), and lamella plate settlers for backwash water reclamation. These facilities would be sized to meet the Total Maximum Discharge Limits (TMDL's) required under the Clean Water Act. In addition to solids handling at ASR, there is also concern of solids settling and microbial growth in the 38 miles of raw water pipeline which may necessitate periodic flushing and pigging activities. The 38 mile raw water main would have over 18 million gallons of water in it. When a flushing/pigging exercise occurred, provisions would be required to capture/treat the flush water.

Pre-sedimentation basins could be installed prior to transporting raw water, but we are concerned of potential challenges associated with this approach. Partial treatment of raw water near the reservoir would likely result in some treatment coordination issues with ASR. Pre-sedimentation would remove a majority of the solids, however there would still be some level of solids buildup in the pipeline creating periodic high turbidity events that may create problems for ASR membranes.

System Capacity

Overall system capacity and headloss in the conveyance system is another issue that would affect overall capital and O&M cost. Typically it is preferred to treat water close to the source in order to reduce the size of the conveyance system and minimize operational cost. The transmission of raw water would result in oversizing of the conveyance system to account for the eventual treatment loss of 5% to 10% of the water being transported. Essentially, water that would be wasted would have to be pumped the full pipeline length.

There would be times when delivery of raw water to the ASR would be not possible. When the ASR is running at full capacity to capture flow from the Little Arkansas River it will not be capable of taking raw water from El Dorado Lake. Based on intended operation of the ASR system, this condition may occur between 30 and 35 percent of the time.

Water Quality

Water age can be a concern with delivery of long finished water transmission mains. The approximate detention time in the 48-inch pipe is 11 hours. Fortunately, both utilities use chloramines as the secondary disinfectant within the distribution system and have low levels of regulated by-products. The delivery time in the pipeline should be evaluated during preliminary design, however, it should not cause noticeable increases in disinfection by-product formation.

Regional Water Needs

Delivery of raw water would inhibit future opportunities to serve neighboring communities with lower cost finished water. Each of the small communities would have to construct their own treatment facility to serve their community, losing the economy of scale which would result in higher cost water.

4.0 Current Status of Wichita Aquifer Storage and Recovery System

In the 1990's, the City of Wichita embarked on a multi-phased aquifer storage and recovery (ASR) project to replenish water supply for the city to meet future demands. Without additional source of supply, Wichita may not be able to meet its future projected demands. The concept of the ASR project is to divert flow from the Little Arkansas River, treat to drinking water standards, and inject into the aquifer to recover groundwater levels that have declined over time due to well production exceeding natural recharge. The total capacity of the system is dependent on available flow that can be diverted above the minimum allowable base flow allotted per negotiated water rights, and the infrastructure in place to convey and treat the supply. Table 4.1 summarizes the components and costs for each phase as originally envisioned.

4-1 ASR Phase Summary

PHASE	PROJECT COMPONENTS	ANTICIPATED PROJECT COST	SCHEDULE
Phase I	• 3 mgd of Bank Storage Wells • 7 mgd surface water intake • 7 mgd Actiflo surface water treatment plant • 4 recharge wells • 2 recharge basins	$27 million	Completed in 2006
Phase II	• 36 mgd raw water intake (expandable to 72 mgd) • 30 mgd surface water treatment plant (expandable to 60 mgd) • 31 dual purpose recharge/recovery wells • 31 miles of pipeline ranging from 12 inch to 66 inches • 40 miles of overhead power lines • SCADA Improvements • Well Field Maintenance Improvements	$275 million	Completed in 2011
Phase III	• Install remaining 36 mgd of pumping capacity at the river intake • Expand surface water treatment plant to 60 MGD • Install additional recharge/recovery wells • Install additional piping	$275 million	On Hold
Phase IV	• Construct bank storage wells along the Little Arkansas River with a total pumping capacity of 30 mgd • Install additional recharge/recovery wells • Install additional piping		On Hold
Information from HDR Report titled "ASR Program Review Water Supply Plan", dated May 2010.			

The total envisioned capacity of the ASR program is 100 mgd when all four phases are complete.

Based on the ASR Program Review Water Supply Plan, prepared by HDR Engineering, May 2010, the conditions for diverting water to the Aquifer are as follows:

- Water Right 46627, issued September 18, 2009, indicates the Little Arkansas River must maintain a minimum of 30 cfs at the Valley Center gage before water can be diverted for the ASR.
- Although the minimum allowable flow is 30 cfs, the raw water intake requires a minimum flow rate in the river of 50 cfs in addition to what is being withdrawn to maintain adequate submergence within the intake wetwell.
- The intake for Phase II is set to pump at constant rates of either 18 mgd or 36 mgd. Therefore, the minimum flow required in the Little Arkansas River before any water is diverted is 78 cfs (50 cfs minimum base flow plus 28 cfs(18mgd)).
- To maximize the full 36 mgd withdrawal capability of Phase II, the minimum flow in the river must be above 105 cfs.
- To maximize the full 72 mgd withdrawal capability of Phase III, the minimum flow in the river must be above 160 cfs.
- The Phase II surface water plant that treats the water prior to injecting into the aquifer can operate at either 15 or 30 mgd.
- The aquifer requires approximately 65 billion gallons of water to completely recharge. Assuming average rainfall and a 100 mgd ASR system, it is expected to take approximately 10 years. To completely recharge the Equus Basin within a 10-yr period, an average recharge of 17.8 mgd in addition to what is withdrawn out of the aquifer for use would be required.

4.1 Maximum Potential Long Term Annual Yield to Aquifer

Request for information on the current status, anticipated long term yield, and operational history of the ASR program has not been received to date from the City of Wichita. Therefore, historical flow rates measured at the Valley Center gage on the Little Arkansas River were analyzed to estimate the potential available yield of Phase II, and also the expected future annual yield if Phase III were implemented. This evaluation is based solely on stream data and the stated diversion criteria in the HDR report with no losses accounted for in capture efficiency, operational scheme, and treatment losses. At this time it is unclear how effective the system will operate in capturing the available supply over the long term without further discussion with the City on how well the system operates.

Data was evaluated for the periods from January 1, 1952 to December 31, 2009, and since 1994 to coincide with the same periods used in the El Dorado Lake yield analysis. The following flow conditions were used for the basis of the yield estimate:

- River flow less than 78 cfs - No flow diverted to treatment plant and aquifer
- ASR Phase II
    - River Flow between 78 and 105 cfs
        - 18 mgd diverted to treatment plant
        - 15mgd treated flow injected into aquifer
    - River flow above 105 cfs
        - 36 mgd diverted to treatment plant
        - 30mgd treated flow injected into aquifer
- ASR Phase III
    - River flow above 160 cfs
        - 72 mgd diverted to treatment plant (both phases)
        - 60mgd treated flow injected into aquifer (both phases)

Table 4-2 summarizes the potential yield to the ASR based on the flow conditions described above for Phase II and Phase III.

Table 4.12- ASR Potential Annual Treated Flow Injected in Aquifer based on Historical Stream Flows

FLOW CONDITION	RATE (MGD)	% OF TIME IN OPERATION		ANNUAL YIELD (ACRE-FT/YR)		AVERAGE FLOW (MGD)	
		Since 1994	1952 to 2009	Since 1994	1952 to 2009	Since 1994	1952 to 2009
Phase II							
River Flow < 78 cfs	0	55%	59%	-	-	-	-
78 cfs<River Flow<105 cfs	15	11%	9%	1,827	1,580	1.6	1.4
River Flow > 105 cfs	30	34%	33%	11,397	10,996	10.2	9.8
Phase II Total				**13,225**	**12,575**	**11.8**	**11.2**
Phase III (River Flow>160 cfs)	30	24%	23%	8,096	7,694	7.2	6.9
TOTAL- BOTH PHASES				**21,320**	**20,269**	**19.0**	**18.1**

The table above reflects the potential available yield if every drop of available water was captured and sent to the aquifer. These flows do not account for effectiveness of the system to capture all the available supply. Information from the HDR report indicates the facility associated with Phase II are expected to operate approximately 33 percent of the time, resulting in an average injection into the aquifer of approximately 10 mgd versus about 11.5 mgd included in this analysis. However, without additional information and understanding of the ASR program and treatment operation, we are unable to accurately estimate what percentage of the available flow is treated and injected into the aquifer. Therefore, for the basis of this evaluation, we have optimistically assumed that all available flow from the river is to be injected into the aquifer.

Conclusions

The data from both intervals of historical records result in similar yield to the aquifer. Based on this analysis, the data shows that the Phase II facilities would operate at full capacity approximately 33% of the time with annual raw water supply of about 11.2 mgd. If Phase III were implemented, an additional 6.9 mgd of flow would be diverted to the aquifer for a total recharge supply of about 18.1 mgd.

4.2 Comparison of El Dorado Water Supply to ASR Phase II and III Supply

The treated flow injected into the aquifer can be compared to the raw supply from El Dorado Lake because the water stored in the aquifer will need to be treated again prior to entering the City of Wichita's distribution system. Therefore, the treatment losses from the El Dorado Water Treatment Plant will be similar to the losses from the City of Wichita's treatment facilities. Table 4-3 compares supply from El Dorado Lake to the treated flow injected into the aquifer from the Little Arkansas River for two return intervals.

Table 4.2-Comparison of Potential Yield Injected in Aquifer versus Historical El Dorado Lake Supply

INTERVAL	ASR –PHASE II		ASR- PHASE III		TOTAL ASR	EL DORADO LAKE	
	% Time Full Capacity	Capacity (MGD)	% Time Full Capacity	Capacity (MGD)	Capacity (MGD)	% Time Full Capacity	Capacity (MGD)
Since 1994	34%	11.8	24%	7.2	19.0	88.5%[2]	26.6 [2]
1952-2009	33%	11.2	23%	6.9	18.1	78% [3]	23.4 [3]

1) Based on current El Dorado Lake volume from draft 2010 KBS lake survey, existing customer average demands of 9 mgd, and 30 mgd for new future customers when lake elevation is between 1334 and 1339 feet.
2) Long-term average is 26.6 mgd, with deliveries of zero (0) mgd for 22 months and deliveries of 30 mgd for 170 months of the analysis.
3) Long-term average is 23.4 mgd, with deliveries of zero (0) mgd for 153 months and deliveries of 30 mgd for 543 months of the analysis.

The comparison shows that the excess water in El Dorado Lake is higher than the total potential diversion into the aquifer for existing Phase II and future Phase III of the ASR program based on both long range analyses. Although no information was discovered for the anticipated recovery of stored river water using groundwater wells; it is likely that the recovery of the stored river water will be less than 100 percent. In addition, the capture efficiency for El Dorado Lake is expected to be higher since the lake levels remain more constant than flows in the Little Arkansas River, thus limiting the cycling on and off of the treatment facility. Therefore, the ASR capacities for actual deliveries to Wichita for finished water supply will be less than shown in the table.

4.3 Equus Bed Water Recharge to Date

Information on the current status and recharge to date for Phase II of the ASR program has currently not be obtained from the City of Wichita. The total gallons of water recharged through recharge basins and wells associated with Phase 1 of the ASR, initiated in 2006, is stored on the USGS Kansas Water Science Center website and summarized in Table 4.4.

Table 4.4 Historical Recharge from Phase I of the ASR Program

YEAR	AVERAGE RECHARGE (MGD)
2006	0.0
2007	1.0
2008	0.9
2009	0.5
2010	0.3
2011	0.0
2012	0.1
Average	**0.4**

The data shows that Phase I operated at its highest capacity of 1 mgd in 2007. The last two years have produced a total of 37 million gallons of recharge into the basin for an average of 0.05 mgd.

The primary goal of the ASR program is to recharge the aquifer to provide sufficient water storage to allow Wichita to continue to meet future demands during extreme drought conditions. If during non-drought periods the water from El Dorado is used to serve the customers while the majority of flow from the ASR is only used to replenish the aquifer, the goals of the ASR could still be achieved, much quicker and more reliably.

These conclusions from the ASR evaluation are based only on the operational conditions stated in the HDR report, available stream gage information, and information on the USGS website. Therefore, the conclusions from this evaluation should be verified with the City of Wichita to confirm the operation of the ASR system.

5.0 Water Treatment Plant Process Design

5.1 RAW WATER QUALITY

The raw water quality of El Dorado Lake is presented in Table 5.1 below.

Table 5-1 El Dorado Lake Raw Water

PARAMETER	AVERAGE
Turbidity	33 to 46 NTU
Total Suspended Solids	21 to 25 mg/l
Total Dissolved Solids	275 mg/l
Total Organic Carbon	2.6 to 4.2 mg/l
True Color	15 PCU (range 1 to 50)
pH	8.2 (range 8 to 9)
Alkalinity	120 mg/l as $CaCO_3$
Total Hardness	175 mg/l
Calcium Hardness	95 mg/l
Nitrate	0.29 mg/l
Total Kjeldahl Nitrogen	0.37 mg/l
Total phosphorous	75 µg/l
Iron	0.75 mg/l
Manganese	0.03 mg/l
Chloride	12 mg/l

The City has an active WRAPS Plan for El Dorado Lake and the City is committed in investing in Best Management Plans (BMPs) in the watershed to reduce sedimentation.

5.2 INTAKE AND RAW WATER TRANSMISSION

Raw water is supplied to El Dorado's existing Water Treatment Plant from two 36-inch inlet connections located in the dam outfall wetwell. The two inlets are piped together beneath the dam and exit through a single 36-inch pipe. The 36-inch pipe is located at the base of the wet well, approximate invert elevation 1272.50 feet. Downstream of the dam the 36-inch branches into a 24 inch cast iron line and a 36 inch prestressed concrete line that convey flow to the existing treatment facility.

The raw water supply to the new treatment facility will connect to the single 36-inch line near the outfall, and will require coordination with the USACE. The existing 24-inch and 36-inch raw water lines will continue to serve the existing treatment facility. The raw water piping would have a capacity of 40 mgd to serve both the existing and new treatment facility. Typically, velocity in the intakes can be designed up to 10 fps, and even up to 15 fps for intermittent flows. The velocity through the 36-inch pipe segment before the interconnect would be approximately 8.8 fps. Since there are two supply entry points at the dam, the inlet velocity at both points is approximately 4.4 fps. The high velocity through the 36-inch piping would create several feet of additional headloss that would have to be accounted for when selecting the treatment plant location. However, connecting to the 36-inch intake falls within acceptable design velocities and is a feasible approach to supply water to the new treatment facility.

It is anticipated that the new facility would be located at an elevation that would allow lake water to gravity flow to the treatment plant. However, the location for the new facility and hydraulic capacity of the intake and existing piping will need to be confirmed during the course of preliminary design.

5.3 CURRENT WATER TREATMENT PROCESSES USED BY WICHITA

Wichita has two primary sources of drinking water, the Cheney Reservoir constructed in 1965 by the US Bureau of Reclamation and the Equus Beds Aquifer wells. The Reservoir water is ozonated to oxidize taste and odor compounds as well as pesticide and herbicide compounds, then the Reservoir water is blended with the groundwater supply prior to coagulation/lime softening, filtration, disinfection using chloramines, and corrosion control through the addition of a phosphate inhibitor.

5.4 CURRENT WATER TREATMENT PROCESSES USED BY EL DORADO

The City of El Dorado treats the El Dorado Lake water using a conventional filtration plant with coagulation, sedimentation and dual media filtration. Powdered activated carbon is available as needed for improved disinfection by-product precursor removal and removal of intermittent taste and odor compounds. Disinfection is accomplished through the use of chloramines and the treated water is fluoridated.

5.5 COMPARISON OF FINISHED WATER QUALITY FROM WICHITA AND EL DORADO

Water utilities are required to provide information to customers about the quality of the water that they serve every year. The information is compiled in Consumer Confidence Reports and sent to each customer account yearly to update customers about regulatory compliance and changes in water quality. Data from the most recent Consumer Confidence Reports for Wichita (2011 data) and El Dorado (2010 data) are compared below.

Table 5-2 Comparison of Finished Water Quality

PARAMETER, UNITS	WICHITA	EL DORADO
Arsenic, µg/l	2.1	<1.6
Fluoride, mg/l	0.32	0.7
pH, S.U.	7.9	7.4 to 8.5
Alkalinity, mg/l as $CaCO_3$	94	90 to 120
Calcium, mg/l	26	23 to 35
Magnesium, mg/l	14	4 to 6
Potassium, mg/l	4.4	2 to 4
Sodium, mg/l	90	<7
Chloride, mg/l	120	7.5 to 9
Nitrate, mg/l	0.8 to 1.4	<0.32
Sulfate, mg/l	67	3.5 to 7
Silica, mg/l	17	2 to 11
Total dissolved Solids, mg/l	400	120 to 155
Bromate, mg/l	ND to 14	No data
Haloacetic Acids (5), µg/l	5 to 11	20
Trihalomethanes, µg/l	18 to 25	19
Total Organic Carbon, mg/l	2.2	Removal ratio 1
90^{th} % Copper, mg/l	0.086	0.25
90^{th} % Lead, µg/l	7.5	4

The treated El Dorado water is similar to the Wichita water with regard to calcium, potassium, and alkalinity concentrations, but significantly different with regard to sodium, chloride, sulfate, and total dissolved solids concentrations. For system blending, the calcium and alkalinity are both very important water quality parameters so having similar values for the Wichita and El Dorado match which is positive. El Dorado currently produces water with a much greater range of pH than Wichita, so pH control will be a critical feature of a new Western Water Supply plant to match the pH of the Wichita supply.

Both utilities use chloramines as the secondary disinfectant within the distribution system and have low levels of the regulated disinfection by-products – haloacetic acids and trihalomethanes.

Further evaluation of the organic removal requirements of the El Dorado Lake water should be undertaken as part of the more detailed design. However, with minimum chemical and treatment adjustments the two water supplies are compatible and can be blended in the distribution system.

5.6 PROPOSED PROCESS FOR WICHITA WESTERN SUPPLY FROM EL DORADO RESERVOIR

Based on the processes used currently to treat the El Dorado Lake water, we propose the following treatment processes for a Western Wichita Supply from El Dorado Lake. The facility will be designed for a maximum capacity of 30 mgd.

Powdered activated carbon would be added upstream of the rapid mix basins while polyaluminum chloride coagulant would be introduced at the rapid mix basins for rapid introduction of the chemical with good mixing into the raw water. The coagulated water would then be gently mixed in the flocculation basins to permit the coagulants to react with particulate and dissolved compounds, such as iron and total organic carbon. The coagulants, turbidity, and dissolved compounds form flocs which will settle to the bottom of the sedimentation basins if given enough time. Final particulate removal will be accomplished by filtration. The filtered water would then be disinfected and have final pH and corrosion control treatments. A description of each proposed chemical addition and process step is provided below. A process schematic is shown in Figure 5-1.

Powdered activated carbon (PAC) is similar to granular activated carbon although it has a much smaller grain size. The carbon undergoes activation to form pores that contain sites on which carbon compounds can attach and be adsorbed. PAC has a relatively high adsorptive affinity for taste and odor-causing compounds and naturally occurring organic matter. Factors that affect PAC performance include contact time with the target contaminants, interference by treatment chemicals, and competition from other naturally-occurring organic compounds in the water being treated. When added in conjunction with coagulants, its adsorption capability can be inhibited by enmeshment in coagulant flocs. Therefore, PAC should be fed upstream from the point of coagulant, polymer, and/or oxidant addition to maximize the potential for adsorbing taste and odor-causing compounds. This could occur in a dedicated PAC adsorption basin that provides mixing to maintain solids suspension and good contact between the PAC and solution. With conventional treatment, PAC is commonly added before or just within the rapid mix basin and contact time is provided during flocculation and sedimentation.

Conventional coagulation-flocculation-sedimentation consists of rapid mixing, coagulation, flocculation, and sedimentation in large rectangular or circular basins. The goals for coagulation, flocculation, and sedimentation include turbidity reduction, particle conditioning for filtration, and removal of organic compounds. Particles are removed by coagulation, which involves the use of a chemical coagulant for neutralizing the electric surface charge on small particles. The particles can then combine to form larger aggregate particles, or floc, by gentle mixing. These larger particles are then allowed to settle in the sedimentation basin. Because many of the pathogens of concern, such as microorganisms, are actual particles, they can be effectively removed by coagulation, flocculation, and sedimentation. Total organic carbon (TOC) compounds can be removed by adsorption onto the floc particles.

Coagulation, flocculation, and sedimentation, followed by filtration has been demonstrated to remove turbidity, oxidized iron and manganese, color, TOC, viruses, bacteria, Giardia cysts, and Cryptosporidium oocysts. Removal of naturally-occurring organic material depends on the pH, the coagulant dose, and the type of organic material. Well-operated plants can produce settled water with a turbidity less than of 1.0 NTU and filtered water with a turbidity of less than 0.10 NTU.

Membrane filtration, which involves removal of suspended particles by using low-pressure hollow polymeric fiber membranes, are becoming increasingly popular as an alternative to conventional granular media filtration processes. Microfiltration (MF) and ultrafiltration (UF) are physical processes of removing colloidal particles from water by straining it through a porous membrane. Both processes achieve exceptional turbidity removal; most operating facilities routinely produce treated water (permeate) with turbidities less than 0.05 NTU. MF membranes are typically used for treatment of surface water and consist of hollow fibers with a nominal pore size of 0.1 to 0.5 micron UF membranes have a slightly smaller nominal pore size of 0.01 to 0.05 micron. As these pore sizes are significantly smaller than Cryptosporidium cysts (2 to 7 microns) and Giardia cysts (5 to 15 microns), both MF and UF achieve excellent removal of these contaminants. Pilot-scale testing has achieved 6 to 8-log removal of Giardia cyst-sized particles, therefore many states grant 3-log, and, in some cases 4-log removal credits for treatment by MF and UF. For encased membrane systems, the feed water is pressurized and forced through the membranes at average pressures between 10 to 20 psi.

More recently, ceramic membranes which were a wastewater treatment technology, are becoming more cost competitive in the water industry. Therefore, a comparisons of encased polymeric membrane filters and ceramic filters should be evaluated during the preliminary design phase.

For a century, water utilities have been using chlorine for primary and secondary disinfection. Chlorine is highly effective for inactivation of viruses and Giardia, although it is ineffective for inactivation of Cryptosporidium and for oxidation of the taste-and odor-causing compounds MIB and Geosmin. Chlorine does need time to react, so a baffled chlorine contact tank is used to provide the required contact times under the worst case flow and temperature conditions for regulatory compliance. The most commonly used forms of chlorine used for disinfection are chlorine gas, delivered sodium hypochlorite, and on-site generated sodium hypochlorite. Which form of chorine is used is usually determined by factors such as specific project requirements, cost of local delivered chemical, and power cost.

Chloramine is formed when free chlorine is combined in water with ammonia. Because chloramine is not as effective as free chlorine for inactivation of most microbial organisms, it is generally used as a secondary disinfectant in the distribution system, rather than as the primary disinfectant. Unlike free chlorine, chloramine does not promote the formation of halogenated byproducts; although its byproducts, nitrosamines and cyanogen chloride, neither of which are currently federally regulated in drinking water, may be of future concern. Presently, there is little definitive information regarding their health risks or the concentrations at which they may become a cause for concern.

Wichita maintains a target pH of 7.9 so the Wichita supply would be provided with pH adjustment chemical additions to permit raising or lowering of the finished water as necessary. Wichita also uses a phosphate inhibitor for corrosion control and sequestering of hardness so chemical feed facilities for this chemical would also be a necessary part of the plant.

The coagulation/sedimentation and filtration processes remove particulates, coagulants and dissolved compounds from the water in the form of flocs. The filters also require periodic backwash with clean water to function effectively. The liquid portions of the sedimentation flocs and the backwash can be recaptured and sent through the treatment process again to maximize the plants efficiency and reduce water losses. The solid residuals must be treated and disposed of in a manner acceptable to KDHE. It is anticipated that water residuals lagoon will be provided to store and dewater residual solids from the sedimentation basin and filter backwash. The sedimentation basin and filter backwash residuals will be gravity fed to a wet well and the combined residuals pumped to a lagoon consisting of two cells. Each cell will be built with enough capacity to store two years of residuals at average plant solids production. It is anticipated each cell will actively receive residuals for a year; followed by a period of one year when the residuals are decanted, allowed to air dry, and removed for landfill disposal. Providing each cell with capacity for two years allows for provisions to delay disposal for a year in the event wet conditions do not allow necessary drying to occur.

Flow from the contact basins would enter a finished water clearwell that will serve as the wetwell for the distribution pumping station. The size of the finished water clearwell may vary based on the end user minimum and peak demands, and overall system configuration. It is anticipated flow from the plant will be pumped to the 10 million gallon Webb Road reservoir. Coordination between El Dorado and Wichita on the operation of the facility and the ability to control flow will be a necessity to size the clearwell to provide adequate equalization. Typically, 5 percent of plant volume is used as general guideline for clearwell storage. Therefore, a 1.5 million gallon finished water clearwell is included.

Table 5-3 summarizes the major components for the treatment facility.

Table 5-3 Summary of Process Components

COMPONENT	NUMBER OF UNITS	DESIGN PARAMETERS
Rapid Mixer	2	Residence time: 20 seconds Dimensions: 8 ft x 8 ft x15 ft
Flocculation Basins	2	Detention Time: 30 minutes Trains: 2, Cells: 3 Cell Dimension: 17 ft x 17 ft x 17 ft Dimension: 51 ft x 51 ft x 17 ft
Sedimentation Basins	2	Detention Time: 3.1 hrs Surface Loading Rate: 0.6 gpm/sf Dimensions: 52 ft x 300 ft x 17 ft
Membrane Facility	8	Recovery Rate (%):95 Design Operating Flux (gfd):35 to 37 System Redundancy (%):50 Wash Cycles: 3
Contact Basin	1	Volume: 1.2 million gallons Dimensions: 75 ft x 220 ft x 12 ft Number of Baffles: 3
Finished Water Clearwell	1	Volume: 1.5 million gallons Dimensions: 120 ft x100 ft x 17 ft
Residuals Pumping Station	1	Volume: 30,000 gallons Dimensions: 15 ft x 15 ft x 22 ft Pumps: 2
Lagoon	1	Cells: 2 Cell Dimensions: 500 ft x 300 ft x 12 ft

5.7 FACILITY SITING

The location of the treatment facility is dependent on land available that the city owns or will need to purchase. Therefore, the exact location of the treatment facility cannot be determined at this time. The facility should be located on property where the site elevation is low enough to allow gravity feed from El Dorado Lake to the treatment process. The minimum lake operating level where flow would no longer be sent to Wichita is at elevation 1,334 ft. However, the plant should be located where it could draw the water level below this elevation if necessary to provide additional treatment capacity to the City of El Dorado or utilize for other purposes in the event of long term drought conditions.

Therefore, the required site elevation is based on the distance the facility will be from the intake and the desired minimum lake level. Generally, for every 1,000 ft the treatment facility is from the intake, the required site elevation should be 2 to 3 ft lower than the minimum lake elevation under which the plant will be operated. Based on a cursory review of existing topography, suitable land for the treatment facility exists downstream of the dam and west of the intake. A minimum of 15 acres should be obtained by the City for the treatment facility, and an additional 10 acres should be obtained for the lagoons. A preliminary site plan is shown in Figure 5-2.

6.0 Delivery Options

Factors that were evaluated to establish the conveyance system components required to deliver finished water to Wichita include point of connection to Wichita's distribution system, transmission main alignment, pumping station configuration, pipe sizing, and material selection. The purpose of this section is to identify the facilities, in sufficient detail, to develop conceptual construction and operation and maintenance costs. It is recognized that there are many features of the system that require input from the owner(s), governing agencies, permitting process, and potentially the contractor during detailed design. Therefore, the facility components presented should be used only as a baseline for required infrastructure to delivery finished water to Wichita.

6.1 POINT OF CONNECTION

Connecting a large wholesale water source to an existing distribution system can be difficult if piping infrastructure is not properly sized and multiple pressure zones exist within the system. Typically, connecting the new water source at either the plant high service pumping station or a distribution system storage reservoir provides the best locations for wholesale connection.

Therefore, the recommended connection point is the Webb Road Pumping Station located in the northeast portion of Wichita's distribution system. This location would afford the City of Wichita good flexibility, such that the water could either be transferred back to the Hess Pumping Station through the Northeast Transmission Main or discharged into the 10 million gallon underground reservoir at Webb Road, or both simultaneously, as demands dictate. The reservoir would serve as equalization between the systems and allow Wichita to deliver the water with their existing pump station to the distribution system. Therefore, the full quantity of supplied water can be utilized for more than just the pressure zones served by the Webb Road Pumping Station.

The Webb Road Pumping Station has a current firm capacity of 38.0 mgd. Minimal pump improvements or the addition of a satellite pumping station may be necessary to distribute the entire 28.5 mgd under all demand conditions. However, we do not anticipate major improvements.

6.2 TRANSMISSION MAIN ALIGNMENT

Several alternative routes were considered to convey finished water to the City of Wichita. Selecting the final route should consider factors such as system hydraulics, operational costs, available easements, total transmission main length, future land use, topography and physical features, rock depth, and known environmental and historical sites. When finalizing the route, the City should also recognize that other potential water users along the route may benefit from the water supply. As stewards of the region, the City should consider adhering to their supply deficiencies to accommodate additional service connections along the route. However, no single user along the route should have a significant impact in the overall alignment. Therefore, for the basis of this evaluation, the most significant cost factor and driver for the selected route was distance between the treatment plant and point of connection to Wichita.

As noted in Section 5.7, the location of the treatment facility is unknown. For the basis of this cost evaluation, the starting point for the finished water transmission main was assumed at the intersection of Highway 77 and 12th Avenue. The most feasible direct route generally is described as from the new treatment facility west to the I-35 Kansas Turnpike, southwest for approximately five miles along the I-35 Kansas Turnpike, then west along a combination of county and private easements. The estimated length of the transmission main is 28 miles. Refer to Figure 6-1 for preliminary transmission main route. Refer to Figure 6-2 for approximate ground profile.

Temporary and permanent easements will be required to be obtained for the transmission main. An estimated 40 ft permanent easement and additional 20 ft temporary easement should be maintained for the majority of the route. In congested areas or areas with existing easements, smaller easements could be considered.

The proposed route includes approximately five miles along the Kansas Turnpike. The Kansas Turnpike Authority (KTA) was contacted to determine if their utility easement could be utilized for the transmission main. The representative indicated that KTA maintains a 300 ft right-of-way. The representative indicated KTA would be open to consider allowing the transmission main within their easement. However, it would need board approval and most likely require a lump sum payment or annual fee. The representative did indicate their utility easement is already congested in some areas and had concern as to whether there would be sufficient space available to install a large diameter main.

During preliminary design alternative routes should be considered, such as easement purchased directly adjacent to the turnpike, or alternatively, a route along Highway 254 between El Dorado and Towanda, that takes advantage of the diagonal route.

Negotiations and easement acquisition may ultimately impact the final transmission main route. Therefore, the City should allow some flexibility in the alignment and sufficient time in the schedule to fully obtain easements before final design of the transmission main.

A significant cost factor for the pipeline installation is depth to bedrock. The state of Kansas has some resources available that can be used to identify bedrock depths along the route. Based on the data available, any route that would go from the El Dorado to Wichita would require the same percentage of rock removal for installation of the pipeline. Therefore, as stated above, the most direct route should be taken to limit rock removal. Additional investigation regarding the rock depth should be completed prior to selection of the final pipeline alignment. Figure 6.1 shows the approximate limits of rock for the route, and costs for removal of rock were included in the overall project costs.

6.3 PIPELINE SIZING

The selection of the transmission main diameter will directly impact the number of pumping stations required, pipe type and pressure classification, and capital and operation costs. Typically for this level of evaluation, standard design practices for long transmission mains are used to size the transmission main. These standard practices limit the velocity to 5 feet per second and overall friction headloss to between 1 to 2 feet per 1,000 ft of linear piping. Three pipe sizes were initially considered as viable alternatives for the transmission main. For each size, the estimated total headloss was calculated in the pipeline to determine the pipe pressure ratings and pumping requirements. Table 6-1 summarizes the estimated headloss and pipeline pressures for a 36-inch, 42-inch, and 48-inch pipeline.

Table 6-1 Finished Water Pipeline Design Criteria

CRITERIA	48-INCH PIPE	42-INCH PIPE	36-INCH PIPE
Flow rate, mgd		28.5	
Velocity, fps	3.5	4.6	6.2
Webb Road Overflow Elevation, ft		1395.4	
Assumed Clearwell Elevation, ft		1310	
Static Head, ft		59.4	
Minimum System Pressure, psi[1]		30	
Pipeline Length, miles	28	28	28
Estimated Number of Isolation Valves [2]	5	5	5
Estimated Minor Loss (90 bends) [3]	28	28	28
Friction Headloss, psi[4]	79.3	151.5	320.3
Working Pressure, psi	116.2	188.5	357.3
Estimated Surge Pressure, psi	100	100	100
Total design pressure, psi	216.2	288.5	457.3
Headloss per 1,000 ft	1.2	2.4	5.0
Travel Time in Pipeline, hrs	11.7	9.0	6.6
Number of Pumping Stations	1	2	3

1. **Minimum design pressure set at 30 psi.**
2. **Based on one isolation valve every 5 miles.**
3. **Minor losses estimated as equivalent to one 90 degree bend per mile**
4. **Based on Friction Factor (C=100)**
5. **Flow rate is based on approximately 5% losses through treatment facility.**

The hydraulic analysis shows that the 48-inch pipeline is within the standard design practice of between 1 to 2 feet of headloss per 1,000 feet. Typically, the total design pipeline pressures should stay below 250 psi to avoid additional costs for high pressure rated pipe, valving, and appurtenances. The total pressure is the working pressure plus the calculated surge. A detailed surge analysis should be performed during design for the transmission main. For ductile iron, steel, and concrete pipe, the surge for a 2 fps velocity change will be approximately 100 psi. Therefore, for this analysis a 100 psi surge pressure was estimated and added to the working pressure to establish the pipeline design pressure. Based on this analysis, the total system pressure is below

250 psi for the 48-inch pipeline. Therefore, only one pumping station, located at new treatment facility, would be required.

The headloss in the 42-inch pipeline is approximately 2.4 ft per 1,000 feet of piping, slightly above the standard recommendation for transmission main friction losses. The velocity in the pipeline is within acceptable limits. Therefore, a 42-inch pipeline could be considered during final pipeline size selection. However, since 42-inch pipe is less common, it typically does not present substantial cost savings as compared to the 48-inch pipe. In addition, the 42-inch pipeline would likely require one additional booster pumping station located along the route. The additional pumping station would require more operational and maintenance cost.

Decreasing the pipe size to 36 inches would significantly increase the velocity and headloss through the piping. Both the velocity and headloss would be well above the recommended design guidelines for long transmission mains. Two booster pumping stations would be required to deliver the water, increasing the operation and maintenance costs. One benefit to the smaller pipe would be the shortened delivery time to the Wichita distribution system. The shorter delivery time could improve the water quality at the delivery point, reducing the potential for disinfection byproducts to form in the long transmission main.

For this analysis, the 48-inch pipeline was used to develop conceptual level costs. The 48-inch pipe both meets the recommended velocity and headloss in a long transmission main, and allows one pumping station to deliver the water.

6.4 PIPELINE MATERIAL

The four pipe materials that should be considered for the transmission main are (1) bar-wrapped steel cylinder concrete pressure pipe (ANSI/AWWA C303), (2) ductile iron pressure pipe (ANSI/AWWA C151), (3) steel pipe (ANSI/AWWA C200), and prestressed concrete cylinder pipe (PCCP). The selection of the pipe material should be based on the following design criteria: history and availability, linings and coatings, joint types, fittings and appurtenances, corrosion protection, opinion of probable pipe cost, and additional considerations. When properly designed, manufactured, tested, installed, and maintained, each of the three pipe materials would be acceptable and perform well for the 48-inch transmission main. Since the majority of the pipeline will be in rural areas that will allow long pipe segments to be installed, steel pipe was selected as the basis of cost for this analysis. The steel pipe can be delivered in segments up to 50 ft long. However, prior to proceeding with installing steel pipe, a conductively analysis to determine the corrosiveness of the selected route should occur to confirm steel pipe is suitable for the type of soil present.

6.5 PUMPING STATION CAPACITY

The pumping station would be located at the treatment plant would draw water from the finished water clearwell for pumping to the Webb Road Reservoir. The pumping station would have a firm capacity of 30.0 mgd with a rated head of approximately 150 psi. Vertical turbine pumps would be located on top of the finished water clearwell. Table 6-2 summarizes the pumping station requirements.

Table 6-2 High Service Pumping Station Capacity

CRITERIA	
Number of Pumps	4
Flow rate per pump, mgd	10
Rated head, psi	150
Total Pumping Capacity, mgd	40.0
Firm Pumping Capacity, mgd	30.0

6.6 PIPING APPURTANCES

Proper design of a transmission main includes appurtenances for the transmission main to function properly, protect the pipeline and equipment, and allow City crews to properly inspect and maintain the pipeline. These appurtenances include the following:

- Sectionalizing valves
- Air release/Vacuum relief valves
- Drain valves (blowoffs)
- Access manways
- Marker posts
- Cathodic protection and corrosion control provisions

Sectionalizing Valves.

Sectionalizing valves are provided in transmission mains so that a reasonable length of the transmission main can be isolated for maintenance and inspection without draining the entire transmission main. An air release/vacuum relief valve is typically provided adjacent to an isolation valve to release air when the valve is closed and the pipeline is filled, and to admit air when the isolation valve is closed and the isolated section of the main is drained. At least one drain valve (blowoff) facility is required between isolation valves. For the basis of cost, an isolation valve was located every five miles along the pipe route.

Air Release/Vacuum Relief Facilities.

Air release and vacuum relief valves will be installed at high points along the pipeline profile and where the transient analysis indicates additional air venting and/or vacuum relief capacity is needed. The air release connection will be configured to allow a manway to allow access inside the pipe for construction and if necessary repairs are required. The merits of sizing and locations of the vacuum relief valves for gravity flow will be further evaluated as part of final design. For the basis of the cost, an air release/vacuum relief facility was located every mile along the pipe route.

Drain Valve (blowoff) Facilities.

Blowoff facilities will be installed at low points in the pipeline for flushing and draining of the pipeline. Blowoff facilities at intermediate low points affecting a short distance of the pipeline will not be provided unless requested by the Owner. The 48-inch pipeline contains 496,000 gallons of water per mile. To limit the amount of water released at one location to less than 1 million gallons, a blowoff facility was included every two miles.

Pipeline Marker Posts.

Marker posts will be placed along the pipeline alignment to aid in locating the pipeline and appurtenances in the field by maintenance crews, and provide a warning to others of buried infrastructure. Marker posts will be provided at the following locations:

- horizontal changes in alignment;
- fence crossings;
- intermediate test lead stations (not adjacent to valves or vaults); and
- other locations requested by the City.

Black & Veatch also requires GPS locating during the construction process to provide detailed location of the main.

Cathodic Protection / Corrosion Control.

The corrosion and cathodic protection measures required will vary depending on the pipe material, the soil conditions, wetting and drying cycles, and AC induction and DC interference sources.

Cathodic protection measures may include galvanic anodes or impressed current anodes with rectifiers, as well as field test stations to allow monitoring of the cathodic protection system. A detailed evaluation of corrosion and cathodic protection requirements will be performed as part of final design.

7.0 Cost Evaluation

Estimated capital, and operation and maintenance (O&M) costs were developed for the facilities required to deliver finished water from El Dorado Lake to the City of Wichita. The costs will be used to develop a business plan for the sale of water.

This section describes the rationale used in developing estimates of capital, operating, and maintenance costs for each facility.

7.1 CAPITAL COSTS CRITERIA

Estimates of capital costs were developed from unit and lump sum prices for the various components for each facility. Pricing was based primarily on material quotes from various vendors and manufacturers, past experience, and information from similar projects recently constructed in the region. Where applicable, quantities were obtained from the Water District No. 1 of Kansas City, Missouri 30 mgd Membrane Treatment Facility, the 10 mgd Sioux City, Iowa Membrane Treatment Facility that were recently designed by Black & Veatch. Additional amounts for general requirements (permitting, contingencies) and engineering, legal, and administrative costs were combined to obtain a total estimated capital cost for the project.

Quantities for structures, building, process components, pipeline lengths, and basin sizes were developed based on preliminary process sizing of the treatment components, preliminary site layout, and similar regional plant facilities. The following general assumptions were used to develop the water treatment plant capital costs.

- The flocculation and sedimentation basins would be uncovered concrete structures. Approximately 25 percent of the excavation was assumed to be in rock. Manufacturers were contacted to obtain budget quotes for the rapid mixers and sludge collection equipment.
- The membrane facility would be housed in a single story pre-engineered metal building. The cost for all the membrane equipment components, including pumps, valves, strainers, and all piping except the inlet and outlet piping were provided by Pall.
- The contact basin was sized to meet the disinfection contact requirements. The covered contact basin includes masonry baffle walls.

- A 1.5 million gallon finished water clearwell was used to serve as the wetwell for the high service pumping station. The clearwell would be constructed adjacent to the contact basin to utilize common walls.
- Four vertical turbine pumps would be situated on top of the clearwell to delivery finished water to Wichita. The pumps would be housed in a pre-engineered metal building.
- The chemical feed facility would be a single story masonry building. All storage tanks and feed equipment would be located inside the building. Pricing for the chemical feed equipment was obtained from similar projects.
- Residuals handling consists of a submersible pumping station to convey the solids to a two cell lagoon. The decant from the lagoon would flow by gravity to the receiving stream. Periodic cleaning of the lagoon would be required.
- Costs do not include excess sizing of components and piping for future expansion.
- Overall plant layout would be condensed to minimize land and piping costs.

Pipe lengths for the raw water and finished water transmission mains length were based on aerial imagery from the USDA Geospatial Data Gateway and Google Earth.

Soil surveys provided by the NRCS (Natural Resources Conservation Service) for Butler and Sedgwick County were used to estimate the composition of soil and rock excavation along the transmission main route. Using the Web Soil Survey tool AOI (Area of Interest) segments were formed and a custom soil resource report was formulated. This provided the location, typical soil profiles, and properties of the AOIs. Using this information Black and Veatch compiled these segments in order to assess the entire route. The results were that the majority of the transmission route had one of the two following soil profiles:

- Soil profiles where the depths to the restrictive feature were 20 to 40 inches
- Soil profile to lithic bedrock of more than 80 inches.

In the areas were the soil was classified as “20 to 40 inches” an average restrictive depth of 30 inches was used for the rock estimate. In areas were the soil was classified as more than 80 inches no restrictive excavation was included in the estimate.

As a result of the use of the small-scale maps and the limited features associated with this level of mapping, these estimates should be considered approximate and for preliminary budgeting purposes only.

Ten percent of the construction cost was added to all components of the water treatment facility as an allowance for mobilization(s), bonds, insurance, supervision, temporary facilities, temporary utilities, equipment rental, and miscellaneous for the water treatment plant construction. Eight percent was added for the transmission main construction since less equipment and components are required for installation.

Contingencies are defined as unknown or unforeseen costs. The level of detail available at the planning/conceptual phase of the project does not provide sufficient definition to fully capture all the costs associated with the project

The Association for the Advancement of Cost Engineering (AACE International) defines five levels of "class estimates" that are typically used for planning purposes. These range in level of complexity from Class 5 (generally associated with conceptual level evaluations) to Class 1 (prepared to confirm the control baseline for a project). A Class 4 Estimate was utilized for this project. Following is a brief description of a Class 4 estimate:

- Class 4 estimates are generally prepared based on limited information and subsequently have fairly wide accuracy ranges. They are typically used for project screening, determination of feasibility, concept evaluation, confirmation of economic and/or technical feasibility, and preliminary budget approval. Typically, engineering is from 1% to 15% complete. Typical accuracy ranges for Class 4 estimates are -15% to -30% on the low side, and +20% to +50% on the high side, depending on the technological complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination. Ranges could exceed those shown in unusual circumstances.

Thirty percent of the construction cost was added to each component of the treatment facility as a contingency, which is customary for projects at this level of development. Twenty percent was used for contingency for the transmission main as there are fewer overall variables relative to cost associated with a transmission main versus treatment facility. Twenty percent of the construction cost was allocated for engineering, easements, permitting and project approvals, legal, and administrative costs associated with each facility. Table 7-1 summarizes the values used to estimate capital costs and Table 7-2 provides a summary of the preliminary capital cost estimates.

Table 7-1 - Capital Cost Criteria

ITEM	UNITS	UNIT PRICE	
		Water Treatment Plant	Transmission Main
General Conditions	% of Total	10%	8%
Electrical	% of Total	18%	N/A
Instrumentation	% of Total	7%	N/A
HVAC	% of Total	5%	N/A
Plumbing	% of Total	5%	N/A
Land Costs(in 2012 dollars)	per acre	$20,000	$20,000
Contingency	% of total	30%	20%
Engineering, Legal, Easements, and Administration	% of total	20%	20%

Notes:

1. The % of Total does not include residuals lagoon and engine generator for the HVAC and plumbing totals.
2. The % of Total does not include the residual lagoon for electrical and instrumentation.

Table 7-2 summarizes the total capital construction costs for the facilities.

Table 7-2 Summary of Construction Costs (in 2012 Dollars)

ITEM	COST
Water Treatment Plant	$62,117,000
Raw Water Transmission Main	$4,523,000
Finished Water Transmission Main	$59,573,000
Total Construction Costs	**$126,213,000**