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Appendix B – Water Rights Information Reports
Appendix C – Well Construction Information
Appendix D – Well Performance Graphs
Appendix E – Previous Studies on Additional Wells
Appendix F – Hydraulic Model Information
Appendix G – Soil Corrosivity Evaluation
1 Introduction

The City of Olathe completed a Water System Master plan in December 2006. The City has continued to experience a high growth rate and development. This Study is intended to be an update of the 2006 Study, that takes into account the water system improvements, additional growth and development and provide a 40-year Master Water Supply Plan.

1.1 Background

The City has been serving drinking water to its residents since approximately 1884. A new water supply was constructed in 1914 at Cedar Lake with a water treatment plant near South Water Works Lake, capable of treating 0.5 million gallons per day (MGD) to serve a population of 3,200. Severe drought in the 1950s necessitated construction of Lake Olathe and Water Treatment Plant No. 1 (WTP1). Lake Olathe was the City’s primary water source until 1964, when the City developed groundwater wells along the Kansas River and constructed Water Treatment Plant No. 2 (WTP2). The City has since taken Lake Olathe and WTP1 out of service, and now fully relies on the Kansas River well field – eight vertical wells (VWs) and four horizontal collector wells (HCWs). Presently, the City provides a maximum day rate of 28 MGD of drinking water to more than 35,000 customers.

1.2 Purpose

The purpose of this Master Plan is to update the existing Water Master Plan by evaluating the exiting system and determining the feasibility and requirements for future development of the City’s water supply and distribution system through ultimate build-out (2055).
2 Water System Characterization

This Section summarizes the components of the existing water system including the water service area; water rights and water supply facilities; water treatment; and high service pumping, distribution system piping, storage, and pumping.

2.1 Water Service Area

The City is the primary water utility provider in the City. Other water utility providers within the current incorporated limits are Water District No. 1 of Johnson County (WaterOne) and Johnson County Rural Water District (RWD) No. 7. New Century Aircenter and Johnson County Consolidated Rural Water District (CRWD) No. 6 have water service areas within the future annexation limits of the City. The City’s water service area and the water service area of other providers within the region are shown in Figure 2-1.

WaterOne generally serves City residents south of 159th Street and east of Interstate 35. The City of Olathe and WaterOne have four (4) interconnections:

- 118th Street & Renner Boulevard (at Renner Tank and Pump Station)
- 151st Street between Keeler and US Highway 169
- Olathe’s Hedge Lane Pump Station
- College Boulevard and Lone Elm Road

The City and WaterOne have an agreement for water sales such that each utility can provide surplus water to the other on a reciprocal basis during emergency conditions. The agreement is in effect until April 2018. “Emergency conditions” are defined as an unexpected failure of a component of the water supply, treatment, or distribution system, or an abnormal system demand such as fire, but does not include regional drought conditions.

The City provides wholesale water service to New Century Aircenter through a contract with the Board of County Commissioners of Johnson County. The current contract to provide wholesale water is in effect until October, 2035. The point of delivery for New Century is located at North Loop Road and New Century Parkway and includes a flow meter to measure the amount of water sold. The City of Olathe, per the contract, has agreed to provide a hydraulic gradient of 1,150 feet under normal conditions at the point of connection. The contract allows for construction of a second point of delivery in the future, at which time the hydraulic gradient and maximum rate of delivery will be reviewed by both parties.

Johnson County CRWD No. 6 is another water utility with a service area within the City limits; the City provides wholesale water to CRWD No. 6. Their service area is primarily in the northwest area of the City. The contract on file with CRWD 6 expires at the end of 2017. CRWD 6 meters the water purchased at its point of delivery, which is located a half-mile west of Hedge Lane on 135th Street. The contract provides for water to be provided under a hydraulic gradient of 1,170 feet under normal conditions with a minimum hydraulic gradient of 1,150 feet.
Notes:
1. Transitional areas are those areas of RWD 7's service area that Olathe has first right of refusal for water service.

Map Feature Key
- Current City Limits
- Future Annexation Limits
- Interconnect with WaterOne
- Wholesale Delivery Point

Water Service Provider
- No Service Provider
- City of Olathe
- WaterOne
- New Century
- Johnson County RWD 7
- Johnson County RWD 6
- Transitional Areas of RWD 7 (See Note 1)

Olathe, Kansas
Water Master Plan Update
Figure 2-1. Olathe and Surrounding Water Service Areas
The City also provides wholesale water service to Johnson County RWD No. 7. The current contract is in effect until December 2031 and includes an increasing schedule of water sold through 2031. The point of delivery to RWD 7 is located along 151st Street west of Lakeshore Drive; the contract provides for delivery at a hydraulic gradient of 1,145 feet under normal conditions. The contract allows for construction of a second point of delivery along 143rd Street in the future, at which time the hydraulic gradient and maximum rate of delivery will be reviewed by both parties.

The contract discusses an agreement between the City and RWD 7 for the City to assume ownership of certain pipelines within the RWD 7 service area. These transitional areas are shown in Figure 2-1. Included within these areas are 8-inch mains in the Greentree Subdivision, along 143rd Street from Greentree Drive to Cedar Niles Road, and north of 143rd Street on Cedar Niles Road, which are expected to enhance services to City water customers in the general area. The City will assume ownership of the RWD 7 pipelines within the transitional areas upon 90 days of written notice to RWD 7; transfer of ownership will be made in exchange for construction of the future second point of delivery to be constructed and funded by the City.

2.2 Water Supply

The City receives all of its raw water supply from alluvial wells located along the Kansas River east of De Soto, Kansas. At this location, the sand and gravel aquifer is directly connected to the Kansas River and relies on in stream river flows to maintain well production rates. The water supply facilities are shown in Figure 2-2.

The well field began with four VWs installed in 1964 and expanded to a total of eleven VWs in 1981. Original VW 3 and VW 4 were replaced with off-set wells in 1991 and 1992. By the late 1990s, the City had experienced loss of vertical well yield due to riverbed degradation and associated losses in aquifer saturated thickness. This loss of vertical well yield led to the installation of HCWs to meet increasing demands. Between 1998 and 2004, the City installed four HCWs; quadrupling the City’s peak raw water production capacity. With installation of HCW 2 within the vertical well field, VWs 1, 2, and 8 were abandoned per an agreement with the Kansas Division of Water Resources (DWR). Table 2-1 and Table 2-2 present a summary of the vertical and HCWs, respectively.
Olathe, Kansas
Water Master Plan Update

Figure 2-2. Water Supply Facilities
### Table 2-1. Existing Raw Water Supply Infrastructure Summary – Vertical Wells

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Year Installed</th>
<th>Nominal Production Potential ¹ (MGD)</th>
<th>Pump Manufacturer</th>
<th>Rated Capacity (gpm)</th>
<th>Rated Head (ft)</th>
<th>Motor (HP)</th>
<th>Speed (RPM)</th>
<th>Drive Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>VW 3R ²</td>
<td>1991</td>
<td>0.9</td>
<td>Sulzer</td>
<td>735</td>
<td>356</td>
<td>75</td>
<td>3450</td>
<td>Constant Speed</td>
</tr>
<tr>
<td>VW 4R</td>
<td>1992</td>
<td>0.9</td>
<td>Fairbanks Morse</td>
<td>555</td>
<td>390</td>
<td>75</td>
<td>1770</td>
<td>Constant Speed</td>
</tr>
<tr>
<td>VW 5</td>
<td>1976</td>
<td>0.8</td>
<td>Goulds</td>
<td>700</td>
<td>318</td>
<td>75</td>
<td>3525</td>
<td>Constant Speed</td>
</tr>
<tr>
<td>VW 6 ³</td>
<td>1976</td>
<td>1.32</td>
<td>Goulds</td>
<td>900</td>
<td>366</td>
<td>125</td>
<td>1770</td>
<td>Constant Speed</td>
</tr>
<tr>
<td>VW 7 ⁴</td>
<td>1978</td>
<td>0.9</td>
<td>Sulzer</td>
<td>730</td>
<td>464</td>
<td>100</td>
<td>3450</td>
<td>Constant Speed</td>
</tr>
<tr>
<td>VW 9</td>
<td>1981</td>
<td>0.8</td>
<td>Goulds</td>
<td>500</td>
<td>390</td>
<td>75</td>
<td>1770</td>
<td>Constant Speed</td>
</tr>
<tr>
<td>VW 10</td>
<td>1981</td>
<td>0.7</td>
<td>Goulds</td>
<td>500</td>
<td>390</td>
<td>75</td>
<td>1770</td>
<td>Constant Speed</td>
</tr>
<tr>
<td>VW 11</td>
<td>1981</td>
<td>0.7</td>
<td>American-Marsh</td>
<td>500</td>
<td>291</td>
<td>60</td>
<td>1780</td>
<td>Constant Speed</td>
</tr>
</tbody>
</table>

¹ As indicated by original well production records. Does not account for reduction in capacity due to pumping interference from adjacent wells, reduction in capacity due to Kansas River stage, or condition of existing wells. These factors will be taken into account and an opinion of the actual, reliable capacity of the well field will be given in Section 4, Raw Water Supply Assessment.

² A 3-stage pump with impeller size “A” shown on the manufacturer’s pump curve. Pump rating assumed to be at the best efficiency point.

³ VW 6 is located within 75 feet of HCW 2 and is expected to have reduced capacity when run concurrently with HCW 2.

⁴ A 4-stage pump with impeller size “A” shown on the manufacturer’s pump curve. Pump rating assumed to be at the best efficiency point.
### Table 2-2. Existing Raw Water Supply Infrastructure Summary – Horizontal Collector Wells

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Year Installed</th>
<th>Nominal Production Potential (^1) (MGD)</th>
<th>Pump ID</th>
<th>Pump Manufacturer</th>
<th>Rated Capacity (MGD)</th>
<th>Rated Head (ft)</th>
<th>Motor (HP)</th>
<th>Speed (RPM)</th>
<th>Drive Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 1</td>
<td>1998</td>
<td>11.0</td>
<td>P-1</td>
<td>Ingersoll-Dresser</td>
<td>5.0</td>
<td>365</td>
<td>400</td>
<td>1780</td>
<td>VFD</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>P-2</td>
<td>Ingersoll-Dresser</td>
<td>5.0</td>
<td>365</td>
<td>400</td>
<td>1780</td>
<td>VFD</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>P-3</td>
<td>Ingersoll-Dresser</td>
<td>5.0</td>
<td>365</td>
<td>400</td>
<td>1780</td>
<td>Constant Speed</td>
</tr>
<tr>
<td>HCW 2</td>
<td>2002</td>
<td>3.5</td>
<td>P-1</td>
<td>Sterling Fluid Systems</td>
<td>3.5</td>
<td>377</td>
<td>300</td>
<td>1785</td>
<td>VFD</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>P-2</td>
<td>Sterling Fluid Systems</td>
<td>5.0</td>
<td>376</td>
<td>400</td>
<td>1775</td>
<td>Constant Speed</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>P-3</td>
<td>Sterling Fluid Systems</td>
<td>3.5</td>
<td>377</td>
<td>300</td>
<td>1785</td>
<td>VFD</td>
</tr>
<tr>
<td>HCW 3</td>
<td>2004</td>
<td>5.5</td>
<td>P-1</td>
<td>Flowserve</td>
<td>3.5</td>
<td>395</td>
<td>300</td>
<td>1785</td>
<td>VFD</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>P-2</td>
<td>Flowserve</td>
<td>4.0</td>
<td>395</td>
<td>350</td>
<td>1785</td>
<td>Constant Speed</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>P-3</td>
<td>Flowserve</td>
<td>3.5</td>
<td>395</td>
<td>300</td>
<td>1785</td>
<td>VFD</td>
</tr>
<tr>
<td>HCW 4</td>
<td>2005</td>
<td>6.0</td>
<td>P-1</td>
<td>Flowserve</td>
<td>3.5</td>
<td>395</td>
<td>300</td>
<td>1775</td>
<td>VFD</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>P-2</td>
<td>Flowserve</td>
<td>4.0</td>
<td>395</td>
<td>350</td>
<td>1775</td>
<td>Constant Speed</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>P-3</td>
<td>Flowserve</td>
<td>3.5</td>
<td>395</td>
<td>300</td>
<td>1775</td>
<td>VFD</td>
</tr>
</tbody>
</table>

\(^1\) As indicated by original well production records. Does not account for reduction in capacity due to pumping interference from adjacent wells, reduction in capacity due to Kansas River stage, or condition of existing wells. These factors will be taken into account and an opinion of the actual, reliable capacity of the well field will be given in Section 4, Raw Water Supply Assessment.
In the State of Kansas, water use is regulated by the DWR and generally requires a water appropriation in order to withdraw water. Each water appropriation allows for a total maximum amount of water to be withdrawn per calendar year and has a maximum instantaneous pumping rate associated with each well authorized under the appropriation. In instances, such as with the City, when additional water appropriations are added as water demand increases and new wells are added, the appropriation may be authorized for the full pumping capacity of the well, but be limited so that when combined with previously authorized appropriations the combined annual quantity and instantaneous rate does not exceed the City’s projected demand for a 40-year planning horizon. With each new appropriation, the City has a perfection period of up to 40 years to prove it can put the appropriated water to beneficial use. As the City provides evidence that it has put the appropriated amounts of water to beneficial use, the water appropriation becomes certified.

Table 2-3 presents a summary of the City’s current municipal use water appropriations associated with the wells listed in Table 2-1 and Table 2-2. The water appropriation for the VWs is for groundwater use. Each HCW requires both a groundwater and a surface water use appropriation. Water appropriation numbers 47,202 and 47,203 are for HCW 5, which has not yet been constructed.

The City’s membership in the Kansas River Water Assurance District (KRWAD) #1 provides for storage releases from Tuttle Creek, Milford, and Perry reservoirs. These managed reservoir releases help ensure, but do not guarantee, sufficient river flow adjacent to the well field during periods of low river flow. The KRWAD #1 is a legal entity established in 1987 composed of municipal and industrial water right holders along the Kansas River Basin. The DWR Chief Engineer has determined these water right holders will benefit from in-stream flow supplementation.

The KRWAD #1 was created to contract with the Kansas Water Office (KWO) to purchase storage in federal reservoirs allocated to municipal and industrial water supply. Releases from storage are made by the U.S. Army Corps of Engineers (USACE) according to operations agreements with the KWO and KRWAD to maintain target in-stream flows at Topeka and De Soto river gages. Target minimum flows at De Soto vary from 700 to 1,000 cubic feet per second (cfs) depending on season and water level in Tuttle Creek Reservoir.
Table 2-3. Existing Water Right Summary

<table>
<thead>
<tr>
<th>Water Right Number</th>
<th>Priority Date</th>
<th>Well</th>
<th>Water Source</th>
<th>Water Right Status</th>
<th>Annual Quantity (AF/yr)</th>
<th>Limited Annual Quantity (AF/yr)</th>
<th>Rate (gpm)</th>
<th>Limited Rate (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10,042</td>
<td>5/18/1964</td>
<td>VW 3R</td>
<td>Ground</td>
<td>Certified</td>
<td>6,094.559</td>
<td>6,094.559</td>
<td>730</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 4R</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>775</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1045</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1035</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1005</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>830</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>770</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>800</td>
<td></td>
</tr>
<tr>
<td>42,541</td>
<td>11/21/1996</td>
<td>HCW 1</td>
<td>Ground</td>
<td>Certified</td>
<td>674.848</td>
<td>674.848</td>
<td>10,500</td>
<td>10,500</td>
</tr>
<tr>
<td>42,542</td>
<td>11/21/1996</td>
<td>Surface</td>
<td>Certified</td>
<td></td>
<td>6,825.205</td>
<td>6,825.205</td>
<td>10,500</td>
<td></td>
</tr>
<tr>
<td>44,613</td>
<td>4/9/2001</td>
<td>HCW 2</td>
<td>Ground</td>
<td>Not Perfected</td>
<td>525.001</td>
<td>525.001</td>
<td>7,000</td>
<td></td>
</tr>
<tr>
<td>44,614</td>
<td>4/9/2001</td>
<td>Surface</td>
<td>Not Perfected</td>
<td></td>
<td>4,725</td>
<td>4,725</td>
<td>7,000</td>
<td></td>
</tr>
<tr>
<td>45,648</td>
<td>9/8/2003</td>
<td>HCW 3</td>
<td>Surface</td>
<td>Not Perfected</td>
<td>4,725</td>
<td>776.53</td>
<td>7,000</td>
<td></td>
</tr>
<tr>
<td>45,649</td>
<td>9/8/2003</td>
<td>Ground</td>
<td>Not Perfected</td>
<td></td>
<td>525.001</td>
<td>0</td>
<td>7,000</td>
<td></td>
</tr>
<tr>
<td>45,993</td>
<td>7/15/2004</td>
<td>HCW 4</td>
<td>Ground</td>
<td>Not Perfected</td>
<td>399.999</td>
<td>399.999</td>
<td>4,500</td>
<td></td>
</tr>
<tr>
<td>45,994</td>
<td>7/15/2004</td>
<td>Surface</td>
<td>Not Perfected</td>
<td></td>
<td>6,800.001</td>
<td>1,909.308</td>
<td>4,500</td>
<td></td>
</tr>
<tr>
<td>47,202</td>
<td>11/18/2008</td>
<td>HCW 5</td>
<td>Ground</td>
<td>Not Completed</td>
<td>299.999</td>
<td>299.999</td>
<td>9,500</td>
<td></td>
</tr>
<tr>
<td>47,203</td>
<td>11/18/2008</td>
<td>Surface</td>
<td>Not Completed</td>
<td></td>
<td>7,000.009</td>
<td>7,000.009</td>
<td>9,500</td>
<td></td>
</tr>
</tbody>
</table>

1 “Certified” appropriations are those where use amounts have been proven or perfected, and the quantity is set. “Not perfected” appropriations are ones that need to prove the initial appropriated amount within a 40-year period, in which they will become certified for the appropriation amount or the highest use, whichever is lower. “Not Completed” appropriations are those where the well has not been put online.

2 Annual quantity allowed with the water appropriation.

3 Effective total annual quantity gained with this appropriation when combined with previous existing appropriations.

4 Maximum instantaneous pumping rate allowed with the water appropriation. In appropriations covering multiple wells, maximum instantaneous rate has been divided between authorized wells.

5 Effective total maximum instantaneous pumping rate when combined with previous existing appropriations.

A network of raw water transmission lines convey water from the well field to WTP2 (see Figure 2-2). HCWs 1, 3, and 4, on the west end of the well field, combine into two transmission mains: a 30-inch ductile iron pipeline constructed in 1998 for HWC 1 and a 48-inch steel pipeline constructed in 2009. The two transmission mains interconnect on the west side of the well field near the HCWs.

The vertical well field transmission main is steel and was constructed in 1964. The main ranges from 12-inch to 24-inch; transition to the 24-inch main occurs downstream of the former VW 1 tie-in. Smaller 8-inch and 10-inch lines connect individual wells to the transmission line.
HCW 2 discharges into a 24-inch ductile iron transmission line constructed in 2004. This line ties in with the 24-inch transmission main from the vertical well field on the southeast end of the well field.

Table 2-4 describes the piping that comprises the raw water transmission system.

Table 2-4. Raw Water Pipe Summary

<table>
<thead>
<tr>
<th>Pipe Diameter (in)</th>
<th>Length (ft)</th>
<th>Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>1,761</td>
<td>Ductile iron/steel</td>
</tr>
<tr>
<td>10</td>
<td>1,688</td>
<td>Ductile iron/steel</td>
</tr>
<tr>
<td>12</td>
<td>722</td>
<td>Steel</td>
</tr>
<tr>
<td>14</td>
<td>570</td>
<td>Steel</td>
</tr>
<tr>
<td>16</td>
<td>579</td>
<td>Steel</td>
</tr>
<tr>
<td>18</td>
<td>226</td>
<td>Steel</td>
</tr>
<tr>
<td>24</td>
<td>9,640</td>
<td>Ductile iron/steel</td>
</tr>
<tr>
<td>30</td>
<td>15,680</td>
<td>Ductile iron</td>
</tr>
<tr>
<td>48</td>
<td>15,170</td>
<td>Steel</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>46,036</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>8.7 mi</strong></td>
<td></td>
</tr>
</tbody>
</table>
2.3 Water Treatment

Water Treatment Plant 2 (WTP2) is the only water plant currently in operation. WTP1, which treated water from Lake Olathe, has not been in use since 2005. WTP2 treats groundwater under the influence of surface water from the vertical and HCWs. The treatment plant includes aeration (Basins 1 and 3 only), lime softening (Basins 1 through 3) or enhanced coagulation (Basin 4), granular media or membrane filtration, and chlorine disinfection.

Basins 1, 2 and 3 provide single-stage softening. Basin 4 provides coagulation and flocculation and is operated in parallel with Basins 1, 2, and 3. Table 2-5 shows the rated capacity of each process at the Plant.

Table 2-5. WTP2 Treatment Process Capacities

<table>
<thead>
<tr>
<th>Process</th>
<th>Rated Capacity (MGD)</th>
<th>Total Rated Capacity (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lime Softening</td>
<td>26.85</td>
<td>38.3</td>
</tr>
<tr>
<td>Enhanced Coagulation</td>
<td>11.45</td>
<td></td>
</tr>
<tr>
<td>Granular Media Filters</td>
<td>19.35</td>
<td>32.35</td>
</tr>
<tr>
<td>Membrane Filters</td>
<td>13</td>
<td></td>
</tr>
</tbody>
</table>


Settled water from the softening basins and coagulation basin combines and is split to either conventional granular media filters or membrane filters. Treated water from the granular media filters flows to a below-grade clearwell and water from the membrane filters flows to an above-grade clearwell. Each clearwell has a capacity of 1 MG. Table 2-6 provides a summary of the clearwells. Both clearwells are baffled to achieve the required disinfection contact time following the application of free chlorine. Downstream of the clearwells ammonia is injected to form chloramines for secondary disinfection within the distribution system.

Table 2-6. WTP2 Clearwell Storage Summary

<table>
<thead>
<tr>
<th>Clearwell ID</th>
<th>Treated Water Source</th>
<th>Year</th>
<th>Overflow Elevation (ft)</th>
<th>Base Elevation (ft)</th>
<th>Capacity (MG)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below Grade</td>
<td>Granular media filter effluent</td>
<td>1964</td>
<td>1003.90</td>
<td>990.00</td>
<td>1.0</td>
</tr>
<tr>
<td>Above grade</td>
<td>Membrane filter effluent</td>
<td>2005</td>
<td>1042.10</td>
<td>1011.50</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total 2.0</td>
</tr>
</tbody>
</table>

1 Baffled and currently used to achieve required disinfection contact time.

Three high service pump stations (HSPS) are used to pump finished water from the clearwells to the distribution system. Pump capacities are shown in Table 2-7. The North and South HSPS pump water from the below-grade clearwell and the Membrane HSPS pumps water from the above-grade clearwell.
<table>
<thead>
<tr>
<th>Pump ID</th>
<th>Pump Type</th>
<th>Year Installed</th>
<th>Pump Manufacturer</th>
<th>Rated Flow (gpm)</th>
<th>Rated Head (ft)</th>
<th>Motor (HP)</th>
<th>Speed (RPM)</th>
<th>Drive Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-2</td>
<td>Vertical Turbine</td>
<td>1964</td>
<td>Fairbanks Morse</td>
<td>1,400</td>
<td>338</td>
<td>150</td>
<td>1,770</td>
<td>VFD</td>
</tr>
<tr>
<td>P-3</td>
<td>Vertical Turbine</td>
<td>1964</td>
<td>Fairbanks Morse</td>
<td>1,400</td>
<td>338</td>
<td>150</td>
<td>1,770</td>
<td>VFD</td>
</tr>
<tr>
<td>P-4</td>
<td>Vertical Turbine</td>
<td>2015</td>
<td>Fairbanks Morse</td>
<td>2,800</td>
<td>290</td>
<td>250</td>
<td>1,770</td>
<td>VFD</td>
</tr>
<tr>
<td>P-5</td>
<td>Vertical Turbine</td>
<td>2015</td>
<td>Fairbanks Morse</td>
<td>2,800</td>
<td>290</td>
<td>250</td>
<td>1,770</td>
<td>VFD</td>
</tr>
</tbody>
</table>

**South High Service Pump Station**

**Firm Capacity**  
5,600

<table>
<thead>
<tr>
<th>Pump ID</th>
<th>Pump Type</th>
<th>Year Installed</th>
<th>Pump Manufacturer</th>
<th>Rated Flow (gpm)</th>
<th>Rated Head (ft)</th>
<th>Motor (HP)</th>
<th>Speed (RPM)</th>
<th>Drive Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-6</td>
<td>Vertical Turbine</td>
<td>1998</td>
<td>Fairbanks Morse</td>
<td>3,472</td>
<td>185</td>
<td>400</td>
<td>1,785</td>
<td>VFD</td>
</tr>
<tr>
<td>P-7</td>
<td>Vertical Turbine</td>
<td>1998</td>
<td>Fairbanks Morse</td>
<td>3,472</td>
<td>185</td>
<td>400</td>
<td>1,785</td>
<td>VFD</td>
</tr>
<tr>
<td>P-8</td>
<td>Vertical Turbine</td>
<td>1998</td>
<td>Fairbanks Morse</td>
<td>3,472</td>
<td>185</td>
<td>400</td>
<td>1,785</td>
<td>VFD</td>
</tr>
</tbody>
</table>

**North High Service Pump Station**

**Firm Capacity**  
6,944

<table>
<thead>
<tr>
<th>Pump ID</th>
<th>Pump Type</th>
<th>Year Installed</th>
<th>Pump Manufacturer</th>
<th>Rated Flow (gpm)</th>
<th>Rated Head (ft)</th>
<th>Motor (HP)</th>
<th>Speed (RPM)</th>
<th>Drive Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-9</td>
<td>Horizontal Split Case</td>
<td>2005</td>
<td>Fairbanks Morse</td>
<td>5,373</td>
<td>280</td>
<td>500</td>
<td>1,775</td>
<td>VFD</td>
</tr>
<tr>
<td>P-10</td>
<td>Horizontal Split Case</td>
<td>2005</td>
<td>Fairbanks Morse</td>
<td>5,373</td>
<td>280</td>
<td>500</td>
<td>1,775</td>
<td>VFD</td>
</tr>
<tr>
<td>P-11</td>
<td>Horizontal Split Case</td>
<td>2005</td>
<td>Fairbanks Morse</td>
<td>3,681</td>
<td>281</td>
<td>500</td>
<td>1,790</td>
<td>VFD</td>
</tr>
</tbody>
</table>

**Membrane High Service Pump Station**

**Firm Capacity**  
9,054

1 Pump motors were rebuilt in 2015.

2 Firm capacity assumes largest pump is out of service.
2.4 Water Distribution

The high service pumps supply treated water from WTP2 to a network of pipelines, storage tanks, and pump stations, and ultimately the City’s customers. Figure 2-3 shows key facilities within the water distribution system.

The water distribution system is divided into two pressure zones. The Main pressure zone receives water directly from WTP2 and serves the area east of I-35 and north of Santa Fe and the entire area west of I-35; this area is approximately 65-percent of the system. Elevations in this zone range from 785 feet to 1,085 feet. The system is pressurized by the HSPS at WTP2 in the northwest part of the zone, which pump to a gradient of between 1,270 ft (111 pound per square inch [psi] based on a reference elevation of 1,014.50 ft on the discharge side of the HSPS) and 1,340 ft (141 psi). The gradient at the southeastern part of the zone, at the Curtis Street Reservoir and Pump Station, is maintained between approximately 1,177 feet and 1,225 feet (65 to 85 psi based on a reference elevation of 1,027.50 ft). Figure 2-4 shows the hydraulic gradient of the system. In addition to the HSPS, the Hedge Lane Pump Station and the Curtis Street Pump Station are used to maintain the system pressure as measured at the inlet to the Curtis Street Reservoirs.

The Southeast pressure zone is pressurized by the Black Bob 2 Pump Station, which receives water from the Main pressure zone through a 36-inch transmission line. Elevations in this zone range from 990 feet to 1,105 feet. The largest pump at the Black Bob 2 Pump Station can lift the gradient to approximately 1,360 feet (120 psi at pump discharge elevation of 1,084.50 ft); whereas the smallest pump lifts the gradient to 1,222 feet (60 psi). However, operations staff typically target a gradient of 1,210 (55 psi) to 1,235 (65 psi) feet leaving the Pump Station.

The system has built-in redundancy such that the two pressure zones can supply each other. Two control valve stations exist between the Main zone and the Southeast zone where distribution mains cross I-35. Each station contains a 12-inch electrically-actuated ball valve and a 4-inch control valve. The ball valves open upon a low pressure set point on the Southeast side to convey water from the Main zone to the Southeast zone. Additionally, piping and valves exist to allow the Black Bob 2 Pump Station to pump water from the Black Bob tanks through the 36-inch transmission main to the Main pressure zone; however, the City has not used this connection and have not verified if it is viable.

Table 2-8 summarizes the capacities of pump stations within the distribution system. Table 2-9 summarizes the capacities of the distribution system storage tanks.
Figure 2-3.
Water Distribution System Key Facilities

Olathe, Kansas
Water Master Plan Update
City of Lenexa
City of Overland Park
City of Gardner

Map Feature Key
- Current City Limits
- Olathe Water Service Area
- Water Facilities
- Pressure Zone Boundary
- Pressure Control Station

Water Mains Transmission
- 16" - 18"
- 20"
- 24"
- 30"
- 36"
- 42"
- 48"

Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Olathe, Kansas
Olathe Water Master Plan Update

WATER DISTRIBUTION SYSTEM HYDRAULIC GRADE LINE

FIG 2-4
Table 2-8. Distribution Pumping Summary

<table>
<thead>
<tr>
<th>Pump ID</th>
<th>Pump Type</th>
<th>Year</th>
<th>Pump Manufacturer</th>
<th>Rated flow (GPM)</th>
<th>Rated Head (Feet)</th>
<th>Motor (HP)</th>
<th>Speed (rpm)</th>
<th>Drive Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-1</td>
<td>Horizontal Split Case</td>
<td>1984</td>
<td>Aurora Pump</td>
<td>2,100</td>
<td>140</td>
<td>100</td>
<td>1,800</td>
<td>VFD</td>
</tr>
<tr>
<td>P-2</td>
<td>Horizontal Split Case</td>
<td>1984</td>
<td>Aurora Pump</td>
<td>2,800</td>
<td>136</td>
<td>125</td>
<td>1,800</td>
<td>VFD</td>
</tr>
<tr>
<td>P-3</td>
<td>Horizontal Split Case</td>
<td>1984</td>
<td>Aurora Pump</td>
<td>2,800</td>
<td>136</td>
<td>125</td>
<td>1,800</td>
<td>VFD</td>
</tr>
<tr>
<td>P-4</td>
<td>Horizontal Split Case</td>
<td>1990</td>
<td>Aurora Pump</td>
<td>5,500</td>
<td>120</td>
<td>200</td>
<td>1,750</td>
<td>VFD</td>
</tr>
<tr>
<td></td>
<td><strong>Firm Capacity</strong> 1</td>
<td></td>
<td></td>
<td>7,700</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-6</td>
<td>Vertical Turbine</td>
<td>1969</td>
<td>Byron Jackson</td>
<td>1,400</td>
<td>190</td>
<td>150</td>
<td>1,770</td>
<td>VFD</td>
</tr>
<tr>
<td>P-7</td>
<td>Vertical Turbine</td>
<td>1969</td>
<td>Byron Jackson</td>
<td>2,100</td>
<td>200</td>
<td>100</td>
<td>1,770</td>
<td>VFD</td>
</tr>
<tr>
<td></td>
<td><strong>Firm Capacity</strong> 1</td>
<td></td>
<td></td>
<td>1,400</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-1</td>
<td>Horizontal Split Case</td>
<td>2001</td>
<td>Patterson Pump</td>
<td>4,930</td>
<td>335</td>
<td>600</td>
<td>1,780</td>
<td>VFD</td>
</tr>
<tr>
<td>P-2</td>
<td>Horizontal Split Case</td>
<td>2001</td>
<td>Patterson Pump</td>
<td>4,930</td>
<td>335</td>
<td>600</td>
<td>1,780</td>
<td>VFD</td>
</tr>
<tr>
<td>P-3</td>
<td>Horizontal Split Case</td>
<td>2001</td>
<td>Patterson Pump</td>
<td>4,930</td>
<td>335</td>
<td>600</td>
<td>1,780</td>
<td>VFD</td>
</tr>
<tr>
<td></td>
<td><strong>Firm Capacity</strong> 1</td>
<td></td>
<td></td>
<td>9,860</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-1</td>
<td>Vertical Turbine</td>
<td>1981</td>
<td>Worthington</td>
<td>1,040</td>
<td>40</td>
<td>15</td>
<td>1,760</td>
<td>Constant</td>
</tr>
<tr>
<td>P-2</td>
<td>Vertical Turbine</td>
<td>1981</td>
<td>Worthington</td>
<td>1,400</td>
<td>50</td>
<td>25</td>
<td>1,760</td>
<td>Constant</td>
</tr>
<tr>
<td></td>
<td><strong>Firm Capacity</strong> 1</td>
<td></td>
<td></td>
<td>1,040</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-1</td>
<td>Horizontal Split Case</td>
<td>1991</td>
<td>Fairbanks Morse</td>
<td>4,924</td>
<td>70</td>
<td>125</td>
<td>1,185</td>
<td>VFD</td>
</tr>
<tr>
<td>P-2</td>
<td>Horizontal Split Case</td>
<td>1991</td>
<td>Fairbanks Morse</td>
<td>5,804</td>
<td>111</td>
<td>250</td>
<td>1,185</td>
<td>VFD</td>
</tr>
<tr>
<td>P-3</td>
<td>Horizontal Split Case</td>
<td>1991</td>
<td>Fairbanks Morse</td>
<td>5,804</td>
<td>111</td>
<td>250</td>
<td>1,185</td>
<td>VFD</td>
</tr>
<tr>
<td>P-4</td>
<td>Horizontal Split Case</td>
<td>1999</td>
<td>Fairbanks Morse</td>
<td>3,900</td>
<td>208</td>
<td>250</td>
<td>1,785</td>
<td>VFD</td>
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<tr>
<td></td>
<td><strong>Firm Capacity</strong> 1</td>
<td></td>
<td></td>
<td>14,628</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 Firm capacity assumes largest pump is out of service.
2 Black Bob 1 Pump Station is not currently used.
3 Black Bob 2 P-1 and P-2 are capable of pumping water from the Black Bob tanks to the Main zone.
### Table 2-9. Distribution Storage Summary

<table>
<thead>
<tr>
<th>Tank ID</th>
<th>Type of Storage</th>
<th>Year Constructed</th>
<th>Overflow Elevation (ft)</th>
<th>Base Elevation (ft)</th>
<th>Head Range (ft)</th>
<th>Capacity (MG)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Main Pressure Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hedge Lane</td>
<td>Ground</td>
<td>2001</td>
<td>1025</td>
<td>985.00</td>
<td>0-40</td>
<td>6.0</td>
</tr>
<tr>
<td>Curtis Street 1</td>
<td>Buried</td>
<td>1950s/1969</td>
<td>1012.75</td>
<td>999.00</td>
<td>0-13.75</td>
<td>1.0</td>
</tr>
<tr>
<td>Renner</td>
<td>Standpipe</td>
<td>1984</td>
<td>1188</td>
<td>1061.00</td>
<td>0-127</td>
<td>1.5</td>
</tr>
<tr>
<td><strong>Total Storage Capacity – Main Pressure Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>8.5</strong></td>
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<tr>
<td><strong>Southeast Pressure Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black Bob No. 1</td>
<td>Standpipe</td>
<td>1981</td>
<td>1194.50</td>
<td>1094.50</td>
<td>0-100</td>
<td>1.5</td>
</tr>
<tr>
<td>Black Bob No. 2</td>
<td>Standpipe</td>
<td>1992</td>
<td>1209</td>
<td>1094.50</td>
<td>0-114.5</td>
<td>5.0</td>
</tr>
<tr>
<td><strong>Total Storage Capacity – Southeast Pressure Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>6.5</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Summary</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total System Storage without WTP2 Clearwells</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>15.0</strong></td>
</tr>
<tr>
<td>Total System Storage with WTP2 Clearwells 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>17.0</strong></td>
</tr>
</tbody>
</table>
### Table 2-10. Water Main Length Summary

<table>
<thead>
<tr>
<th>Nominal Diameter (in)</th>
<th>Length (ft)</th>
<th>Nominal Diameter (in)</th>
<th>Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Distribution</td>
<td>Transmission</td>
<td></td>
</tr>
<tr>
<td>4-in</td>
<td>43,334</td>
<td>16-in</td>
<td>46,461</td>
</tr>
<tr>
<td>6-in</td>
<td>1,236,480</td>
<td>18-in</td>
<td>5,448</td>
</tr>
<tr>
<td>8-in</td>
<td>948,978</td>
<td>20-in</td>
<td>713</td>
</tr>
<tr>
<td>10-in</td>
<td>8,593</td>
<td>24-in</td>
<td>45,338</td>
</tr>
<tr>
<td>12-in</td>
<td>508,928</td>
<td>30-in</td>
<td>61,731</td>
</tr>
<tr>
<td></td>
<td></td>
<td>36-in</td>
<td>39,092</td>
</tr>
<tr>
<td></td>
<td></td>
<td>42-in</td>
<td>36,355</td>
</tr>
<tr>
<td></td>
<td></td>
<td>48-in</td>
<td>2,111</td>
</tr>
<tr>
<td>Total</td>
<td>2,746,313</td>
<td></td>
<td>237,249</td>
</tr>
<tr>
<td>Total</td>
<td>520 mi</td>
<td></td>
<td>45 mi</td>
</tr>
</tbody>
</table>

### Table 2-11. Water Main Material Summary

<table>
<thead>
<tr>
<th>Material</th>
<th>Length (ft)</th>
<th>Percent of System</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asbestos Cement</td>
<td>9,992</td>
<td>0.3%</td>
</tr>
<tr>
<td>Cast Iron</td>
<td>765,346</td>
<td>25.7%</td>
</tr>
<tr>
<td>Ductile Iron</td>
<td>2,102,481</td>
<td>70.5%</td>
</tr>
<tr>
<td>HDPE</td>
<td>31,847</td>
<td>1.1%</td>
</tr>
<tr>
<td>PVC</td>
<td>25,512</td>
<td>0.9%</td>
</tr>
<tr>
<td>Steel</td>
<td>34,874</td>
<td>1.2%</td>
</tr>
<tr>
<td>Unknown</td>
<td>13,510</td>
<td>0.5%</td>
</tr>
<tr>
<td>Total (ft)</td>
<td>2,983,570</td>
<td></td>
</tr>
<tr>
<td>Total (mi)</td>
<td>565</td>
<td></td>
</tr>
</tbody>
</table>
Figure 2-5. Water Distribution System Mains By Diameter
Figure 2-6.
Water Distribution System By Age

Olathe, Kansas
Water Master Plan Update

<table>
<thead>
<tr>
<th>Installation Decade</th>
<th>Color</th>
</tr>
</thead>
<tbody>
<tr>
<td>1940s or Earlier</td>
<td>1980 - 1989</td>
</tr>
<tr>
<td>1950 - 1959</td>
<td>1990 - 1999</td>
</tr>
<tr>
<td>1970 - 1979</td>
<td>2010 or Newer</td>
</tr>
</tbody>
</table>

Map Feature Key
- Current City Limits
- Olathe Water Service Area
- Pressure Zone Boundary

Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
Figure 2-7. Water Distribution System By Material

Olathe, Kansas

Water Master Plan Update

Pressure Zone Boundary

Olathe Water Service Area

Map Feature Key

- Current City Limits
- Olathe Water Service Area
- Pressure Zone Boundary

Water Mains MATERIAL

- Unknown / Other
- ASBESTOS CONCRETE
- HDPE
- PVC
- STEEL

Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
2.5 Summary

Table 2-11 summarizes the capacities of the water distribution system.

Table 2-12. Summary of Key System Capacities

<table>
<thead>
<tr>
<th>Water Supply</th>
<th>Average</th>
<th>MGD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Rights 1</td>
<td>19.6</td>
<td>MGD</td>
</tr>
<tr>
<td>Well Field Firm Capacity</td>
<td>30.7</td>
<td>MGD</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Water Treatment</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Softening/Coagulation</td>
<td>38.3</td>
<td>MGD</td>
</tr>
<tr>
<td>Filtration</td>
<td>32.4</td>
<td>MGD</td>
</tr>
<tr>
<td>Clearwell Storage</td>
<td>2.0</td>
<td>MG</td>
</tr>
<tr>
<td>High Service Pumping</td>
<td>40.0</td>
<td>MGD</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Water Distribution</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Booster Pumping</td>
<td>48.0</td>
<td>MGD</td>
</tr>
<tr>
<td>Distribution Storage</td>
<td>15.0</td>
<td>MG</td>
</tr>
</tbody>
</table>

1 Maximum Available Water Rights is 49.3 MGD
3 Demand Projections

Determination of future water use needs is the foundation for development and implementation of a comprehensive capital improvements plan for the City’s water system. This Section provides updated water use (demand) projections and the basis for those projections.

Two methods for projection of future water demands will be compared:
- Population-based projections using historical water use trends; and
- Land-use-based projections using City’s future land-use plan and established unit demands.

3.1 Historical Water Use Summary

Water system demands vary on an hourly, daily, and seasonal basis. Variations of water consumption can best be explained by such factors as weather patterns, social patterns, economic factors, and technological advances. Because of the unique water characteristics of a community, historical system operating records typically serve as the basis for predicting future water requirements. The City’s annual municipal water use reports to the DWR from 2001 through 2014 and the City’s daily system demand records were used in this Study. The annual DWR reports divide municipal water requirements into five categories:
- Domestic consumption (generally residential, commercial, and institutional uses)
- Industrial consumption
- Wholesale supply to other public water suppliers
- Non-revenue water for public use that is metered and provided for free
- Unaccounted for water (losses that occur within the plant or the distribution system that are not metered and are generally unknown)

The historical water usage from 2001 through 2014 is summarized in Table 3-1. Water sold to residential and commercial customers account for approximately 75-percent of the total water produced over the period of record.

The efficiency of a water system is commonly measured by determining the average amount of water a customer uses per day, referred to as gallons per capita per day (gpcd). This measurement consists of residential and commercial water use, non-revenue water, and unaccounted-for water and is based on the population of the water service area; wholesale use and industrial use are not included in this value. As shown in Table 3-2, water use has ranged from 92 gpcd to 129 gpcd since 2001 with an average of 107 gpcd. During the period of 2009 to 2013, other public water suppliers that serve over 10,000 people within the same geographical area as the City of Olathe averaged 125 gpcd\(^1\); based on Table 3-2 for this same period, the City’s average remained 107 gpcd.

Table 3-1. Summary of Historical Annual Water Use (2001 – 2014)

<table>
<thead>
<tr>
<th>Year</th>
<th>Estimated Population Served</th>
<th>Raw Water Pumped from Water Rights</th>
<th>Wholesale Water Use</th>
<th>Industrial Water Use</th>
<th>Residential/ Commercial Water Use</th>
<th>Non-Revenue Water Use</th>
<th>Unaccounted For Water</th>
<th>Average Daily Supply (MGD)</th>
<th>Average Unaccounted-For Water (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2001</td>
<td>97,753</td>
<td>4,441,196</td>
<td>258,851</td>
<td>497,922</td>
<td>3,234,990</td>
<td>21,398</td>
<td>428,035</td>
<td>1.17</td>
<td>12.2</td>
</tr>
<tr>
<td>2002</td>
<td>99,226</td>
<td>4,801,451</td>
<td>4,876,631</td>
<td>278,430</td>
<td>3,640,100</td>
<td>25,100</td>
<td>663,561</td>
<td>1.82</td>
<td>13.4</td>
</tr>
<tr>
<td>2003</td>
<td>101,426</td>
<td>4,678,720</td>
<td>75,180</td>
<td>283,240</td>
<td>3,713,730</td>
<td>29,800</td>
<td>579,626</td>
<td>1.59</td>
<td>13.4</td>
</tr>
<tr>
<td>2004</td>
<td>103,657</td>
<td>4,359,030</td>
<td>202,030</td>
<td>256,120</td>
<td>3,640,100</td>
<td>27,200</td>
<td>611,516</td>
<td>1.82</td>
<td>13.4</td>
</tr>
<tr>
<td>2005</td>
<td>105,219</td>
<td>4,727,840</td>
<td>0</td>
<td>223,860</td>
<td>227,840</td>
<td>26,800</td>
<td>597,936</td>
<td>1.64</td>
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<tr>
<td>2006</td>
<td>107,245</td>
<td>5,048,350</td>
<td>0</td>
<td>239,540</td>
<td>239,540</td>
<td>26,300</td>
<td>634,078</td>
<td>1.74</td>
<td>13.8</td>
</tr>
<tr>
<td>2007</td>
<td>108,492</td>
<td>4,772,460</td>
<td>0</td>
<td>227,840</td>
<td>227,840</td>
<td>22,812</td>
<td>632,359</td>
<td>1.73</td>
<td>13.1</td>
</tr>
<tr>
<td>2008</td>
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<td>4,351,170</td>
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<td>209,620</td>
<td>19,759</td>
<td>525,601</td>
<td>1.44</td>
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</tr>
<tr>
<td>2009</td>
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<td>4,230,290</td>
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<td>189,936</td>
<td>22,959</td>
<td>652,535</td>
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<td>4,705,990</td>
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<td>185,934</td>
<td>185,934</td>
<td>27,212</td>
<td>786,331</td>
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<td>183,624</td>
<td>33,158</td>
<td>734,395</td>
<td>2.01</td>
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<tr>
<td>2012</td>
<td>113,509</td>
<td>5,915,450</td>
<td>3</td>
<td>206,554</td>
<td>206,554</td>
<td>43,500</td>
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<td>2013</td>
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<td>249,189</td>
<td>249,189</td>
<td>48,357</td>
<td>888,922</td>
<td>2.44</td>
<td>13.5</td>
</tr>
<tr>
<td>2014</td>
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<td>4,421,720</td>
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<td>213,599</td>
<td>213,599</td>
<td>72,445</td>
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</table>

Statistics of Historical Record

<table>
<thead>
<tr>
<th></th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw Water</td>
<td>0.70</td>
<td>0.82</td>
<td>1.05</td>
</tr>
<tr>
<td>Wholesale Water</td>
<td>0.45</td>
<td>0.67</td>
<td>1.36</td>
</tr>
<tr>
<td>Industrial Water</td>
<td>8.51</td>
<td>9.63</td>
<td>11.38</td>
</tr>
<tr>
<td>Non-Revenue Water</td>
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<td>0.09</td>
<td>0.20</td>
</tr>
<tr>
<td>Unaccounted For</td>
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<td>1.84</td>
<td>3.17</td>
</tr>
<tr>
<td>Water</td>
<td>11.6</td>
<td>13.0</td>
<td>16.2</td>
</tr>
</tbody>
</table>

1 Information provided by City of Olathe Planning Department.
2 Information from Municipal Water Use Reports submitted to the Kansas Division of Water Resources.
## Table 3-2. Average Per Capita Water Use Evaluation (2001 – 2014)

<table>
<thead>
<tr>
<th>Year</th>
<th>Estimated Water Service Area Population</th>
<th>Residential/Commercial Water Use Per Person (gpcd)</th>
<th>Non-Revenue Water Use Per Person (gpcd)</th>
<th>Unaccounted For Water Use Per Person (gpcd)</th>
<th>Total Water Use Per Person (gpcd)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2001</td>
<td>97,753</td>
<td>91</td>
<td>0.60</td>
<td>12.0</td>
<td>103</td>
</tr>
<tr>
<td>2002</td>
<td>99,226</td>
<td>101</td>
<td>0.69</td>
<td>18.3</td>
<td>120</td>
</tr>
<tr>
<td>2003</td>
<td>101,426</td>
<td>100</td>
<td>0.80</td>
<td>15.7</td>
<td>117</td>
</tr>
<tr>
<td>2004</td>
<td>103,657</td>
<td>87</td>
<td>0.72</td>
<td>16.2</td>
<td>104</td>
</tr>
<tr>
<td>2005</td>
<td>105,219</td>
<td>94</td>
<td>0.70</td>
<td>15.6</td>
<td>110</td>
</tr>
<tr>
<td>2006</td>
<td>107,245</td>
<td>96</td>
<td>0.67</td>
<td>16.2</td>
<td>113</td>
</tr>
<tr>
<td>2007</td>
<td>108,492</td>
<td>90</td>
<td>0.58</td>
<td>16.0</td>
<td>107</td>
</tr>
<tr>
<td>2008</td>
<td>112,019</td>
<td>81</td>
<td>0.48</td>
<td>12.9</td>
<td>95</td>
</tr>
<tr>
<td>2009</td>
<td>112,348</td>
<td>76</td>
<td>0.56</td>
<td>15.9</td>
<td>92</td>
</tr>
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<td>2010</td>
<td>112,685</td>
<td>83</td>
<td>1.14</td>
<td>19.1</td>
<td>104</td>
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<tr>
<td>2011</td>
<td>113,011</td>
<td>88</td>
<td>0.80</td>
<td>17.8</td>
<td>107</td>
</tr>
<tr>
<td>2012</td>
<td>113,509</td>
<td>100</td>
<td>1.05</td>
<td>27.9</td>
<td>129</td>
</tr>
<tr>
<td>2013</td>
<td>114,057</td>
<td>83</td>
<td>1.16</td>
<td>21.4</td>
<td>105</td>
</tr>
<tr>
<td>2014</td>
<td>114,346</td>
<td>79</td>
<td>1.74</td>
<td>12.1</td>
<td>93</td>
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</table>

### Statistics (Historical, 2001 – 2014)

<table>
<thead>
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<th></th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential/Commercial Water Use Per Person (gpcd)</td>
<td>76</td>
<td>89</td>
<td>101</td>
</tr>
<tr>
<td>Non-Revenue Water Use Per Person (gpcd)</td>
<td>0.48</td>
<td>0.84</td>
<td>1.74</td>
</tr>
<tr>
<td>Unaccounted For Water Use Per Person (gpcd)</td>
<td>12.0</td>
<td>16.9</td>
<td>27.9</td>
</tr>
<tr>
<td>Total Water Use Per Person (gpcd)</td>
<td>92</td>
<td>107</td>
<td>129</td>
</tr>
</tbody>
</table>

### Statistics (Last 10 Years, 2005 – 2014)

<table>
<thead>
<tr>
<th></th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential/Commercial Water Use Per Person (gpcd)</td>
<td>76</td>
<td>87</td>
<td>100</td>
</tr>
<tr>
<td>Non-Revenue Water Use Per Person (gpcd)</td>
<td>0.48</td>
<td>0.89</td>
<td>1.74</td>
</tr>
<tr>
<td>Unaccounted For Water Use Per Person (gpcd)</td>
<td>12.1</td>
<td>17.5</td>
<td>27.9</td>
</tr>
<tr>
<td>Total Water Use Per Person (gpcd)</td>
<td>92</td>
<td>105</td>
<td>129</td>
</tr>
</tbody>
</table>

The City’s daily system demand records for 2006 to 2014 were used to review seasonal variations in water use. Water supply and treatment systems are typically planned and designed for the maximum day demand while distribution storage volumes are designed to meet peak demands over the course of a day. However, it is recognized that in some years this demand can be isolated to a single day and the question arises of whether water supply and treatment systems can be planned for a demand that is less than the single “maximum day.” The City’s system demand records were reviewed for the extent of time that peak demands occur in order to answer this question. Table 3-3 presents a summary of the seasonal peak water use.
### Table 3-3. Summary of Seasonal Peak Water Use

<table>
<thead>
<tr>
<th>Year</th>
<th>Rainfall (in.)</th>
<th>Olathe WSA Population</th>
<th>Average Day Water Use (MGD)</th>
<th>Maximum Day</th>
<th>3-Day Maximum</th>
<th>7-Day Maximum</th>
<th>30-Day (Monthly) Maximum</th>
<th>Summer Seasonal Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1-Day Maximum (MGD)</td>
<td>3-Day Maximum (MGD)</td>
<td>7-Day Maximum (MGD)</td>
<td>30-Day Maximum (MGD)</td>
<td>Seasonal Average (MGD)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MD:AD Peaking Factor</td>
<td>Peaking Factor</td>
<td>Peaking Factor</td>
<td>Peaking Factor</td>
<td>Peaking Factor</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Difference from Max Day (MGD)</td>
<td>Difference from Max Day (MGD)</td>
<td>Difference from Max Day (MGD)</td>
<td>Difference from Max Day (MGD)</td>
<td>Difference from Max Day (MGD)</td>
</tr>
<tr>
<td>2006</td>
<td>32.88</td>
<td>107,245</td>
<td>13.8</td>
<td>24.6</td>
<td>1.78</td>
<td>24.5</td>
<td>1.77</td>
<td>-0.1</td>
</tr>
<tr>
<td>2007</td>
<td>45.14</td>
<td>108,492</td>
<td>13.1</td>
<td>25.6</td>
<td>1.96</td>
<td>23.0</td>
<td>1.76</td>
<td>-2.5</td>
</tr>
<tr>
<td>2008</td>
<td>47.44</td>
<td>112,019</td>
<td>11.9</td>
<td>22.0</td>
<td>1.84</td>
<td>21.5</td>
<td>1.80</td>
<td>-0.5</td>
</tr>
<tr>
<td>2009</td>
<td>47.62</td>
<td>112,348</td>
<td>11.6</td>
<td>20.5</td>
<td>1.77</td>
<td>18.9</td>
<td>1.63</td>
<td>-1.5</td>
</tr>
<tr>
<td>2010</td>
<td>44.92</td>
<td>112,685</td>
<td>12.9</td>
<td>25.5</td>
<td>1.98</td>
<td>22.3</td>
<td>1.73</td>
<td>-3.2</td>
</tr>
<tr>
<td>2011</td>
<td>32.47</td>
<td>113,011</td>
<td>13.5</td>
<td>25.9</td>
<td>1.92</td>
<td>25.0</td>
<td>1.85</td>
<td>-0.9</td>
</tr>
<tr>
<td>2012</td>
<td>26.48</td>
<td>113,509</td>
<td>16.2</td>
<td>28.1</td>
<td>1.73</td>
<td>27.3</td>
<td>1.68</td>
<td>-0.8</td>
</tr>
<tr>
<td>2013</td>
<td>34.08</td>
<td>114,057</td>
<td>13.5</td>
<td>28.6</td>
<td>2.12</td>
<td>28.0</td>
<td>2.08</td>
<td>-0.5</td>
</tr>
<tr>
<td>2014</td>
<td>32.69</td>
<td>114,346</td>
<td>12.1</td>
<td>24.1</td>
<td>1.99</td>
<td>23.4</td>
<td>1.93</td>
<td>-0.7</td>
</tr>
</tbody>
</table>

**Comparison Statistics**

Above-Average Rainfall Years (2007-2010)

- Average Spread Between Maximum Day and 3-Day Maximum = -2.0
- Average Spread Between Maximum Day and 7-Day Maximum = -2.7
- Average Spread Between Maximum Day and 30-Day Maximum = -5.1
- Average Spread Between Maximum Day and Seasonal Average = -7.2

Below-Average Rainfall Years (2006, 2011-2014)

- Average Spread Between Maximum Day and 3-Day Maximum = -0.6
- Average Spread Between Maximum Day and 7-Day Maximum = -1.5
- Average Spread Between Maximum Day and 30-Day Maximum = -4.0
- Average Spread Between Maximum Day and Seasonal Average = -6.9
As shown in Table 3-3, the extent of time that peak demands occur is highly dependent on precipitation conditions. Generally, during below-normal years of precipitation, the demand is sustained at a higher level for longer, as evidenced by the spread between the maximum day and the 3-day and 7-day maximums. Looking specifically at 2012, a recognized drought year in the region, the demand sustained for 7 consecutive days (the 7-day maximum) was only 1 MGD less than the maximum single day demand. A similar result is seen in 2011, 2013, and 2014. Therefore, it is recommended that water supply and treatment systems continue to be designed for the single maximum day. Designing for anything less is not likely to save significant capital investments and may put the City in a position of having to impose long-term conservation restrictions on water users during drought years.

### 3.2 Wholesale Water Use Projections

Projections for the City’s current wholesale users were determined based on contractual requirements and will be used for both demand projection methods. Historical water use varies substantially and the contractual amounts are typically more than the wholesale user purchases in any given year. Since the City has contracts in place to provide a specific amount of water, the City should have the infrastructure in place to deliver the contractual amounts to each wholesale user. The following is a summary of the contracts with each wholesale user and a summary of the basis for the future demand projections:

- **RWD #7:** The water purchase contract on file was executed in 2011 and expires in 2031. The contract allows for an increasing schedule of annual water purchased from 113 million gallons (MG) per year from 2010 to 2015 up to 219 MG per year in 2031. For purposes of water use projections, the contractual amount in 2031 was carried forward to 2035, consistent with prior years of the contract. Projections beyond 2035 were made based on a linear projection of the contractual amounts for 2012 to 2031.

- **RWD #6:** The water purchase contract on file was executed in 2013 and expires in 2017. The contractual amount through 2017 is 184.5 MG per year, which is approximately 0.5 MGD. Projections for 2020 were held constant at 0.5 MGD; projections for 2025 and beyond were made based on the same growth rate as actual growth that occurred from 2001 to 2014.

- **New Century:** The water purchase contract on file was executed in 2015 and expires in 2035. The contract allows for a water purchase of 219 MG gallons per year. The City currently anticipates New Century will require 1.0 MGD in the years beyond the existing contract.

Figure 3-1 and Table 3-4 show the future demand projections for the current wholesale water users.
Figure 3-1. Wholesale Water Use Projections
Table 3-4. Wholesale Water Use Projections

<table>
<thead>
<tr>
<th>Year</th>
<th>RWD #1</th>
<th>RWD #6</th>
<th>RWD #7</th>
<th>New Century</th>
<th>WaterOne</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Use (1,000 gal)</td>
<td>Annual Average Use (MGD)</td>
<td>Total Use (1,000 gal)</td>
<td>Annual Average Use (MGD)</td>
<td>Total Use (1,000 gal)</td>
<td>Annual Average Use (MGD)</td>
</tr>
<tr>
<td></td>
<td>Total Use (1,000 gal)</td>
<td>Annual Average Use (MGD)</td>
<td>Total Use (1,000 gal)</td>
<td>Annual Average Use (MGD)</td>
<td>Total Use (1,000 gal)</td>
<td>Annual Average Use (MGD)</td>
</tr>
<tr>
<td>2001</td>
<td>20,424</td>
<td>0.06</td>
<td>51,738</td>
<td>0.14</td>
<td>50,574</td>
<td>0.14</td>
</tr>
<tr>
<td>2002</td>
<td>950</td>
<td>0.00</td>
<td>63,489</td>
<td>0.17</td>
<td>67,770</td>
<td>0.19</td>
</tr>
<tr>
<td>2003</td>
<td>827</td>
<td>0.00</td>
<td>74,398</td>
<td>0.20</td>
<td>56,218</td>
<td>0.15</td>
</tr>
<tr>
<td>2004</td>
<td>1,270</td>
<td>0.00</td>
<td>83,752</td>
<td>0.23</td>
<td>54,208</td>
<td>0.15</td>
</tr>
<tr>
<td>2005</td>
<td>4,178</td>
<td>0.01</td>
<td>96,605</td>
<td>0.26</td>
<td>58,142</td>
<td>0.16</td>
</tr>
<tr>
<td>2006</td>
<td>29,562</td>
<td>0.08</td>
<td>111,273</td>
<td>0.30</td>
<td>73,538</td>
<td>0.20</td>
</tr>
<tr>
<td>2007</td>
<td>0</td>
<td>0</td>
<td>109,161</td>
<td>0.30</td>
<td>67,688</td>
<td>0.19</td>
</tr>
<tr>
<td>2008</td>
<td>0</td>
<td>0</td>
<td>96,745</td>
<td>0.27</td>
<td>44,222</td>
<td>0.12</td>
</tr>
<tr>
<td>2009</td>
<td>0</td>
<td>0</td>
<td>167,231</td>
<td>0.46</td>
<td>47,507</td>
<td>0.13</td>
</tr>
<tr>
<td>2010</td>
<td>0</td>
<td>0</td>
<td>115,699</td>
<td>0.32</td>
<td>62,839</td>
<td>0.17</td>
</tr>
<tr>
<td>2011</td>
<td>0</td>
<td>0</td>
<td>118,465</td>
<td>0.32</td>
<td>71,871</td>
<td>0.20</td>
</tr>
<tr>
<td>2012</td>
<td>0</td>
<td>0</td>
<td>129,221</td>
<td>0.35</td>
<td>80,263</td>
<td>0.22</td>
</tr>
<tr>
<td>2013</td>
<td>0</td>
<td>0</td>
<td>103,677</td>
<td>0.28</td>
<td>65,992</td>
<td>0.18</td>
</tr>
<tr>
<td>2014</td>
<td>0</td>
<td>0</td>
<td>120,839</td>
<td>0.33</td>
<td>71,426</td>
<td>0.20</td>
</tr>
<tr>
<td>2015</td>
<td>0</td>
<td>0</td>
<td>82,587</td>
<td>0.23</td>
<td>81,161</td>
<td>0.22</td>
</tr>
<tr>
<td>2020</td>
<td>0</td>
<td>0</td>
<td>184,464</td>
<td>0.51</td>
<td>132,000</td>
<td>0.36</td>
</tr>
<tr>
<td>2025</td>
<td>0</td>
<td>0</td>
<td>184,464</td>
<td>0.51</td>
<td>153,000</td>
<td>0.42</td>
</tr>
<tr>
<td>2030</td>
<td>0</td>
<td>0</td>
<td>206,262</td>
<td>0.57</td>
<td>183,000</td>
<td>0.50</td>
</tr>
<tr>
<td>2035</td>
<td>0</td>
<td>0</td>
<td>231,812</td>
<td>0.64</td>
<td>219,000</td>
<td>0.60</td>
</tr>
<tr>
<td>2040</td>
<td>0</td>
<td>0</td>
<td>257,362</td>
<td>0.71</td>
<td>238,184</td>
<td>0.65</td>
</tr>
<tr>
<td>2045</td>
<td>0</td>
<td>0</td>
<td>282,911</td>
<td>0.78</td>
<td>264,483</td>
<td>0.72</td>
</tr>
<tr>
<td>2050</td>
<td>0</td>
<td>0</td>
<td>308,461</td>
<td>0.85</td>
<td>290,781</td>
<td>0.80</td>
</tr>
<tr>
<td>2055</td>
<td>0</td>
<td>0</td>
<td>334,012</td>
<td>0.92</td>
<td>317,079</td>
<td>0.87</td>
</tr>
</tbody>
</table>

1 2001 through 2014 represents actual water use. 2015 through 2055 represents projected demand.
2 The City does not currently have a contract to serve RWD #1; therefore it is assumed the City will not serve to RWD #1 in the future.
3 The contract for service to RWD 7 is through 2017. Projections through 2020 are based on the contractual amount of 504,000 gallons per day. Projections beyond 2020 are based on the same growth rate as actual growth that occurred from 2001 to 2014.
4 The City's current contract with WaterOne is on an emergency basis if the City has the supply available.
3.3 Population-Based Demand Projections

Water use projections based on population and past statistics of water use are commonly used in water supply planning. This method is based on evaluation of historic water use trends applied to the projected population of the water service area.

3.3.1 Population Projections

Population for the City of Olathe has been recorded by the U.S. Census Bureau since 1870. Population growth was modest until the 1950s, when the growth of the City began in earnest. Since 1980, population growth has been steady, with an average annual growth rate between 1980 and 2010 of 6.5-percent. The historical population of the City of Olathe based on decennial data from the U.S. Census Bureau is shown in Figure 3-2.

![Figure 3-2. Historical Population, City of Olathe](image)

As discussed in Section 2.1, the City does not currently serve water to all citizens within the incorporated city boundaries. Other service providers, including WaterOne, Johnson County RWD No. 6, Johnson County RWD No. 7, and New Century provide water to a portion of the population within the City of Olathe. In order to determine the water use projections for the City’s water service area, it is necessary to determine the population which the City serves.
Estimates of population within the water service area were gathered including from the City’s Planning Department. It should be noted that the data are estimates; the City does not maintain specific data on the population which the City serves. The population served by the water utility was approximately equal to the City population until 2002, when the growth of the water service area began diverging from the City of Olathe at a slower growth rate due to the presence of other water utility providers within the City boundaries. Figure 3-3 shows a comparison of historical data for the City of Olathe versus the Olathe water service area.

The City’s Planning Department provided population projections of the water service area for 2015, 2025, and 2040. Based on this information, as well as the historical population of the water service area, supplementary population projections were developed. Table 3-5 and Figure 3-4 present the population projections for the City of Olathe and the Olathe water service area. The projections continue to reflect the divergence of the water service area and the City. The projected population of the water service area at the planning horizon of this Study of 2055 is approximately 150,000.

Table 3-5. Population Projections (2015 – 2055)

<table>
<thead>
<tr>
<th>Year</th>
<th>City of Olathe Population</th>
<th>Olathe Water Service Area</th>
<th>Population</th>
<th>Percent Growth</th>
</tr>
</thead>
<tbody>
<tr>
<td>2015</td>
<td>138,956</td>
<td>115,056</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2020</td>
<td>153,724</td>
<td>122,000</td>
<td>6.0%</td>
<td></td>
</tr>
<tr>
<td>2025</td>
<td>168,492</td>
<td>128,182</td>
<td>5.1%</td>
<td></td>
</tr>
<tr>
<td>2030</td>
<td>183,260</td>
<td>133,603</td>
<td>4.2%</td>
<td></td>
</tr>
<tr>
<td>2035</td>
<td>198,029</td>
<td>138,263</td>
<td>3.5%</td>
<td></td>
</tr>
<tr>
<td>2040</td>
<td>212,797</td>
<td>142,161</td>
<td>2.8%</td>
<td></td>
</tr>
<tr>
<td>2045</td>
<td>227,565</td>
<td>145,299</td>
<td>2.2%</td>
<td></td>
</tr>
<tr>
<td>2050</td>
<td>242,333</td>
<td>147,675</td>
<td>1.6%</td>
<td></td>
</tr>
<tr>
<td>2055</td>
<td>257,101</td>
<td>149,289</td>
<td>1.1%</td>
<td></td>
</tr>
</tbody>
</table>

1 Provided by City of Olathe Planning Department
Figure 3-3. Historical Population of Olathe Water Service Area versus City of Olathe
Figure 3-4. Population Projections (2015 – 2055)
3.3.2 Demand Projections

Demand projections based on the methodology of population and historical water use are determined as follows:

- Projections of future wholesale water use are estimated as presented in Section 3.2.
- Future industrial water use projections are estimated based on historical water use patterns and the potential for future industrial development.
- Residential and commercial water use projections, which also include non-revenue water and unaccounted-for water, are based on application of a selected water use rate in gallons per capita per day with the projected population of the water service area.

Projections of Industrial Water Use. Future water use projections for the City’s industrial users was determined based on review of historical water use trends and the future industrial growth plans of the City. When considering the historical period of record of 2001 through 2014 presented in Table 3-1, industrial water use has seen a declining trend, although the number of service accounts has stayed relatively static. Since 2007, a year selected based on the Olathe Comprehensive Plan, industrial water use has been slightly increasing with a maximum use of 0.68 MGD. This maximum use will be the basis of future projections. Historical and future industrial demands are shown in Table 3-6.
Table 3-6. Industrial Water Use Projections

<table>
<thead>
<tr>
<th>Year</th>
<th>Number of Accounts</th>
<th>Base Industrial Demand (MGD)</th>
<th>Reserve for Industrial Growth (MGD)</th>
<th>Total Projected Industrial Demand (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2001</td>
<td>28</td>
<td>1.36</td>
<td>-</td>
<td>1.36</td>
</tr>
<tr>
<td>2002</td>
<td>31</td>
<td>0.74</td>
<td>-</td>
<td>0.74</td>
</tr>
<tr>
<td>2003</td>
<td>33</td>
<td>0.78</td>
<td>-</td>
<td>0.78</td>
</tr>
<tr>
<td>2004</td>
<td>31</td>
<td>0.70</td>
<td>-</td>
<td>0.70</td>
</tr>
<tr>
<td>2005</td>
<td>30</td>
<td>0.61</td>
<td>-</td>
<td>0.61</td>
</tr>
<tr>
<td>2006</td>
<td>30</td>
<td>0.66</td>
<td>-</td>
<td>0.66</td>
</tr>
<tr>
<td>2007</td>
<td>30</td>
<td>0.62</td>
<td>-</td>
<td>0.62</td>
</tr>
<tr>
<td>2008</td>
<td>28</td>
<td>0.57</td>
<td>-</td>
<td>0.57</td>
</tr>
<tr>
<td>2009</td>
<td>28</td>
<td>0.52</td>
<td>-</td>
<td>0.52</td>
</tr>
<tr>
<td>2010</td>
<td>24</td>
<td>0.45</td>
<td>-</td>
<td>0.45</td>
</tr>
<tr>
<td>2011</td>
<td>26</td>
<td>0.50</td>
<td>-</td>
<td>0.50</td>
</tr>
<tr>
<td>2012</td>
<td>28</td>
<td>0.57</td>
<td>-</td>
<td>0.57</td>
</tr>
<tr>
<td>2013</td>
<td>40</td>
<td>0.68</td>
<td>-</td>
<td>0.68</td>
</tr>
<tr>
<td>2014</td>
<td>30</td>
<td>0.59</td>
<td>-</td>
<td>0.59</td>
</tr>
<tr>
<td>2015</td>
<td>-</td>
<td>0.57</td>
<td>-</td>
<td>0.57</td>
</tr>
<tr>
<td>2020</td>
<td>-</td>
<td>0.68</td>
<td>0.07</td>
<td>0.75</td>
</tr>
<tr>
<td>2025</td>
<td>-</td>
<td>0.68</td>
<td>0.13</td>
<td>0.81</td>
</tr>
<tr>
<td>2030</td>
<td>-</td>
<td>0.68</td>
<td>0.20</td>
<td>0.88</td>
</tr>
<tr>
<td>2035</td>
<td>-</td>
<td>0.68</td>
<td>0.27</td>
<td>0.95</td>
</tr>
<tr>
<td>2040</td>
<td>-</td>
<td>0.68</td>
<td>0.33</td>
<td>1.01</td>
</tr>
<tr>
<td>2045</td>
<td>-</td>
<td>0.68</td>
<td>0.40</td>
<td>1.08</td>
</tr>
<tr>
<td>2050</td>
<td>-</td>
<td>0.68</td>
<td>0.47</td>
<td>1.15</td>
</tr>
<tr>
<td>2055</td>
<td>-</td>
<td>0.68</td>
<td>0.53</td>
<td>1.21</td>
</tr>
</tbody>
</table>

It is unknown what types of industrial users may come to the City in the future and the amount of water required; therefore, an industrial reserve capacity is built into the future growth projections. The purpose of the reserve capacity is to ensure water supply capacity is readily available for any industry that may relocate to the City in the future. A reserve supply can promote economic development as an incentive to prospective industries.

Based on the 2010 Comprehensive Plan, which uses 2007 as the base year when projecting industrial development, the City of Olathe is projecting to 8.34 million square feet of additional industrial space by 2030. The Comprehensive Plan states the industrial space was 14.8 million square feet in 2007 for a projected development of 23 million square feet in 2030. As of 2014, actual industrial development has been on pace with the projected development, as shown in Table 3-7.
Table 3-7. Projected Industrial Growth

<table>
<thead>
<tr>
<th></th>
<th>Year</th>
<th>Industrial Development (sq. ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Projected Industrial Development (2007 – 2030) ¹</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Starting Industrial Development</td>
<td>2007</td>
<td>14,796,600</td>
</tr>
<tr>
<td>Projected Industrial Development</td>
<td>2030</td>
<td>23,136,900</td>
</tr>
<tr>
<td>Projected Industrial Development Added</td>
<td></td>
<td>8,340,300</td>
</tr>
<tr>
<td>Projected Annual Average Industrial Development Added (sq. ft/yr)</td>
<td></td>
<td>362,622</td>
</tr>
<tr>
<td>Actual Industrial Development (2007 – 2014) ²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actual Industrial Development</td>
<td>2007</td>
<td>13,655,218</td>
</tr>
<tr>
<td>Actual Industrial Development</td>
<td>2014</td>
<td>16,297,048</td>
</tr>
<tr>
<td>Total Industrial Development Added</td>
<td></td>
<td>2,641,830</td>
</tr>
<tr>
<td>Annual Average Industrial Development Added (sq. ft/yr)</td>
<td></td>
<td>377,404</td>
</tr>
</tbody>
</table>

² Source: City of Olathe Demographics and Development, 2007-2010

The projected 2030 industrial development represents a 42-percent increase from the 2014 industrial development. For planning purposes, it is assumed the required water capacity for industrial uses in 2030 will be 42-percent greater than current conditions. The industrial demand in 2030, with the industrial reserve, is projected as 0.88 MGD. The industrial demand for the remaining years is determined based on a linear projection between 2015 and 2030. At the end of the planning horizon of this Study (2055), it is projected the industrial water use will be approximately 1.2 MGD.

Projections of Residential and Commercial Water Use. Demand projections for residential and commercial water use (including non-revenue water use and unaccounted for water) are estimated by multiplying a water use factor GPCD by the projected population of the water service area. As shown in Table 3-2, presented previously, the average water use per person has varied between 92 gpcd to 129 gpcd with an average of 107 gpcd since 2001 (including the drought periods of 2002 to 2006 and 2011 to 2012). Average demand projections using the average GPCD water use rate and the maximum GPCD are shown in Figure 3-5 and Table 3-8.

For water supply planning, it is recommended that the demand projections be based on the maximum GPCD water use rate, which occurred in 2012, a recognized drought year. Since the demand projections will form a basis for determination of the adequacy of water rights, it is recommended that the maximum value of 129 gpcd be used to project average demands, which will provide for enough water supply during drought years.

Projections of Maximum Day Demand. As discussed previously, it is recommended that the peak demand projections, to be used as the basis for planning of water supply and treatment capacity, be based on the single maximum day demand. As was shown in Table 3-3, the ratio of the maximum day to average day water use (peaking factor) has ranged from 1.73 to 2.12 with an average of 1.80 since 2006. The maximum ratio of 2.12 occurred in 2013. As shown in Figure 3-5, the selection of the peaking factor, in
combination with the GPCD water use rate, can give a wide range of demand projections. Use of the worst-case peaking factor of 2.12 in combination with the worst-case GPCD provides demand projections may be overly conservative; therefore, it is recommended that the maximum day demand projections be based on the average peaking factor of 1.80.
Figure 3-5. Demand Projections, Population-Based Method
### Table 3-8. Demand Projections, Population-Based

<table>
<thead>
<tr>
<th>Year</th>
<th>Olathe WWA Population</th>
<th>Wholesale Water Use (MGD)</th>
<th>Industrial Water Use (MGD)</th>
<th>Avg GPCD, Avg MD:AD</th>
<th>Max GPCD, Avg MD:AD</th>
<th>Max GPCD, MD:AD of 1.8 (^2) (Recommended)</th>
<th>Avg GPCD, Max MD:AD</th>
<th>Max GPCD, Max MD:AD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2015</td>
<td>115,056</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>11.4</td>
<td>19.3</td>
<td>-</td>
<td>11.4</td>
</tr>
<tr>
<td>2020</td>
<td>122,000</td>
<td>1.47</td>
<td>0.75</td>
<td>13.1</td>
<td>15.3</td>
<td>29.0</td>
<td>15.7</td>
<td>18.0</td>
</tr>
<tr>
<td>2025</td>
<td>128,182</td>
<td>1.52</td>
<td>0.81</td>
<td>13.7</td>
<td>16.1</td>
<td>30.5</td>
<td>16.5</td>
<td>18.9</td>
</tr>
<tr>
<td>2030</td>
<td>133,603</td>
<td>1.67</td>
<td>0.88</td>
<td>14.3</td>
<td>16.8</td>
<td>32.0</td>
<td>17.2</td>
<td>19.8</td>
</tr>
<tr>
<td>2035</td>
<td>138,263</td>
<td>2.21</td>
<td>0.95</td>
<td>14.8</td>
<td>18.0</td>
<td>34.1</td>
<td>17.8</td>
<td>21.0</td>
</tr>
<tr>
<td>2040</td>
<td>142,161</td>
<td>2.36</td>
<td>1.01</td>
<td>15.2</td>
<td>18.6</td>
<td>38.3</td>
<td>18.3</td>
<td>21.7</td>
</tr>
<tr>
<td>2045</td>
<td>145,299</td>
<td>2.50</td>
<td>1.08</td>
<td>15.5</td>
<td>19.1</td>
<td>36.3</td>
<td>18.7</td>
<td>22.3</td>
</tr>
<tr>
<td>2050</td>
<td>147,675</td>
<td>2.64</td>
<td>1.15</td>
<td>15.8</td>
<td>19.6</td>
<td>37.2</td>
<td>19.1</td>
<td>22.8</td>
</tr>
<tr>
<td>2055</td>
<td>149,289</td>
<td>2.78</td>
<td>1.21</td>
<td>16.0</td>
<td>20.0</td>
<td>37.9</td>
<td>19.3</td>
<td>23.3</td>
</tr>
</tbody>
</table>

1 2015 is actual demand information.
2 Peaking factor is based on the 2013 maximum day of 28.6 MGD (the highest maximum day that occurred since 2006) and the 2012 average day of 16.2 MGD (the highest average day that has occurred since 2006).
3.4 Land Use-Based Demand Projections

Water use projections based on land use are estimated using a determination of unit demands for various land use types (e.g., single-family residential, commercial) coupled with a future land use plan for the City.

A determination of unit demands (in gallons per acre) for various types of land use form the basis of projection of future demands. Because unit demands can vary from community to community, the City’s individual meter data and land use were used to estimate unit demands. Individual meter data from 2014 was overlaid on the City’s future land use plan. The City’s future land use plan contains the following classifications:

- Commercial
- Industrial
- Industrial/Business Park (generally office space districts)
- Park
- Public (schools, government offices, hospitals and other institutional uses)
- Residential High Density
- Residential Medium Density
- Residential Low Density
- Retail Commercial

For the purposes of this Report, land uses were combined into the following simplified classifications:

- Commercial (commercial, industrial/business park, public, and retail commercial)
- Industrial
- Park/Open Space
- Multi-Family Residential (high and medium density residential)
- Single-Family Residential (low density residential)

The water service area was divided into six (6) distinct areas to aid in calculation of unit demands and determination of future demands. The areas are described as follows:

- Area 1 – Main Pressure Zone, Cedar Creek Development
- Area 2 – Main Pressure Zone, West of I-35 and Generally North of 127th Street
- Area 3 – Main Pressure Zone, West of I-35 and Generally Between 127th Street and 143rd Street
- Area 4 – Main Pressure Zone, West of I-35 and South of 143rd Street
- Area 5 – Main Pressure Zone, East of I-35
- Area 6 – Southeast Pressure Zone
For each area, the following tasks were completed to determine the unit water use for each land use classification:

- The amount of developable area (i.e. area to be developed in the future) of each land-use classification was estimated and is shown in Figures 3-6 through 3-11. This analysis was based on review of aerial imagery of the City and local development knowledge.

- Based on the 2014 individual meter data, the amount of water use for each land-use classification was estimated.

- Based on the total area of each land use and the developable area of each land use, the existing developed area for each land use classification was estimated.

- A unit demand in gallons per acre was determined for each land use based on the 2014 water use and the developed area.

Based on the steps performed above, the unit demands across the system were estimated and are presented in Table 3-9.

### Table 3-9. Unit Demand Statistics

<table>
<thead>
<tr>
<th></th>
<th>Single Family</th>
<th>Multi Family</th>
<th>Commercial</th>
<th>Industrial</th>
<th>Park/Open Space</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Calculated Unit Demands by Area and Land Use Classification</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area 1</td>
<td>649</td>
<td>-</td>
<td>526</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Area 2</td>
<td>750</td>
<td>490</td>
<td>454</td>
<td>895</td>
<td>-</td>
</tr>
<tr>
<td>Area 3</td>
<td>514</td>
<td>550</td>
<td>405</td>
<td>186</td>
<td>17</td>
</tr>
<tr>
<td>Area 4</td>
<td>714</td>
<td>1,215</td>
<td>590</td>
<td>364</td>
<td>-</td>
</tr>
<tr>
<td>Area 5</td>
<td>745</td>
<td>1,030</td>
<td>436</td>
<td>248</td>
<td>8</td>
</tr>
<tr>
<td>Area 6</td>
<td>599</td>
<td>486</td>
<td>322</td>
<td>538</td>
<td>107</td>
</tr>
<tr>
<td><strong>System-Wide Statistics</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>662</td>
<td>754</td>
<td>456</td>
<td>446</td>
<td>44</td>
</tr>
<tr>
<td>Minimum</td>
<td>514</td>
<td>486</td>
<td>322</td>
<td>186</td>
<td>8</td>
</tr>
<tr>
<td>Maximum</td>
<td>750</td>
<td>1,215</td>
<td>590</td>
<td>895</td>
<td>107</td>
</tr>
<tr>
<td><strong>Basis of Unit Demand Projections</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Based on 2014</td>
<td>665</td>
<td>760</td>
<td>450</td>
<td>450</td>
<td>40</td>
</tr>
<tr>
<td>Representative (^1)</td>
<td>930</td>
<td>1065</td>
<td>630</td>
<td>450</td>
<td>40</td>
</tr>
</tbody>
</table>

\(^1\) A “representative” unit demand was calculated to account for the limited water use history used in this analysis. Based on the historical water use analysis shown in Table 3-1, 2014 was a relatively low water use year with 93 gpcd. In order to be comparable to the population-based demands, which is based on the maximum gpcd, the representative unit demand factor is calculated as follows: 129 gpcd (in 2012) divided by 93 gpcd (in 2014) equals a factor of 1.4. The 2014 unit demands for single-family, multi-family, and commercial uses were multiplied by this factor to come up with a unit demand that is representative of recent high water use years.
Using the developable area for each land-use classification and the representative unit demand, demand projections were made for the planning horizon. The following assumptions were made in projecting the demands:

- The water service area will be built out by 2055.
- The additional water use between 2015 and the build-out year of 2055 will be distributed evenly throughout the period.
- Unaccounted-for water will be 15-percent of the metered water use. 15-percent is the average water loss experienced over the historical period as shown in Table 3-10.
- A factor of 5-percent is applied to account for infill within areas that appear developed on aerial imagery but are vacant as well as other unknown development factors.

### Table 3-10. Demand Projections – Land Use-Based Method

<table>
<thead>
<tr>
<th>Year</th>
<th>Wholesale Water Use (MGD)</th>
<th>Remaining Metered Water Use (MGD)</th>
<th>Unaccounted-For Water (MGD)</th>
<th>Average Day Demand (MGD)</th>
<th>Maximum Day Demand (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2014</td>
<td>0.93</td>
<td>9.83</td>
<td>1.38</td>
<td>12.1</td>
<td>23.1</td>
</tr>
<tr>
<td>2015</td>
<td>0.94</td>
<td>10.41</td>
<td>0.2</td>
<td>11.4</td>
<td>19.3</td>
</tr>
<tr>
<td>2020</td>
<td>1.47</td>
<td>11.29</td>
<td>1.83</td>
<td>14.6</td>
<td>27.9</td>
</tr>
<tr>
<td>2025</td>
<td>1.52</td>
<td>12.18</td>
<td>1.98</td>
<td>15.7</td>
<td>30.1</td>
</tr>
<tr>
<td>2030</td>
<td>1.67</td>
<td>13.07</td>
<td>2.13</td>
<td>16.9</td>
<td>32.3</td>
</tr>
<tr>
<td>2035</td>
<td>2.21</td>
<td>13.96</td>
<td>2.28</td>
<td>18.4</td>
<td>34.6</td>
</tr>
<tr>
<td>2040</td>
<td>2.36</td>
<td>14.85</td>
<td>2.43</td>
<td>19.6</td>
<td>37.0</td>
</tr>
<tr>
<td>2045</td>
<td>2.50</td>
<td>15.74</td>
<td>2.59</td>
<td>20.8</td>
<td>39.4</td>
</tr>
<tr>
<td>2050</td>
<td>2.64</td>
<td>16.63</td>
<td>2.75</td>
<td>22.0</td>
<td>41.8</td>
</tr>
<tr>
<td>2055</td>
<td>2.78</td>
<td>17.52</td>
<td>2.91</td>
<td>23.2</td>
<td>44.2</td>
</tr>
</tbody>
</table>

1 Represents residential, commercial, and industrial users and includes a 5-percent factor to include infill development and other unknowns.
2 Calculated as 15-percent of the base water use.
3 Based on a peaking factor of 1.90 as established previously.
4 Represents actual water use – values are actual and not calculated.
5 Appears to be an anomaly because records show only a 2% water loss compared the lowest number in the last 10 year of 11.4% water loss in 2014.
Figure 3-6. Area 1 - Main Pressure Zone

**Olathe, Kansas**

Water Master Plan Update

**Future Landuse**

- Commercial
- Industrial
- Park / Open Space
- Public
- Multi-Family
- Single Family

**Water Facilities**

- Pressure Zone Boundary
- Pressure Control Station

**Summary**

Total Area (ac) = 4,410
Developed Area (ac) = 936
Undeveloped Area (ac) = 3,410
Water Area (ac) = 56

Developable Area Breakdown:

- Single Family (ac) = 2,299
- Multi-Family (ac) = 0
- Commercial / Public (ac) = 542
- Park / Open Space (ac) = 569

**Source:** Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
Figure 3-7. Area 2
Main Pressure Zone
North and West of I-35
Developable Area

Summary

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Area (ac)</td>
<td>2,389</td>
</tr>
<tr>
<td>Developed Area (ac)</td>
<td>1,749</td>
</tr>
<tr>
<td>Undeveloped Area (ac)</td>
<td>640</td>
</tr>
<tr>
<td>Water Area (ac)</td>
<td>0</td>
</tr>
<tr>
<td>Developable Area Breakdown</td>
<td></td>
</tr>
<tr>
<td>Single Family (ac)</td>
<td>76</td>
</tr>
<tr>
<td>Multi-Family (ac)</td>
<td>74</td>
</tr>
<tr>
<td>Commercial / Public (ac)</td>
<td>490</td>
</tr>
<tr>
<td>Park / Open Space (ac)</td>
<td>0</td>
</tr>
<tr>
<td>Industrial (ac)</td>
<td>0</td>
</tr>
</tbody>
</table>
Olathe, Kansas
Water Master Plan Update

Figure 3-8. Area 3
Main Pressure Zone
Northwest
Developable Area

Summary
Total Area (ac) = 7,004
Developed Area (ac) = 5,264
Undeveloped Area (ac) = 1,591
Water Area (ac) = 149

Developable Area Breakdown
Single Family (ac) = 620
Multi-Family (ac) = 167
Commercial / Public (ac) = 140
Park / Open Space (ac) = 549
Industrial (ac) = 60

Map Feature Key
- Olathe Water Service Area
- Future Landuse
  - Commercial
  - Industrial
  - Park / Open Space
  - Public
  - Multi-Family
  - Single Family
- Water Facilities
- Pressure Zone Boundary
- Pressure Control Station
Figure 3-9. Area 4 Main Pressure Zone West of I-35 and S. 143rd Developable Area
Figure 3-10. Area 5
Main Pressure Zone North of Santa Fe
Developable Area

Summary

- Total Area (ac) = 3,052
- Developed Area (ac) = 2,698
- Undeveloped Area (ac) = 354
- Water Area (ac) = 0
- Developable Area Breakdown:
  - Single Family (ac) = 141
  - Multi-Family (ac) = 63
  - Commercial / Public (ac) = 150
  - Park / Open Space (ac) = 0
  - Industrial (ac) = 0

Future Landuse

- Commercial
- Industrial
- Park / Open Space
- Public
- Multi-Family
- Single Family

Map Feature Key

- Water Facilities
- Pressure Zone Boundary
- Pressure Control Station

Olathe, Kansas

Water Master Plan Update

Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
City of Overland Park

Southwest Zone

135TH ST
143RD ST
151ST
159TH
MUR-LEN RD
PFLUMM
BLACK BOB
Renner Rd
Standpipe & Pump Station
Black Bob1 & 2 Standpipe & Pump Station

Total (ac) = 41
60% Developed
Developable Area (ac) = 17

Total (ac) = 35
50% Developed
Developable Area (ac) = 17

Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

Future Landuse

<table>
<thead>
<tr>
<th>Landuse</th>
<th>Total Area (ac)</th>
<th>Developed Area (ac)</th>
<th>Undeveloped Area (ac)</th>
<th>Water Area (ac)</th>
<th>Water Area (% of Developed)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Commercial</td>
<td>393</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Multi-Family</td>
<td>232</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Industrial</td>
<td>105</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Public</td>
<td>33</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Park / Open Space</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>7,359</td>
<td>6,615</td>
<td>744</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

Developable Area Breakdown

- Single Family (ac) = 367
- Multi-Family (ac) = 0
- Commercial / Public (ac) = 232
- Park / Open Space (ac) = 0
- Industrial (ac) = 105

Summary

Total Area (ac) = 7,359
Developed Area (ac) = 6,615
Undeveloped Area (ac) = 744
Water Area (ac) = 0
Developable Area (% of Developed) = 60%

Olathe, Kansas
Water Master Plan Update

Figure 3-11. Area 6
Southeast Pressure Zone
Developable Area
3.5 Comparison of Demand Projection Methods

Table 3-11 presents a comparison between the population-based demand projections and the land use-based demand projections. Ultimately, at build-out, the land use projections are higher by 5-percent. In interim years, however, the population-based demand projections are as much as 14-percent higher than the land-use based demands tapering off over time. The difference is attributed to assumptions between the two methods.

Table 3-11. Comparison of Demand Projection Methods

<table>
<thead>
<tr>
<th>Year</th>
<th>Wholesale Users (MGD)</th>
<th>Remaining Users + UAF (MGD)</th>
<th>Total Average Day (MGD)</th>
<th>Max Day Demand (MGD)</th>
<th>Wholesale Users (MGD)</th>
<th>Remaining Users + UAF (MGD)</th>
<th>Total Average Day (MGD)</th>
<th>Max Day Demand (MGD)</th>
<th>Percent Change Based on Maximum Day</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012</td>
<td>0.97</td>
<td>15.2</td>
<td>16.2</td>
<td>28.1</td>
<td>0.97</td>
<td>15.2</td>
<td>16.2</td>
<td>28.1</td>
<td>-</td>
</tr>
<tr>
<td>2013</td>
<td>0.84</td>
<td>12.7</td>
<td>13.5</td>
<td>28.6</td>
<td>0.94</td>
<td>12.7</td>
<td>13.5</td>
<td>28.6</td>
<td>-</td>
</tr>
<tr>
<td>2014</td>
<td>0.92</td>
<td>11.2</td>
<td>12.1</td>
<td>24.1</td>
<td>0.92</td>
<td>11.2</td>
<td>12.1</td>
<td>24.1</td>
<td>-</td>
</tr>
<tr>
<td>2015</td>
<td>0.94</td>
<td>10.6</td>
<td>11.4</td>
<td>19.3</td>
<td>0.94</td>
<td>10.6</td>
<td>11.4</td>
<td>19.3</td>
<td>-</td>
</tr>
<tr>
<td>2020</td>
<td>1.47</td>
<td>16.5</td>
<td>18.0</td>
<td>31.4</td>
<td>1.47</td>
<td>13.1</td>
<td>14.6</td>
<td>27.7</td>
<td>-14%</td>
</tr>
<tr>
<td>2025</td>
<td>1.52</td>
<td>17.3</td>
<td>18.9</td>
<td>33.2</td>
<td>1.52</td>
<td>14.2</td>
<td>15.7</td>
<td>29.8</td>
<td>-13%</td>
</tr>
<tr>
<td>2030</td>
<td>1.67</td>
<td>18.1</td>
<td>19.8</td>
<td>34.9</td>
<td>1.67</td>
<td>15.2</td>
<td>16.9</td>
<td>32.0</td>
<td>-11%</td>
</tr>
<tr>
<td>2035</td>
<td>2.21</td>
<td>18.8</td>
<td>21.0</td>
<td>36.4</td>
<td>2.21</td>
<td>16.2</td>
<td>18.4</td>
<td>35.1</td>
<td>-6%</td>
</tr>
<tr>
<td>2040</td>
<td>2.36</td>
<td>19.4</td>
<td>21.7</td>
<td>37.7</td>
<td>2.36</td>
<td>17.3</td>
<td>19.6</td>
<td>37.3</td>
<td>-4%</td>
</tr>
<tr>
<td>2045</td>
<td>2.50</td>
<td>19.8</td>
<td>22.3</td>
<td>39.0</td>
<td>2.50</td>
<td>18.3</td>
<td>20.8</td>
<td>39.6</td>
<td>-1%</td>
</tr>
<tr>
<td>2050</td>
<td>2.64</td>
<td>20.2</td>
<td>22.8</td>
<td>40.0</td>
<td>2.64</td>
<td>19.4</td>
<td>22.0</td>
<td>41.8</td>
<td>2%</td>
</tr>
<tr>
<td>2055</td>
<td>2.78</td>
<td>20.5</td>
<td>23.3</td>
<td>40.9</td>
<td>2.78</td>
<td>20.4</td>
<td>23.2</td>
<td>44.1</td>
<td>5%</td>
</tr>
</tbody>
</table>

1 Represents actual use.  
2 Appears to be an anomaly because records show only a 2% water loss compared the lowest number in the last 10 year of 11.4% water loss in 2014.

It is recommended the capital improvements as developed in this Report be based on the population-based demand projections method with a minor modification. In 2004 the City implemented a water conservation plan in response to having to purchase water from WaterOne. The actual maximum demand projection before 2005 will not be considered since conservation modified maximum demand use. Since the maximum projected demand far exceed the last ten years of maximum demand data it will be modified to fit. The following Table 3-12 shows the data as modified to best fit the curve of existing maximum demand data and projected demands.
Table 3-12. Comparison of Demand Projection Methods Modified
Max Day Demands

<table>
<thead>
<tr>
<th>Year</th>
<th>Total Average Day (MGD)</th>
<th>Max Day Demand (MGD)</th>
<th>Total Average Day (MGD)</th>
<th>Max Day Demand (MGD)</th>
<th>Percent Difference Based on Maximum Day</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012</td>
<td>16.2</td>
<td>28.1</td>
<td>16.2</td>
<td>28.1</td>
<td>-</td>
</tr>
<tr>
<td>2013</td>
<td>13.5</td>
<td>28.6</td>
<td>13.5</td>
<td>28.6</td>
<td>-</td>
</tr>
<tr>
<td>2014</td>
<td>12.1</td>
<td>24.1</td>
<td>12.1</td>
<td>24.1</td>
<td>-</td>
</tr>
<tr>
<td>2015</td>
<td>11.4</td>
<td>19.3</td>
<td>11.4</td>
<td>19.3</td>
<td>-</td>
</tr>
<tr>
<td>2020</td>
<td>18.0</td>
<td>31.4</td>
<td>14.6</td>
<td>27.7</td>
<td>-12%</td>
</tr>
<tr>
<td>2025</td>
<td>18.9</td>
<td>33.2</td>
<td>15.7</td>
<td>29.8</td>
<td>-10%</td>
</tr>
<tr>
<td>2030</td>
<td>19.8</td>
<td>34.9</td>
<td>16.9</td>
<td>32.0</td>
<td>-8%</td>
</tr>
<tr>
<td>2035</td>
<td>21.0</td>
<td>36.4</td>
<td>18.4</td>
<td>35.1</td>
<td>-4%</td>
</tr>
<tr>
<td>2040</td>
<td>21.7</td>
<td>37.7</td>
<td>19.6</td>
<td>37.3</td>
<td>-1%</td>
</tr>
<tr>
<td>2045</td>
<td>22.3</td>
<td>39.0</td>
<td>20.8</td>
<td>39.6</td>
<td>1%</td>
</tr>
<tr>
<td>2050</td>
<td>22.8</td>
<td>40.0</td>
<td>22.0</td>
<td>41.8</td>
<td>5%</td>
</tr>
<tr>
<td>2055</td>
<td>23.3</td>
<td>40.9</td>
<td>23.2</td>
<td>44.1</td>
<td>8%</td>
</tr>
</tbody>
</table>

1 Represents actual use.
2 Appears to be an anomaly because records show only a 2% water loss compared the lowest number in the
last 10 year of 11.4% water loss in 2014.

As shown in Figure 3-12, the regression of the population-based demand curves corresponds well with historical water use data. Past planning with the population-based projections would have met all actual historical water use conditions except for a maximum demand that occurred in 2003. In contrast, the historical water use exceeds the regression of the land use-based demand projections indicating that heavy water use restrictions would have been required of customers if the land use projections had been relied upon.

The demand projections provided herein are based on current wholesale supply contracts or known modifications thereof and anticipated levels of industrial, commercial, and residential growth. If wholesale supply contracts or growth changes in the future, the demand projections and resulting capital improvements plan should be revisited with the most current information.
Figure 3-12. Comparison of Demand Projection Methods
4 Raw Water Supply and Transmission Assessment

The City's network of vertical wells (VWs), horizontal collector wells (HCWs), and water transmission mains comprise the raw water supply system and are the first step in supplying water to the City's customers. The purpose of this Section is to determine the reliable capacity of the City's raw water system as it exists today and to identify capital improvements necessary to meet the projected demands.

4.1 Background

4.1.1 Historical Water Supply Development

The City's principal supply was surface water until 1964, when the City began development of a vertical well field and WTP2. Initially, the City's well field consisted of four gravel packed wells (VW 1 through VW 4) constructed in the alluvium adjacent to the Kansas River two miles east of De Soto, Kansas (near River mile 26.5). Additional wells were constructed in 1976, 1978, and 1981 bringing the total to eleven wells (VW 1 through VW 11). In 1991, VW 3 and VW 4 were relocated (VW 3R and 4R), while VWs 1, 2, and 8 were abandoned in 2006, prior to construction of HCW 2.

Between 1964 and 1991, riverbed degradation of eight (8) feet in the Kansas River, adjacent to the well field, led to a corresponding decline of seven (7) feet in the non-pumping, static water level at the well field. This decline in water levels reduced vertical well capacities by 30 to 50 percent; assuming pumping water levels were to be maintained above the top of the well screen.

With the decline in aquifer saturated thickness and the associated lower yields from the VWs, the City began constructing HCWs to increase the City's raw water supply. HCWs allow for horizontal well screens to be placed at approximately seven feet above the base of the alluvial aquifer and extend beneath the Kansas River. The HCW effective radius and ability to induce well production water from the river allows for greater production yield than VWs.

The City's collector well field began with construction of HCW 1 in 1998 (near River Mile 30). In 2002, HCW 2 was constructed near VW 6. HCW 3 and HCW 4 were constructed near HCW 1 in 2004 and 2005, respectively.

Currently, the City obtains all of its raw water supply from the eight remaining VWs and four HCWs. In 2010, the Lake Olathe's surface water right was converted from municipal to recreational use. Figure 4-1 depicts the City's existing vertical and horizontal well fields.
Olathe, Kansas
Water Master Plan Update

Figure 4-1. Well Field Location Map

Map Feature Key

- Current City Limits

Raw Water Mains
- 8” - 10”
- 12” - 14”
- 16” - 20”
- 24”
- 30”
- 48”

Wells
- Horizontal Collector Well
- Vertical Well
- Abandoned Vertical Well

Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP , swisstopo, and the GIS User Community
4.1.2 Water Rights Overview

On June 28, 1945, the State of Kansas enacted the Kansas Water Appropriation Act, which allows the State to conserve, protect, control, and regulate the use, development, diversion, and appropriation of water for beneficial and public purposes. Under the Water Appropriation Act, the right to use water is based on a “first in time, first in right” priority system. The law is administered by the Kansas Department of Agriculture’s Division of Water Resources (DWR). The DWR issues permits to appropriate water, regulates usage, and keeps records of all water rights in the state.

Vested and appropriated water rights, as used in the Water Appropriation Act, are two types of water rights defined as follows:

- Vested Right – the right to continue use of water having been used for a beneficial use on or before June 28, 1945, when the Kansas Water Appropriation Act became effective.
- Appropriated Rights – water rights applied for after creation of the Kansas Water Appropriation Act and have relative priority based on a priority date.

During times of water shortage, the water use of junior appropriators is curtailed while senior appropriators receive full allotment. Vested rights have priority over appropriated rights. All vested rights have the same priority date, and relative priority amongst vested rights is determined through adjudication.

Each water right has an annual quantity and maximum instantaneous pumping rate associated with it. With each new appropriation, the water right holder has a perfection period of up to 40 years (20 years plus extensions) to prove it can put the appropriated water to beneficial use. As the water right holder provides evidence that it has put the appropriated amounts of water to beneficial use, the water appropriation becomes certified. Should the water right holder not prove use of the full appropriated annual amount and instantaneous rate through the allotted perfection period, the water right is certified at the highest amount of use. Since the City has combined net limits, water rights need to be perfected in order of priority date.

In some instances, with a new appropriation, the individual appropriation is authorized for the full pumping capacity of the well, but the combined water rights may be limited so that the combined annual quantity and instantaneous rate does not exceed the projected demand for a 40-year planning horizon.

Water right allowances for maximum instantaneous rate will not necessarily match the maximum rate the physical infrastructure of the well is capable of. When rights are applied for, the application is submitted before the well is constructed and actual production rate is known. Common practice is to request the maximum rate the well might be capable of, as the requested rate and quantity can not be increased once the application has been accepted by the DWR. Through the perfection process, the well production capacity will need to be proven and DWR maximum rate allowed will be adjusted down to match the well’s maximum documented rate over the perfection period. Typically, the well’s maximum rate is proven when the well is in new condition and has not experienced age-related declines in performance. Therefore, it is not uncommon for DWR maximum instantaneous rate to allow higher production rates than a well is actually capable of producing. In uncertified water rights, maximum rate was overestimated and
the well was never capable of being produced at the rate originally requested on the water right application. In certified water rights, maximum rate was perfected when the well was in a new condition, but subsequently either the well’s production capacity has declined with age, or aquifer conditions have changed where the well can no longer produce at the DWR maximum allowed rate. Should the DWR-allowed maximum instantaneous rate be much lower than what the well is physically capable of producing, an additional new water right for rate only could be applied for to allow the well to produce at its maximum capacity.

For example, the water rights applications for HCW 1 requested a rate 12,000 gpm based on the preliminary aquifer siting study. Through the perfection process 10,500 gpm was determined as the highest rate the well had ever been pumped and the water right was certified with a rate of 10,500 gpm. In the well’s current condition, HCW 1 is only capable of pumping 8,500 gpm.

The existing water rights for the City’s VWs and HCWs are summarized in Table 4-1. Water Rights Information Reports (WRIR) for the water rights listed in Table 4-1 are included in Appendix B. It should be noted that groundwater near the well field had been fully appropriated with the water right on the vertical well field. All of the water rights for the groundwater component of the collector wells and any new rights the City applies for are contingent upon being “backed up” by surface water in the Kansas River through the Kansas River Water Assurance District #1.
### Table 4-1. Existing Water Right Summary

<table>
<thead>
<tr>
<th>Water Right Number</th>
<th>Priority Date</th>
<th>Well</th>
<th>Water Source</th>
<th>Water Right Status 1</th>
<th>Annual Quantity (AF/yr) 2</th>
<th>Net Annual Quantity (AF/yr) 3</th>
<th>Maximum Instantaneous Rate (gpm) 4</th>
<th>Net Combined Rate (gpm) 5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Vertical Wells</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10,042</td>
<td>5/18/1964</td>
<td>VW 3R</td>
<td>Ground</td>
<td>Certified</td>
<td>6,094.559</td>
<td>6,094.559</td>
<td>730</td>
<td>5,215</td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 4R</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>775</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1,045</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1,035</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1,005</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>830</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>VW 10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>770</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>VW 11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>800</td>
<td></td>
</tr>
<tr>
<td><strong>Horizontal Collector Wells</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>42,541</td>
<td>11/21/1996</td>
<td>HCW 1</td>
<td>Ground</td>
<td>Certified</td>
<td>674.848</td>
<td>674.848</td>
<td>10,500</td>
<td>10,500</td>
</tr>
<tr>
<td>42,542</td>
<td>11/21/1996</td>
<td>HCW 2</td>
<td>Surface</td>
<td>Certified</td>
<td>6,825.205</td>
<td>6,825.205</td>
<td>7,000</td>
<td>7,000</td>
</tr>
<tr>
<td>44,613</td>
<td>4/9/2001</td>
<td>HCW 3</td>
<td>Ground</td>
<td>Not Perfected</td>
<td>525.001</td>
<td>525.001</td>
<td>7,000</td>
<td>7,000</td>
</tr>
<tr>
<td>44,614</td>
<td>4/9/2001</td>
<td>HCW 4</td>
<td>Surface</td>
<td>Not Perfected</td>
<td>4,725</td>
<td>4,725</td>
<td>7,000</td>
<td>7,000</td>
</tr>
<tr>
<td>45,648</td>
<td>9/8/2003</td>
<td>HCW 5</td>
<td>Surface</td>
<td>Not Perfected</td>
<td>4,725</td>
<td>776.53</td>
<td>7,000</td>
<td>7,000</td>
</tr>
<tr>
<td>45,649</td>
<td>9/8/2003</td>
<td>HCW 5</td>
<td>Ground</td>
<td>Not Perfected</td>
<td>525.001</td>
<td>0</td>
<td>7,000</td>
<td>7,000</td>
</tr>
<tr>
<td>45,993</td>
<td>7/15/2004</td>
<td>HCW 6</td>
<td>Ground</td>
<td>Not Perfected</td>
<td>399.999</td>
<td>399.999</td>
<td>4,500</td>
<td>4,500</td>
</tr>
<tr>
<td>45,999</td>
<td>7/15/2004</td>
<td>HCW 6</td>
<td>Surface</td>
<td>Not Completed</td>
<td>6,800.001</td>
<td>1,909.308</td>
<td>4,500</td>
<td>4,500</td>
</tr>
<tr>
<td>47,202</td>
<td>11/18/2008</td>
<td>HCW 5</td>
<td>Ground</td>
<td>Not Completed</td>
<td>299.999</td>
<td>299.999</td>
<td>9,500</td>
<td>9,500</td>
</tr>
<tr>
<td>47,203</td>
<td>11/18/2008</td>
<td>HCW 6</td>
<td>Surface</td>
<td>Not Completed</td>
<td>7,000.009</td>
<td>7,000.009</td>
<td>9,500</td>
<td>9,500</td>
</tr>
</tbody>
</table>

1. “Certified” appropriations are those where use amounts have been proven or perfected, and the quantity is set. “Not perfected” appropriations are those that need to prove the initial appropriated amount within a 40-year period, in which they will become certified for the appropriation amount or the highest use, whichever is lower. “Not Completed” appropriations are those where the well has not been put online.
2. Annual quantity allowed with the water appropriation.
3. Effective total annual quantity gained with this appropriation when combined with previous existing appropriations.
4. Maximum instantaneous pumping rate allowed with the water appropriation. In appropriations covering multiple wells, maximum instantaneous rate has been divided between authorized wells.
5. Effective total maximum instantaneous pumping rate when combined with previous existing appropriations.
6. The City has water rights for the future HCW 5, which has not yet been constructed.
4.1.3 Perfection and Certification of Water Rights

Currently, the vertical well field and HCW 1 have certified associated water rights. Table 4-2 details the water rights for active wells that still require perfection. The HCWs 2, 3, and 4 are in the original 20-year perfection period, which can be extended up to 40 years total.

Table 4-2. Collector Wells Water Rights to be Perfected

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Water Right No.</th>
<th>Current Perfection Deadline</th>
<th>Quantity (AF/yr)</th>
<th>Rate (gpm)</th>
<th>Net Quantity 1 (AF/yr)</th>
<th>Highest Annual Use 2 (AF/yr)</th>
<th>Highest Recorded Rate 3 (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 2</td>
<td>44,613</td>
<td>12/31/2021</td>
<td>525.001</td>
<td>7,000</td>
<td>525.001</td>
<td>3,187.75 (2003)</td>
<td>4,800</td>
</tr>
<tr>
<td></td>
<td>44,614</td>
<td>12/31/2021</td>
<td>4,725</td>
<td>7,000</td>
<td>4,725</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW 3</td>
<td>45,648</td>
<td>12/31/2024</td>
<td>4,725</td>
<td>7,000</td>
<td>776.53</td>
<td>4,512.15 (2011)</td>
<td>6,033</td>
</tr>
<tr>
<td></td>
<td>45,649</td>
<td>12/31/2024</td>
<td>525.001</td>
<td>7,000</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW 4</td>
<td>45,993</td>
<td>12/31/2024</td>
<td>399.999</td>
<td>4,500</td>
<td>399.999</td>
<td>4,386.63 (2007)</td>
<td>4,833</td>
</tr>
<tr>
<td></td>
<td>45,994</td>
<td>12/31/2024</td>
<td>6,800.001</td>
<td>4,500</td>
<td>1,909.308</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 Net quantity determined by not to exceed combined limit, minus previous authorized municipal water rights (13,594.612 AF for Rights #10,042, #42,541 and #42,542).
2 Based on annual water use reports through Water Use Year 2014.
3 Based on records provided by City of Olathe.
4 Water Rights #44,613 and #44,614 combined will not exceed 7,000 gpm instantaneous rate, and will provide a total not to exceed 19,621.14 AF/yr for municipal purposes when combined with earlier water rights.
5 Water Rights #45,648 and #45,649 combined will not exceed 7,000 gpm instantaneous rate, and will provide a total not to exceed 19,621.14 AF/yr for municipal purposes when combined with earlier water rights.
6 Water Rights #45,993 and #45,994 combined will not exceed 4,500 gpm instantaneous rate, and will provide a total not to exceed 21,930.447 AF/yr for municipal purposes when combined with earlier water rights.

Perfection of Instantaneous Rate. HCWs maximum instantaneous rate is based on multiple variables such as river water stage, scouring of the river bed, and river water temperature. Pumping rates during scheduled inspections may not reflect the maximum rate the HCW can achieve. Most commonly, the highest reported instantaneous rate will occur early in the HCWs pumping history before capacity has declined with seasoning. Perfection of maximum instantaneous rate can come from validated sources such as consultant or contractor reports, SCADA records, or routine well field operations measurements.

Instantaneous rate water rights requested on HCWs 1, 2, and 3 were greater than maximum rates predicted from preliminary siting aquifer tests. Maximum yield for a planned HCW is estimated based on a vertical well pumping test performed at one-tenth the rate of the HCW potential yield. There is uncertainty when upsizing yield projections to the HCW.

Water appropriations must be procured before the City is allowed to construct the HCW. Once the water right application is accepted by the DWR, the rate and quantity of that water right generally cannot be increased, so the City typically applies for a higher rate than is expected, as the rate will be adjusted downward in the perfection process. The requested maximum instantaneous rates for HCWs 1, 2, and 3 were higher than the constructed HCW were capable.
As with perfection process for HCW 1, it is expected that HCWs 2 and 3 will be certified for a lower maximum instantaneous rate than currently allowed by water rights. In the case of HCW 4, the instantaneous rate water rights requested was lower that the maximum rate for which the HCW is capable. An addition junior water right for rate only can be combined with the existing rights for HCW 4 to allow the City to pump this well to its maximum rate.

It is advisable to pump at and document maximum instantaneous rate early in the lifespan of an HCW, as maximum pumping rates can decline early with well seasoning. Presently, DWR has not combined limitations on rate for multiple HCWs. If the perfected maximum instantaneous rate is higher than what the well is expected to produce during its lifespan this does reduce the City’s combined allotment of rights. Should DWR change this practice in the future, the City can voluntarily reduce allowed maximum instantaneous rate to better reflect the aged condition of the HCW.

The current allowable maximum rate by DWR for HCW 2 is 7,000 gpm. The highest documented rate for HCW 2 through 2016 was 4,800 gpm. It was expected HCW 2 should have been capable of 7,000 gpm by that time. Presently, this well is expected to provide a maximum rate of 4,500 gpm under average river stage in summer assuming it has been recently cleaned. Without improvements to the stream channel (discussed in further detail in Section 4.6.4), pumping at a rate higher than 4,800 gpm might be achieved when river water temperature is warmest, the river stage is high, and HCW 2 and the verticals wells have been rested to allow aquifer levels to rebound.

The current DWR allowed maximum instantaneous rate for HCW 3 is 7,000 gpm. The highest documented rate for HCW 3 was 6,033 gpm, which occurred on September 29, 2004 when the HCW first came online. This was before well specific capacity declined with seasoning. Presently, this well is expected to provide a maximum rate of 5,000 gpm under average river stage in summer assuming it has been recently cleaned. Without improvements to the stream channel (discussed in further detail in Section 4.6.4), pumping at a rate higher than 6,000 gpm might be achieved when river water temperature is warmest, the river stage is high, and HCW 1 is offline.

The current DWR allowed maximum instantaneous rate for HCW 4 is 4,500 gpm. The highest documented rate for HCW 4 was 4,833 gpm on June 26, 2012 after the HCW was cleaned for the first time. This is higher than the DWR allowed maximum instantaneous rate. Presently, this well is expected to provide a maximum rate of 4,000 gpm under average river stage in summer assuming it has been recently cleaned. Without improvements to the stream channel (discussed in further detail in Section 4.6.4), pumping at a rate of 5,500 gpm and higher might be achieved when river water temperature is warmest, the river stage is high, and HCW 1 is offline. The DWR limitation of 4,500 gpm would need to be requested to match the water right with the collector well capacity potential.

Perfection of Annual Quantity. HCW 1 did not have a combined annual quantity limitation, but all of the water rights obtained following HCW 1 (i.e., HCW 2, 3, and 4) have had a combined annual quantity. With the cumulative limitations imposed, each successive right is limited in combination with the rights preceding it; water rights will need to be perfected in the order in which applications were submitted.
The next water rights to be perfected are for HCW 2. The City will need to pump the collector well at 5,250 AF (1,710.7 MG) in a single year in order to maximize the right. Given the production capacity of HCW 2, maintaining an average of 3,255 gallons per minute continuously for a year does not appear feasible during a dry year. It is best to attempt this production following cleaning of the well. Furthermore, HCW 2 is limited in combination with previous municipal water rights to an annual quantity of 19,621.14 AF (6,393.6 MG). This limitation is greater than the combined individual annual quantities on the vertical well field, HCW 1 and HCW 2 because the limitation originally included an amount for Lake Olathe rights. The City will need to try and produce up to maximum allowed annual quantities for the vertical well field, HCW 1, and HCW 2 within a single year to maximize this limitation.

HCW 3 will also need to be perfected for annual quantity on HCW 3 alone and in combination with the previous water rights. Any total combined quantity not perfected for HCW 2, may be perfected with HCW 3 since HCW 3 is limited to the same 19,621.14 AF in combination with the VWs, HCW 1, and HCW 2. Similar to HCW 2, HCW 3 has a maximum annual quantity of 5,250 AF. It will need to maintain an average of 3,255 gpm for a year in order to get this well certified at its maximum annual quantity.

HCW 4 water rights present a challenge with perfecting the full annual quantity. For HCWs 1 through 3, allowed maximum instantaneous rate is approximately 2.2 times annual allowed quantity (on a gpm basis) to allow for flexibility in operation which accounting for peak production needs. The water rights on HCW 4 have an allowed maximum instantaneous rate of 4,500 gpm that is equal to the annual allowed quantity of 7,200 AF (on a gpm basis). In order to maximize the HCW 4 water rights, the well will need to run at 4,500 gpm (the maximum allowed instantaneous rate) for 99.2 percent of the year.

The City's certified water rights are typically based on documented highest historical annual use and maximum rate. The City’s un-perfected rights are typically based on a projected maximum annual quantity and maximum rate that the City might be able to pump the wells. The unperfected value should only be considered as possible allowed pumping within the DWR regulator framework, and not a metric of actual well field yield or capacity.

4.1.4 Kansas River Water Assurance District #1

In 1996 the Kansas Legislature passed the Water Assurance Program Act. The Act provided the foundation for the formation of three river Water Assurance Districts in Kansas. Members of Water Assurance Districts include municipal and industrial water rights holders from a river and its connected alluvial aquifer below a Federal reservoir in which the State has purchased storage. The Assurance District members pool resources to pay for State-owned reservoir storage space in a quantity that should yield enough river flow to cover Assurance members’ water rights during a two percent occurrence drought. During times of drought, the State releases water stored in reservoirs for use by downstream members of the Water Assurance Districts.

The City of Olathe is a member of the Kansas River Water Assurance District #1 (KRWAD #1), along with other users along the Kansas River including Junction City, Manhattan, Topeka, Lawrence, Kansas City Board of Public Utilities, and WaterOne. The
KRWAD #1 has storage space in Milford, Tuttle Creek and Perry Reservoirs allowing inflow into the dams to be stored via reservation rights. Water is released from storage when the flow at the United States Geological Survey (USGS) stream flow gage at the Wyandotte Street Bridge near De Soto falls below the target flows presented in Table 4-3.

Table 4-3. Kansas River Target Flows at De Soto Bridge

<table>
<thead>
<tr>
<th>Tuttle Creek Reservoir Pool Elevation</th>
<th>Summer Target Flow (cfs) May - October</th>
<th>Winter Target Flow (cfs) November – April</th>
</tr>
</thead>
<tbody>
<tr>
<td>Above 1070 feet</td>
<td>1,000</td>
<td>1,000</td>
</tr>
<tr>
<td>1065 – 1070 feet</td>
<td>1,000</td>
<td>800</td>
</tr>
<tr>
<td>Below 1065 feet</td>
<td>750</td>
<td>700</td>
</tr>
</tbody>
</table>

1 Winter flows can temporarily drop lower than target values due to icing of the river.

Once released, this water is available to the members of KRWAD #1 either by direct diversion of surface water or withdrawal via VWs or HCWs. In this manner, membership in the KRWAD #1 provides reliability for water supply in the event of a drought. While the KRWAD #1 provides assurance that full water rights should be available during a drought of the magnitude of two percent occurrence or less, the KRWAD #1 does not guarantee water will always be available.

Being a member of KRWAD #1 allows the City to continue to apply for new groundwater rights, even though groundwater rights are no longer available within a 2-mile radius of the VWs. This is because of annual volume being pumped from the VWs. DWR uses the concept of “safe yield” when doing water balance accounting for how much groundwater rights can be given out in a 2-mile radius. For the Kansas River, they take the average amount of annual precipitation expect to infiltrate into the aquifer over the 2-mile radius, then limit total volume of groundwater appropriations within the 2-mile radius to 75% of that total.

4.2 Existing Raw Water Supply Capacity

The reliable capacity of the City’s well field is affected by many factors including aquifer capacity; seasonal impacts; well monitoring, operations, and condition; infrastructure reliability; and water rights limitations. This Section presents the current raw water supply capacity based on these factors.

4.2.1 Aquifer Capacity

Aquifer capacity is estimated based on how well the aquifer is capable of supplying groundwater to a well. The current well condition is not considered in estimations of aquifer capacity. Aquifer capacity depends on the aquifer’s hydraulic conductivity, saturated thickness, and recharge. With riverbank filtration well fields, such as the City’s, a hydraulic connection between the river and the alluvial aquifer is essential in maximizing the aquifer capacity.

Vertical Wells. Hydraulic conductivity is high across the entire well field but appears higher at the VWs in the center of the well field (VWs 3, 4, 5, 6, 11) and lower at those on
the south end (VWs 1, 2, 10) and the north end (VWs 7, 8, 9). The VWs with the greatest saturated thickness are located within a west to east trough running from HCW 1 to the southern two-thirds of the vertical well field in which the bedrock channel at the base of the alluvial aquifer is deepest. Under average conditions, the entire well field is expected to benefit from induced recharge from the Kansas River, bounding the well field on the east. However, during periods of low river flow (less than 1,000 cfs) the VWs further inland benefit less from induced recharge than those directly adjacent to the river. Table 4-4 presents the aquifer capacity parameters of the wells when originally constructed.

Table 4-4. Aquifer Capacity Parameters – Vertical Wells

<table>
<thead>
<tr>
<th>Well Location</th>
<th>Depth to Bedrock (ft)</th>
<th>Saturated Thickness (ft)</th>
<th>Hydraulic Conductivity (gpd/ft²)</th>
<th>Aquifer Transmissivity (gpd/ft)</th>
<th>Aquifer 100% Available Yield (gpm)</th>
<th>Aquifer 100% Available Yield (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>VW 1</td>
<td>66</td>
<td>46.1</td>
<td>4,592</td>
<td>211,700</td>
<td>1,746</td>
<td>2.5</td>
</tr>
<tr>
<td>VW 2</td>
<td>62</td>
<td>42.0</td>
<td>4,304</td>
<td>180,773</td>
<td>1,512</td>
<td>2.2</td>
</tr>
<tr>
<td>VW 3</td>
<td>59</td>
<td>38.4</td>
<td>6,008</td>
<td>230,713</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VW 3R</td>
<td>60.5</td>
<td>32.1</td>
<td>6,118</td>
<td>196,396</td>
<td>1,380</td>
<td>2.0</td>
</tr>
<tr>
<td>VW 4</td>
<td>55.5</td>
<td>37.3</td>
<td>4,247</td>
<td>158,400</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VW 4R</td>
<td>60</td>
<td>26.0</td>
<td>5,385</td>
<td>140,000</td>
<td>1,430</td>
<td>2.1</td>
</tr>
<tr>
<td>VW 5</td>
<td>64</td>
<td>36.3</td>
<td>6,253</td>
<td>227,000</td>
<td>1,547</td>
<td>2.2</td>
</tr>
<tr>
<td>VW 6</td>
<td>65</td>
<td>40.2</td>
<td>5,124</td>
<td>206,000</td>
<td>1,660</td>
<td>2.4</td>
</tr>
<tr>
<td>VW 7</td>
<td>54</td>
<td>26.3</td>
<td>3,627</td>
<td>95,400</td>
<td>1,440</td>
<td>2.1</td>
</tr>
<tr>
<td>VW 8</td>
<td>52.5</td>
<td>25.5</td>
<td>3,757</td>
<td>95,800</td>
<td>1,200</td>
<td>1.7</td>
</tr>
<tr>
<td>VW 9</td>
<td>51</td>
<td>24.8</td>
<td>5,403</td>
<td>134,000</td>
<td>1,144</td>
<td>1.6</td>
</tr>
<tr>
<td>VW 10</td>
<td>63</td>
<td>35</td>
<td>4,189</td>
<td>146,600</td>
<td>1,368</td>
<td>2.0</td>
</tr>
<tr>
<td>VW 11</td>
<td>62</td>
<td>34</td>
<td>2,500</td>
<td>85,000</td>
<td>1,904</td>
<td>2.7</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>11,692</strong></td>
<td><strong>16.8</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 Original saturated thickness at construction.
2 Original well plugged – see Appendix D for further details.
3 The aquifer testing results for transmissivity at VW 11 are suspect of being less than half actual, given how this well has performed.

Horizontal Collector Wells. The highest yielding locations for the City’s HCWs are where the thickest, best aquifer formation material is located adjacent to the cutting bank of the river. For the City’s HCWs, the aquifer hydraulic conductivity is higher along an east-west band extending from HCW 1 to HCW 2. This band coincides with the greatest depth to bedrock. The greater hydraulic conductivity is most likely related to having a higher percentage of gravel and cobbles found in the deepest portion of the alluvial aquifer. Table 4-5 lists aquifer parameters of the City’s HCWs when originally installed.
### Table 4-5. Aquifer Capacity Parameters – Horizontal Collector Wells

<table>
<thead>
<tr>
<th>Well Location</th>
<th>Depth to Bedrock (feet)</th>
<th>Original Saturated Thickness ¹ (feet)</th>
<th>Hydraulic Conductivity ² (gpd/ft²)</th>
<th>Aquifer Transmissivity ¹ (gpd/ft)</th>
<th>Aquifer Available Yield (gpm)</th>
<th>Aquifer Available Yield (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 1</td>
<td>70</td>
<td>37</td>
<td>4,963</td>
<td>184,000</td>
<td>24,000</td>
<td>34.6</td>
</tr>
<tr>
<td>HCW 2</td>
<td>66</td>
<td>36</td>
<td>4,115</td>
<td>148,000</td>
<td>5,500</td>
<td>7.9</td>
</tr>
<tr>
<td>HCW 3</td>
<td>60</td>
<td>36</td>
<td>5,500</td>
<td>198,000</td>
<td>8,250</td>
<td>11.9</td>
</tr>
<tr>
<td>HCW 4</td>
<td>64</td>
<td>37</td>
<td>3,125</td>
<td>116,000</td>
<td>12,600</td>
<td>18.1</td>
</tr>
</tbody>
</table>

¹ Based on initial performance testing.
² Based on vertical well pumping tests conducted at the location prior to collector well construction.

#### 4.2.2 Seasonal Impacts

The majority of water produced at the City’s HCWs comes from induced infiltration of river water through the river bed. Therefore, HCW yield is expected to vary with Kansas River stage and river water temperature.

**River Temperature.** River water temperature plays an important role with horizontal collector well capacity. Warmer water has a lower viscosity and density, allowing it to flow through the aquifer material easier. Records of Kansas River water temperature and raw water temperature collected at the City’s HCWs indicate that water temperature at the collector wells are very close to that in the River. It is assumed for each degree Fahrenheit the water is warmer specific capacity will increase by 1.5 percent.

Figure 4-2 shows the annual temperature variation of the Kansas River at the USGS De Soto gage for data collected between June 1999 and June 2014. Figure 4-3 shows the corresponding specific capacity multiplier based on an annual average river temperature of 59.3 degrees Fahrenheit.

Based on changes in River temperature alone, the collector wells are anticipated to be capable of the highest specific capacities (gpm/ft of drawdown) during periods that coincide with the highest City demand.
Figure 4-2. Annual Temperature Variation – Kansas River Water

Figure 4-3. Water Temperature Related Change in Specific Capacity
River Stage. Historically, river flow at the well field has varied greatly with periods of flood and drought. After severe flood and droughts in the early to mid-1900s, reservoirs were built on the tributaries to the Kansas River to moderate flows, particularly flood flows. With the inception of KRWAD #1 in 1996, river flows were further moderated to maintain minimum flows at the Wyandotte Street Bridge. Figure 4-4 presents average and minimum Kansas River flows at the De Soto gage for three periods of record: the historic period of record, post completion of Milford Reservoir, and post operation of the KRWAD #1.

Average river flow at the USGS gage over the three periods of record has stayed fairly consistent. However, development of the reservoirs on the tributaries to the river has evened out the high and low flow periods. Establishment of the KRWAD #1 has minimized the occurrence of low Kansas River flows below 700 cfs in the winter; and has maintained flows above 1,000 cfs during the summer months, when municipal water use is high. As presented in Figure 4-4, flow targets at the De Soto gage have mostly been met or exceeded since KRWAD #1 began operation.

Seasonal variations of river stage also creates variance in the saturated thickness of the alluvial aquifer adjacent to the river.

Table 4-6. Seasonal Observed Capacity Parameters (Horizontal Collector Wells)

<table>
<thead>
<tr>
<th>Well Location</th>
<th>Depth to Bedrock (feet)</th>
<th>Original Saturated Thickness * (feet)</th>
<th>Hydraulic Conductivity * (gpd/ft²)</th>
<th>Aquifer Transmissivity * (gpd/ft)</th>
<th>Average Winter Capacity (gpm)</th>
<th>Average Winter Capacity (MGD)</th>
<th>Average Summer Capacity (gpm)</th>
<th>Average Summer Capacity (MGD)</th>
<th>Average Drought Capacity (gpm)</th>
<th>Average Drought Capacity (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 1</td>
<td>70</td>
<td>37</td>
<td>4,963</td>
<td>184,000</td>
<td>4,000 [5.8]</td>
<td>7,600 [10.9]</td>
<td>6,912 [9.95]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW 2</td>
<td>66</td>
<td>36</td>
<td>4,115</td>
<td>148,000</td>
<td>1,400 [2.0]</td>
<td>3,600 [5.2]</td>
<td>2,888 [4.16]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW 3</td>
<td>60</td>
<td>36</td>
<td>5,500</td>
<td>198,000</td>
<td>2,700 [3.9]</td>
<td>4,500 [6.5]</td>
<td>4,015 [5.78]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW 4</td>
<td>64</td>
<td>37</td>
<td>3,125</td>
<td>116,000</td>
<td>2,700 [3.9]</td>
<td>4,500 [6.5]</td>
<td>3,427 [4.93]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>10,300 [14.8]</strong></td>
<td><strong>20,200 [29.1]</strong></td>
<td><strong>17,242 [24.82]</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Reliability of KRWAD #1 Flow Supplementation. As mentioned in Section 4.1.4, KRWAD #1 releases flows from reservoir storage to meet certain flow targets at the USGS gage at De Soto depending on time of year and water elevation in Tuttle Creek Reservoir. Since Tuttle Creek Reservoir was filled in 1963, it has only been below an elevation of 1065 feet msl (the tipping point for lowest target flows at De Soto) during two periods: November 19, 1966 to April 14, 1967 and October 15, 2012 to April 10, 2013. Both periods were during the winter lower target flow periods. During the second period, daily average river flow dropped below the winter target flow of 700 cfs multiple times, and as low as 546 cfs. During August 2012 through April 2013, KRWAD #1 was supplementing flow with releases of up to 235 cfs continuous flow to meet summer targets of 1,000 cfs.
Figure 4-4. Average and Minimum Kansas River Flows
4.2.3 Water Level Monitoring

The City maintains 20 piezometers throughout the well field area to monitor water levels within the VW field and adjacent to the HCWs. Figure 4-5 shows the location of Piezometers #1 through #5 in relation to property owned by the City and to property owned by David Penny (Masters Dredging). Piezometers #1 through #5 are set further back from the wells and provide data on the aquifer generally outside the influences of any particular well. Originally Piezometers #1 and #2 monitored changes in aquifer levels outside the vertical well field with pumping and river stage. Once the City began development of the HCWs, Piezometers #3, #4 and #5 began monitoring changes in the aquifer attributed to the new HCWs. Figure 4-6 shows aquifer water levels in relation to Kansas River flows since the collector wells were added to the City’s raw water supply.

Observations of the Piezometers are as follows:

- Piezometers #1 and #2 indicate that during periods of low flow (below 1,000 cfs) in the Kansas River, water levels in the aquifer near the VW field and HCW 2 can be reduced as much as ten feet, as observed in 2003 to 2004 and 2012 to 2014. The greater decline in 2003 to 2004 is attributed to higher pumping at both the vertical well field and HCW 2 during the time period. By 2012, the production capability at both the vertical well field and HCW 2 had declined from 2003 to 2004 due to seasoning of HCW 2 and removal of three VWs.

- Piezometers #4 and #5 show development of HCWs 1, 3, and 4 have caused a decline in water levels of one to two feet within the aquifer to the east of the HCWs during times of low river flow. Before the addition of HCWs 3 and 4, when HCW 1 was pumped, the aquifer to the east slowly declined approximately 2 feet, but rebounded when flows in the river increased. Once HCWs 3 and 4 were added, the aquifer did not rebound back as much with an increase in river flow. In 2012, a more dramatic lowering of the aquifer was observed when river flows dropped below 1,000 cfs, and then remained low throughout 2013 and 2014.

- Piezometer #3 is sufficiently far from the wells and close to the river that it is not substantially impacted by pumping in the well field. Approximately 0.3 feet of decline is observed at this location with development of HCWs 1, 3, and 4.
Olathe, Kansas
Water Master Plan Update

Figure 4-5. Piezometer Location Map
Figure 4-6. Aquifer Water Level with River Stage
4.2.4 Well Capacity Degradation

Drinking water production wells have a limited usable life span. Typically, well condition declines with age until the well can no longer produce to required demand or loses mechanical integrity and fails. Several factors may reduce well capacity throughout its lifespan including:

- Well screen and aquifer clogging due to fine particle impingement or mineral deposition (geochemical or biological). This results in lower specific capacity and may be remedied by regular mechanical and chemical cleaning. Wells should be cleaned before specific capacity has declined more than 30 percent for VWs and more than 20 percent for HCWs. For alluvial VWs, this is typically every 3 to 5 years. For HCWs, this is typically every 7 to 10 years. Once a vertical well is irreversibly clogged, the well must be replaced. For HCWs, additional laterals can be installed to compensate for capacity lost in existing clogged laterals.

- Wear and mechanical integrity failure of well components such as well screen, pump, or pump column. This can be from abrasion from fine particles, vibration of well components, damage from well cleaning activities, or corrosion. When a vertical well screen fails it can lead to either a full loss of the well or requires relining with smaller diameter casing or screen, which reduces capacity. When a HCW screen fails, an additional lateral can be installed.

- Clogging of the pump and/or pump column by mineral deposition (geochemical or biological). Deposition of iron and manganese oxides is common in alluvial wells. This results in a loss of well production rates without a loss in well specific capacity and is remedied by cleaning or replacement of the impacted components.

- Decline in aquifer saturated thickness through over-pumping of an aquifer or riverbed degradation of an adjacent stream. These can reduce both well specific capacity and the height of water above the well screen.

- Clogging of the river/aquifer interface. In riverbank infiltration well fields, such as the City’s, increased pumping and associated aquifer drawdown can cause fine particles to impinge on the riverbed surface and reduce infiltration of surface water into the aquifer. This is prevalent with high capacity collector wells and results in declining specific capacity for some period of time after the well goes online (i.e., seasoning effect). Some of this clogging may be scoured away during high river flows.
4.3 Vertical Wells

4.3.1 River Impacts

Riverbed degradation has had a major impact on aquifer capacity of the vertical well field. When the first VWs were developed in 1964, static water levels were 21 ft bgs (below ground surface) and saturated thickness was 35 to 45 feet. Over the next 30 years, the bottom of the riverbed degraded (lowered) eight feet, resulting in the static water level lowering seven feet. This riverbed degradation has also caused:

- A long-term reduction in aquifer capacity of 16 to 20 percent;
- Seven less feet of available drawdown above the top of the well screen; and
- A reduction of one-third to over one-half the individual well’s production capacity when operated under recommended pumping conditions (i.e., pumping only when water levels are above the screen).

The effects of varying river stage can cause the static water level in the aquifer to fluctuate as much as eight to ten feet, greatly influencing the available drawdown above the well screen and the available yield. When the VWs were originally constructed, the saturated thickness of the aquifer was sufficient for the wells to have 20-foot long well screens and still maintain pumping water levels above the top of the screen at flow rates upwards of 1,000 gpm. Currently, the VWs are being operated with pumping water levels within the screened interval due to a number of factors including declining saturated thickness of the aquifer caused by riverbed degradation, degrading well performance caused by age, and increased development of wells within the well field. Pumping within the screens is not a recommended practice as it promotes accelerated clogging of the wells. However, given the current well construction, it allows the City to gain an increase in production from the vertical well field during the periods of drought or high demand.

It is recommended that new replacement wells be designed with seven to twelve foot long, 36-inch diameter well screens to meet inflow velocity requirements for DWR maximum pumping rates and to have sufficient available drawdown above the top of the well screen.

Under normal circumstances, mutual interference between the VWs is minimal. During periods of low river flow (less than 1,000 cfs), particularly when the wells are pumped more often, a localized cone of drawdown develops around the well field resulting in lower yields, most likely due to induced infiltration, is lower under a low river stage condition. The addition of HCW 2 has compounded the effect. At periods when HCW 2 is pumping at rates greater than 3,000 gpm the mutual interference ranges from one foot or less at VW 8 to greater than 10 feet at VW 6.

When newly constructed the specific capacity at the VWs ranged from 75 to 177 gpm/ft. VWs 1, 2, 3 and 4 had lower specific capacities when constructed. This may be a function of the smaller well diameter and well construction method, since replacement wells VW 3R and 4R had specific capacities within 125 gpm/ft. VW 10 had a specific capacity of only approximately 80 gpm/ft, likely due to its location on the south end of the well field where hydraulic conductivity is lower. VWs 8 and 9 on the shallower north end
of the field had specific capacities of approximately 100 gpm/ft. Other wells in the center of the well field had specific capacities ranging between 100 and 175 gpm/ft (VW 5, 6, 7, and 11). The original well construction details are presented in Appendix C. Table 4-7 presents the vertical well details for the existing active wells.

Table 4-7. Vertical Well Construction Details (Existing Active Wells)

<table>
<thead>
<tr>
<th>Well Number</th>
<th>Date Drilled</th>
<th>Well Diameter (in)</th>
<th>Well Screen (ft)</th>
<th>Approx. Ground Surface Elevation (ft msl)</th>
<th>Top of Screen Elevation (ft msl)</th>
<th>Approximate Recent Non-Pumping Water Level Elevation (^1) (ft msl)</th>
<th>Original Specific Capacity (gpm/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>VW 3R</td>
<td>Sep-91</td>
<td>30</td>
<td>15</td>
<td>783</td>
<td>737.5</td>
<td>740 to 750</td>
<td>107.2</td>
</tr>
<tr>
<td>VW 4R</td>
<td>Jun-92</td>
<td>30</td>
<td>15</td>
<td>780</td>
<td>736.0</td>
<td>744 to 748</td>
<td>136.7</td>
</tr>
<tr>
<td>VW 5</td>
<td>Aug-76</td>
<td>26</td>
<td>20</td>
<td>781</td>
<td>739.0</td>
<td>743 to 750</td>
<td>147.0</td>
</tr>
<tr>
<td>VW 6</td>
<td>Aug-76</td>
<td>26</td>
<td>25</td>
<td>780</td>
<td>739.0</td>
<td>742 to 746</td>
<td>177.0</td>
</tr>
<tr>
<td>VW 7</td>
<td>Oct-78</td>
<td>30</td>
<td>23</td>
<td>782</td>
<td>750.1</td>
<td>747 to 751</td>
<td>92.3</td>
</tr>
<tr>
<td>VW 9</td>
<td>Mar-81</td>
<td>30</td>
<td>20</td>
<td>781</td>
<td>748.5</td>
<td>747 to 751</td>
<td>92.4</td>
</tr>
<tr>
<td>VW 10</td>
<td>Feb-81</td>
<td>30</td>
<td>25</td>
<td>781</td>
<td>742.8</td>
<td>743 to 747</td>
<td>78.3</td>
</tr>
<tr>
<td>VW 11</td>
<td>Feb-81</td>
<td>30</td>
<td>25</td>
<td>780</td>
<td>742.8</td>
<td>741 to 751</td>
<td>116.5</td>
</tr>
</tbody>
</table>

\(^1\) Based on high pumping periods and low flow river data collected by the City between 2011 and 2015.

Note: VWs 1, 2, and 8 were abandoned in 2006 prior to construction of HCW 2.

4.3.2 Condition Assessment

The following section details the analysis of available well performance data to estimate declines in specific capacity and production rates with time and capacity recovery when maintenance activities were performed.

The typical usable life of VWs is 20 to 30 years. In cases where wells are pumping at rates far exceeding mechanical capacity or where water is corrosive or prone to precipitating minerals, a well may last only 5 to 10 years. In some cases, wells that have been pumped at low inflow rates, have exceedingly good water quality, and are regularly maintained can provide reliable capacity up to 50 years.

The original VW 3 and VW 4 were replaced at approximately 27 years following construction. Abandoned wells VW 1, VW 2, and VW 8 were relined with smaller diameter screens two to three times throughout the 28 to 42 years following construction. This resulted in wells with 10-inch diameter well screen, which would require smaller diameter, lower capacity pumps.

Available data for the City’s VWs includes initial well performance tests performed when the wells were originally constructed, maintenance records from periodic well inspection and cleaning, and monthly water levels collected by City staff. Table 4-8 presents a summary of historic performance data for the active wells. Graphs of vertical well historic performance are included in Appendix D.
### Table 4-8. Vertical Wells – Historic Performance Data through 2014

<table>
<thead>
<tr>
<th>Well</th>
<th>Year Built</th>
<th>Last Cleaned</th>
<th>Original Specific Capacity (gpm/ft)</th>
<th>Highest Recorded Specific Capacity (gpm/ft)</th>
<th>2014 Recorded Specific Capacity (gpm/ft)</th>
<th>Potential Specific Capacity</th>
<th>Estimated Percent Decline from Screen Clogging</th>
</tr>
</thead>
<tbody>
<tr>
<td>VW 3R</td>
<td>Sep 1991</td>
<td>2009</td>
<td>107.2</td>
<td>160</td>
<td>94.5</td>
<td>110</td>
<td>14%</td>
</tr>
<tr>
<td>VW 4R</td>
<td>Jun 1992</td>
<td>2012</td>
<td>136.7</td>
<td>137</td>
<td>82.1</td>
<td>110</td>
<td>25%</td>
</tr>
<tr>
<td>VW 5</td>
<td>Aug 1976</td>
<td>2006</td>
<td>147.0</td>
<td>154</td>
<td>63.1</td>
<td>100</td>
<td>37%</td>
</tr>
<tr>
<td>VW 6</td>
<td>Aug 1976</td>
<td>2013</td>
<td>177.0</td>
<td>177</td>
<td>68.1</td>
<td>120</td>
<td>43%</td>
</tr>
<tr>
<td>VW 7</td>
<td>Oct 1978</td>
<td>2013</td>
<td>92.3</td>
<td>167</td>
<td>109.3</td>
<td>125</td>
<td>13%</td>
</tr>
<tr>
<td>VW 9</td>
<td>Mar 1981</td>
<td>2013</td>
<td>92.4</td>
<td>135</td>
<td>59.6</td>
<td>100</td>
<td>40%</td>
</tr>
<tr>
<td>VW 10</td>
<td>Feb 1981</td>
<td>2013</td>
<td>78.3</td>
<td>83</td>
<td>54.5</td>
<td>70</td>
<td>22%</td>
</tr>
<tr>
<td>VW 11</td>
<td>Feb 1981</td>
<td>2012</td>
<td>116.5</td>
<td>152</td>
<td>59.5</td>
<td>135</td>
<td>56%</td>
</tr>
</tbody>
</table>

1 Estimated potential specific capacity achievable based on historic specific capacities achieved following cleanings.

2 Estimated specific capacity decline from Potential Specific Capacity to 2014 Recorded Specific Capacity. Specific capacity of VWs should not be allowed to decline more than 30% between cleanings.

Overall, the VWs are regularly monitored for mechanical deficiencies and decline in specific capacity. The VWs are repaired and cleaned to regain lost capacity. However, at the time the wells were constructed the well screen were too long for the current aquifer thickness, and due to age, some of the wells no longer respond well to cleaning.

The following summarizes the condition of the VWs based on the available performance data as of 2014:

- VWs 3R, 4R, 7, and 11 are in decent operating condition considering age. VW 4R did not respond well to its 2012 cleaning. More aggressive cleaning may be required in the future. As of 2014, VW 11 has had significant decline in specific capacity since the most recent cleaning in 2012.

- VWs 5 and 6 appear to be suffering from age related capacity declines where cleaning is no longer effective in regaining well capacity. VW 5 needs to be cleaned, to recover some specific capacity, although it is not expected a great gain of specific capacity can be achieved with cleaning. VW 6 did not respond well to cleaning after it was relined in 2011. If construction of additional VWs is not an immediate option for providing increased capacity, then more aggressive cleaning may be required at VWs 5 and 6 to get through two layers of well screen and gravel pack.

- VWs 9 and 10 are experiencing production capacity issues, which may be related to the pumps. These wells require further inspection.

**Minimum Pumping Level.** The minimum pumping water level is used to estimate well capacity to ensure the pump level allows enough submergence of the pump intake based on the pump manufacturer’s recommendations. For the VWs, the recommendation to
keep the pumping water level above the top of the well screen will provide adequate submergence over the pumps in all cases.

Reliability of Mechanical and Electrical Components. The mechanical equipment (i.e., pumps, piping, and other appurtenances) of a well must be capable of withdrawing water regardless of the aquifer capacity, its physical condition, and other factors discussed above. Pumps are the primary mechanical equipment to consider for the reliability of a vertical well.

Reliability of the well is also dependent on the electrical service and whether backup power is supplied. The reliability of the power supply to the City's VWs was evaluated. There is limited record information available regarding power feed to the VWs. It is known that there is no stand-by generator power for the vertical well field. Considering the lack of standby power at the VWs, it is assumed that if the electrical service is lost all VWs will be offline. It is, therefore, recommended to install a redundant power feed loop for the VW field.

In addition, a Report completed by Black & Veatch\(^2\) for WTP2 indicated the City experiences lightning strikes in the vertical well field which shuts down the VWs. This Report recommends installation of lightning protection facilities to increase the reliability of the water supply.

4.4 Horizontal Collector Wells

4.4.1 River Impacts

For HCWs, the A-distance represents how far the groundwater level drawdown cone from the well will extend underneath the riverbed and the areal extent of riverbed that is needed to supply recharge to the well given the infiltration capacity of the riverbed. A larger A-distance means river water is infiltrating more slowly or the source of surface water is further from the collector well. The highest yielding locations for the City's collector wells are where the thickest, best aquifer formation material is located adjacent to the cutting bank of the river.

The A-distance to recharge was calculated based on the initial performance tests performed for each HCW following construction. Initial performance testing at HCWs 1, 3 and 4 resulted in relatively low A-distance values, indicating a good connection of the collector well with the river and a relatively high infiltration rate through the stream bed. The apparent A-distance to recharge has since increased to approximately 1.3 to 2.5 times greater than when the HCWs were initially installed. Table 4-9 presents the HCW well details including estimated A-distance and specific capacity.

Following installation, HCW 2 had an initial higher apparent A-distance than expected given that it was located in the bedrock channel, adjacent to the Kansas River, and had a

fairly high aquifer transmissivity. This was also seen in pumping test data originally conducted on VW 6. It is suspected that horizontal clay or silt layers may exist east of HCW 2 impeding recharge from the riverbed to the collector well laterals. With a higher apparent A-distance and a lower flow rate, infiltration rates through the streambed adjacent to HCW 2 will be lower and seasoning is expected to be less.

Infiltration capacity of the riverbed adjacent to the HCWs can decrease over time as the HCWs are put into service and pumped. Pumping of a HCW causes river water to infiltrate into the stream bed at a higher rate, leading to silt and clay particle impingement and clogging of the porous riverbed materials over time. This is known as “seasoning” of a HCW. The clogging is expected to initially occur in the streambed near the collector well and spread outward until the clogging effect equalizes with the scouring ability of the river. At the City’s HCWs, this seasoning effect appears to happen over the first 3 to 4 years of operation.

Table 4-9. Horizontal Collector Well Details

<table>
<thead>
<tr>
<th>Well Number</th>
<th>Date Constructed</th>
<th>Number of Well Laterals</th>
<th>Number of Spare Lateral Ports</th>
<th>A-Distance to Recharge (^1) (feet)</th>
<th>Original Specific Capacity (^2) (gpm/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 1</td>
<td>Feb-98</td>
<td>5</td>
<td>2</td>
<td>120</td>
<td>753</td>
</tr>
<tr>
<td>HCW 2</td>
<td>Oct-02</td>
<td>4</td>
<td>2</td>
<td>340</td>
<td>196 (^3)</td>
</tr>
<tr>
<td>HCW 3</td>
<td>Oct-04</td>
<td>4</td>
<td>2</td>
<td>255</td>
<td>355</td>
</tr>
<tr>
<td>HCW 4</td>
<td>Jul-05</td>
<td>4</td>
<td>1</td>
<td>140</td>
<td>352</td>
</tr>
</tbody>
</table>

\(^1\) Based on back calculation using Hantush and Papadopulous (1962) from initial testing.

\(^2\) For a consistent comparison, specific capacity values have been corrected to 60 degrees Fahrenheit.

\(^3\) Extrapolated to steady-state conditions since drawdown at HCW 2 did not stabilize within the 72 hours.

4.4.2 Condition Assessment

The following section details the analysis of available well performance data to estimate declines in specific capacity and production rates with time and capacity recovery when maintenance activities were performed.

The usable life of a horizontal collector well is typically much longer than a vertical well. Some of the first HCWs installed in the 1930s are still in use today; although new laterals were required at some wells as old laterals have clogged or failed.

Available data for the City’s HCWs includes initial well performance tests done when the wells were originally constructed, maintenance records from periodic well inspection and cleaning, and monthly water levels collected by City staff. Specific capacity of the HCWs varies more than VWs as a function of river water temperature, river stage, and pumping rate. Table 4-10 presents a summary of historic performance data for the City’s active HCWs. Specific capacity data was normalized to a temperature of 60 degrees Fahrenheit, where enough data was available, to assess capacity declines. Graphs of horizontal collector well historic performance are included in Appendix D.
Table 4-10. Horizontal Collector Wells – Historic Performance Data through 2015

<table>
<thead>
<tr>
<th>Well</th>
<th>Year Built</th>
<th>Last Cleaned</th>
<th>Original Specific Capacity (gpm/ft)</th>
<th>Specific Capacity with Maximum Pumping (gpm/ft)</th>
<th>2015 Recorded Specific Capacity (gpm/ft)</th>
<th>Potential Specific Capacity (gpm/ft)</th>
<th>Estimated Percent Decline from Seasoning</th>
<th>Estimated Percent Decline from Screen Clogging</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 1</td>
<td>Feb 1998</td>
<td>Oct 2011</td>
<td>753</td>
<td>301</td>
<td>250</td>
<td>310</td>
<td>59%</td>
<td>19%</td>
</tr>
<tr>
<td>HCW 2</td>
<td>Oct 2002</td>
<td>Nov 2012</td>
<td>196</td>
<td>125</td>
<td>140</td>
<td>175</td>
<td>10%</td>
<td>20%</td>
</tr>
<tr>
<td>HCW 3</td>
<td>Oct 2004</td>
<td>Jan 2013</td>
<td>355</td>
<td>213</td>
<td>190</td>
<td>235</td>
<td>34%</td>
<td>19%</td>
</tr>
<tr>
<td>HCW 4</td>
<td>Jul 2005</td>
<td>Nov 2011</td>
<td>352</td>
<td>162</td>
<td>140</td>
<td>220</td>
<td>38%</td>
<td>36%</td>
</tr>
</tbody>
</table>

1 Based on initial HCW performance testing prior to seasoning (corrected to 60 degrees Fahrenheit).
2 Specific capacities observed July 19, 2013 representing low river flow, maximum HCW production, and minimum pumping water level in HCWs (i.e. resulting in lower specific capacity due to partial dewatering of aquifer).
3 Estimated potential specific capacities achievable by cleaning of laterals (corrected to 60 degrees Fahrenheit).
4 The difference between estimated potential specific capacity and original specific capacity indicates the seasoning effect.
5 Estimated specific capacity decline from Potential Specific Capacity (see Note 3) to 2015 Recorded Specific Capacity. Specific capacity of HCWs should not be allowed to decline more than 20% between cleanings.

Based on observations collected at the end of 2015, performance of the HCWs has declined by approximately 30 to 65 percent from original tested specific capacities. Two mechanisms are considered to contribute to decline in HCW capacity: clogging of the riverbed/aquifer interface and clogging of the well screen and aquifer adjacent to the screen.

As discussed previously, clogging of the riverbed/aquifer interface is referred to as seasoning of the HCW and appears to occur in the City’s HCWs over the first three to four years of operation. It is expected that specific capacity declines due to this clogging can only be reversed through scouring or resurfacing of the riverbed to remove the clogged layer. Based on the City’s HCW performance data in Table 4-10, the decline due to riverbed clogging can be estimated from original capacity to the potential capacity achieved by cleaning.

- **HCW 1** experienced an estimated decline due to riverbed clogging of nearly 60 percent; initially by 45 percent, and then another 10 to 15 percent when HCWs 3 and 4 were installed on either side of HCW 1. Construction of HCW 3 and 4 in the vicinity forced HCW 1 to recharge at a higher rate from a narrower swath of riverbed. While the 1,000 foot spacing of the collector wells seems to have only 1 foot or so of mutual interference from an aquifer drawdown perspective, the wells compete with each other for recharge through available riverbed area.

- **HCW 2** experienced an estimated decline due to riverbed clogging of 10 percent. However, with the vertical well field at full capacity HCW 2 and the well field are expected to compete for recharge through available riverbed area.
- HCW 3 experienced an estimated decline due to riverbed clogging of approximately 35 percent.
- HCW 4 experienced an estimated decline due to riverbed clogging of approximately 38 percent.

Clogging of the well screen and aquifer adjacent to the screen includes fine particle impingement and mineral incrustation, and occurs as a normal part of well operation. Higher dissolved iron and manganese concentrations tend to cause more incrustation. As discussed earlier, the City maintains a network of 20 piezometers to monitor the water levels in the well field. Fifteen of the piezometers are spaced close to the four HCWs to measure drawdown in the aquifer between the riverbed/aquifer interface and the HCW laterals to monitor progression of well decline. As the difference between drawdown in a HCW and associated piezometers for a given pumping rate increases (drawdown differential), it indicates lateral screen clogging. Effectiveness of cleaning can be determined by how close the drawdown differentials return to original.

Regular cleaning of the HCWs is expected to regain lost capacity. HCW 1 has been cleaned twice in its history; the remaining HCWs have only been cleaned once. It is recommended that the HCWs be cleaned once capacity of the well at the laterals has declined by 20 percent. As indicated in Table 4-10, based on estimated percent decline from potential specific capacity at the collector wells, the City should consider cleaning HCW 4 immediately and HCWs 1, 2, and 3 within the next two years.

As the HCW laterals age and cleaning becomes less effective at regaining well capacity, new laterals can be projected from the caisson to help regain lost HCW capacity. An extra port is no longer required for additional laterals, as the laterals can be bored through the caisson wall. However, the number of additional laterals and lateral spacing will be limited by the structural integrity of the caisson wall.

Minimum Pumping Level. The minimum pumping water level is used to estimate well capacity to ensure the pump level allows enough submergence of the pump intake based on the pump manufacturer’s recommendations.

For the HCWs, it is recommended the minimum pumping water level be set to 5 feet above the centerline of the well lateral. It was determined 5 feet above the centerline of the laterals is greater than the manufacturer’s minimum submergence requirement for the pumps and provides adequate submergence of the pumps. Table 4-11 shows the results of this analysis for the HCWs.

It is recommended that the lateral centerline elevations and minimum pumping water level elevations that are currently set in SCADA be confirmed to ensure the City is utilizing the HCWs most efficiently. If the controlling water level is set in SCADA at a higher elevation than those shown in Table 4-11, it is possible there is unutilized capacity within the laterals.
Table 4-11. Horizontal Collector Wells Allowable Minimum Pumping Water Level Analysis

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Pump ID</th>
<th>Pump Capacity (MGD)</th>
<th>Based on Lateral Elevation</th>
<th>Based on Pump Intake Elevation</th>
<th>Controlling Pumping Water Level (ft) 5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Centerline of Laterals Elev (ft)</td>
<td>Minimum Pumping Elev (ft) 4</td>
<td>Pump Intake Elev (ft)</td>
</tr>
<tr>
<td>HCW 1 1</td>
<td>P-1</td>
<td>5.0</td>
<td>722.6</td>
<td>727.6</td>
<td>722.00</td>
</tr>
<tr>
<td></td>
<td>P-2</td>
<td>5.0</td>
<td></td>
<td></td>
<td>722.00</td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td>5.0</td>
<td></td>
<td></td>
<td>722.00</td>
</tr>
<tr>
<td>HCW 2 2</td>
<td>P-1</td>
<td>3.5</td>
<td>722.42</td>
<td>727.42</td>
<td>723.98</td>
</tr>
<tr>
<td></td>
<td>P-2</td>
<td>5.0</td>
<td></td>
<td></td>
<td>724.03</td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td>3.5</td>
<td></td>
<td></td>
<td>723.98</td>
</tr>
<tr>
<td>HCW 3 3</td>
<td>P-1</td>
<td>3.5</td>
<td>728.42</td>
<td>733.42</td>
<td>729.60</td>
</tr>
<tr>
<td></td>
<td>P-2</td>
<td>4.0</td>
<td></td>
<td></td>
<td>729.60</td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td>3.5</td>
<td></td>
<td></td>
<td>729.60</td>
</tr>
<tr>
<td>HCW 4 2</td>
<td>P-1</td>
<td>3.5</td>
<td>726.07</td>
<td>731.07</td>
<td>727.50</td>
</tr>
<tr>
<td></td>
<td>P-2</td>
<td>4.0</td>
<td></td>
<td></td>
<td>727.50</td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td>3.5</td>
<td></td>
<td></td>
<td>727.50</td>
</tr>
</tbody>
</table>

1 Pump intake elevation is based on manufacturer’s O&M manual. The manufacturer’s recommended minimum pump submergence was not provided; therefore, it is assumed the minimum submergence is 3 feet.
2 Pump intake elevation and minimum pump submergence are based on manufacturer’s O&M manual.
3 Pump intake elevation was not available in the manufacturer’s O&M manual or other information on file; therefore, the intake elevation was assumed to be the same distance above the finished floor of the well caisson as HCW 4.
4 Established based on 5 feet above the centerline of the laterals.
5 Based on the larger value when comparing Minimum Pumping Elevation Based on Lateral Elevation versus Minimum Pumping Elevation Based on Pump Intake Elevation

Reliability of Mechanical and Electrical Components. The mechanical equipment (i.e., pumps, piping, and other appurtenances) of a well must be capable of withdrawing water regardless of the aquifer capacity, its physical condition, and other factors discussed above. Pumps are the primary mechanical equipment to consider for the reliability of a well. For the HCWs, each well has three vertical turbine pumps; two of the three pumps at each well are operated on VFDs.

Reliability of the well is also dependent on the electrical service and whether backup power is supplied. The reliability of the power supply to each of the HCWs was evaluated. A summary of the power supply follows:

- **HCW 1** has a single incoming power feed from the utility to the MCC. A 1500 kW stand-by generator is connected to the MCC and interlocked with the utility feed with an automatic transfer switch.
- **HCW 2** has a single incoming power feed from the utility to the switchboard. An 800 kW stand-by generator is connected to the switchboard and interlocked with the utility feed with electrically interlocked circuit breakers.

4-26
• HCW 3 has a single incoming power feed from the utility to the MCC. There is no stand-by generator.

• HCW 4 has a single incoming power feed from the utility to the MCC. There is no stand-by generator.

The power supply record drawing at each HCW was reviewed for single point of failure in the electrical service that would prevent use of an entire collector well during a power outage. At all wells the main MCC/switchboard bus is a single point of failure; however, it is rare to lose an entire MCC/switchboard. The following are the observations and potential solutions for electrical redundancy at each HCW:

• HCW 1 – there is a single point of failure at the circuit from the automatic transfer switch to the MCC. A solution to provide redundancy is to add interlocked circuit breakers in the MCC; however, the MCC would need to be evaluated to determine if it can be modified.

• HCW 2 – there are no single points of failure.

• HCW 3 – there is a single point of failure at the circuit between the transformer and the MCC. It is recommended to install a generator that is electrical interlocked with the utility via circuit breakers in the MCC. The MCC would need to be evaluated to determine if it can be modified.

• HCW 4 – there is a single point of failure at the circuit between the transformer and the MCC. It is recommended to install a generator that is electrical interlocked with the utility via circuit breakers in the MCC. The MCC would need to be evaluated to determine if it can be modified.

A Report completed by Black & Veatch\(^3\) for WTP2 identifies several of the recommendations listed as capital projects. This Report recommends that generators be installed at HCW 3 and 4 and installation of lightning protection at the HCW field.

4.5 Summary of Reliable Capacity

The following is a summary of the estimated reliable capacity of the well field based on the rated pump capacities of the wells and the estimated hydrogeological capacities.

Existing Firm Pump Capacity. The firm pump capacity for VWs is determined based on the largest capacity well in the well field being out of service (see Table 4-12). In actuality, as described in Section 4.2, loss of the electrical service at the VW field will cause all VWs to be offline. However, for the purposes of estimating firm capacity it is assumed a generator may be brought onsite to supply temporary power to the well field. The recommendation to install a redundant power feed loop for the VW field is included in the recommended actions presented in Section 4.7.

---

Table 4-12. Existing Vertical Well Field Pump Capacity

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Rated Capacity (gpm)</th>
<th>Firm Capacity (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>VW 3R</td>
<td>735</td>
<td>735</td>
</tr>
<tr>
<td>VW 4R</td>
<td>555</td>
<td>555</td>
</tr>
<tr>
<td>VW 5</td>
<td>700</td>
<td>700</td>
</tr>
<tr>
<td>VW 6</td>
<td>900</td>
<td>-</td>
</tr>
<tr>
<td>VW 7</td>
<td>730</td>
<td>730</td>
</tr>
<tr>
<td>VW 9</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>VW 10</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>VW 11</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>5,120</strong></td>
<td><strong>4,220</strong></td>
</tr>
</tbody>
</table>

1 Vertical Well 6 is located within 75 feet of HCW 2 and is expected to have reduced capacity when run concurrently with HCW 2.

Typically, in a HCW field, firm pump capacity is determined assuming the largest HCW is out of service. Another way to calculate it would be to assume the largest pump from each well is out of service.

In addition to pump capacity, electrical reliability for each of the HCWs was also considered. HCW 1 and 2 each have standby generators as detailed in Section 4.4.2. Due to the lack of standby power at both HCW 3 and HCW 4 firm pump capacity is calculated with one of these HCWs offline. As shown in Table 4-13, the firm capacity of the remaining HCWs in the collector well field would be 38 MGD, assuming HCW 3 is offline. If you were to assume the largest pump out of service in each HCW the resulting capacity would be 31.0 MGD, however, this may be overly conservative for collector well field. For the purposes of estimating firm capacity we recommend using 38.0 MGD.
### Table 4-13. Existing Horizontal Collector Well Field Pump Capacity

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Pump ID</th>
<th>Rated Capacity (MGD)</th>
<th>Total Capacity (MGD [gpm])</th>
<th>Firm Capacity ¹ (MGD [gpm])</th>
<th>Alternate Firm Capacity² (MGD [gpm])</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 1</td>
<td>P-1</td>
<td>5.0</td>
<td>15.0 (10,417)</td>
<td>15.0 (10,417)</td>
<td>10.0 (6,944)</td>
</tr>
<tr>
<td></td>
<td>P-2</td>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW 2</td>
<td>P-1</td>
<td>3.5</td>
<td>12.0 (8,333)</td>
<td>12.0 (8,333)</td>
<td>7.0 (4,861)</td>
</tr>
<tr>
<td></td>
<td>P-2</td>
<td>3.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW 3</td>
<td>P-1</td>
<td>3.5</td>
<td>11.0 (7,639)</td>
<td>-</td>
<td>7.0 (4,861)</td>
</tr>
<tr>
<td></td>
<td>P-2</td>
<td>3.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td>4.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW 4</td>
<td>P-1</td>
<td>3.5</td>
<td>11.0 (7,639)</td>
<td>11.0 (7,639)</td>
<td>7.0 (4,861)</td>
</tr>
<tr>
<td></td>
<td>P-2</td>
<td>3.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P-3</td>
<td>4.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total (MGD)</td>
<td></td>
<td></td>
<td>49.0</td>
<td>38.0</td>
<td>31.0</td>
</tr>
</tbody>
</table>

¹ HCW firm capacity assumes HCW 3 is offline.
² Alternate firm capacity assumes largest pump out of service in each HCW.

**Hydrogeological Capacity.** The hydrogeological capacities for the VWs and HCWs were estimated based on the available drawdown and the most current recorded specific capacities. As a representation of average operating conditions the estimated capacities are presented as the total capacity of the well field. Additionally, the estimated available capacities assume low-flow conditions of 750 cfs in the Kansas River.

In future years, when river stage is up, the static water level will increase allowing more water to be pumped from the wells. However, since water supply is most commonly an issue during drought conditions, the low flow of the Kansas River is assumed for the purposes of estimating available capacity. The estimated available capacities for VWs and HCWs are below in Table 4-14 and Table 4-15, respectively.

The available hydrogeological capacity of a vertical well presented in Table 4-14 is determined based on the amount of drawdown available between the static (non-pumping) water level and the top of the screen of the well. Several of the existing VWs are limited in capacity because the static water level has decreased over time and a greater thickness of the aquifer is screened compared to current design standards.

For example, at VW 10, the well site originally had 35 feet of saturated thickness with the well screen installed across the bottom 25 feet. At the time, the ten feet of available drawdown above the screen was sufficient for the well to produce at the 770 gpm allowed by DWR. Currently, at low river flow conditions, there is only 0.20 ft of drawdown available from the static water level to the top of the screen, limiting the capacity of the
well to approximately 10 gpm. Similarly, the static water level at VWs 7, 9, and 11 are within the screen; therefore, the reliable capacities are effectively 0.0 gpm.

Table 4-14. Vertical Wells Hydrogeological Capacity

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Static Water Level</th>
<th>Top of Screen Elevation</th>
<th>Available Drawdown</th>
<th>2014 Specific Capacity</th>
<th>Available Capacity</th>
<th>Available Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(gpm/ft)</td>
<td>(gpm)</td>
<td>(MGD)</td>
</tr>
<tr>
<td>VW 3R</td>
<td>744.0</td>
<td>737.5</td>
<td>6.5</td>
<td>94.5</td>
<td>614</td>
<td>0.88</td>
</tr>
<tr>
<td>VW 4R</td>
<td>744.2</td>
<td>736.0</td>
<td>8.2</td>
<td>82.1</td>
<td>555</td>
<td>0.80</td>
</tr>
<tr>
<td>VW 5</td>
<td>740.4</td>
<td>739.0</td>
<td>1.4</td>
<td>63.1</td>
<td>88</td>
<td>0.13</td>
</tr>
<tr>
<td>VW 6</td>
<td>740.8</td>
<td>739.0</td>
<td>1.8</td>
<td>68.1</td>
<td>123</td>
<td>0.18</td>
</tr>
<tr>
<td>VW 7</td>
<td>746.1</td>
<td>750.1</td>
<td>-4.0</td>
<td>109.3</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>VW 9</td>
<td>747.0</td>
<td>748.5</td>
<td>-1.5</td>
<td>83.6</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>VW 10</td>
<td>743.0</td>
<td>742.8</td>
<td>0.2</td>
<td>54.5</td>
<td>11</td>
<td>0.01</td>
</tr>
<tr>
<td>VW 11</td>
<td>740.8</td>
<td>742.8</td>
<td>-2.0</td>
<td>59.5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1,391</td>
<td>2.00</td>
</tr>
</tbody>
</table>

1 Based on a Kansas River stage of 757.55 ft at De Soto gage (river flow of 750 cfs) less hydraulic losses to the well.

2 The estimated available capacity based on available drawdown and specific capacity for VW 4R is 673 gpm. However, the capacity is limited by the rated capacity of the pump of 555 gpm (see Table 4-12).

When considering production from VWs, the withdrawal rates are typically only affected by the aquifer level. It is important to recognize, however, when dealing with HCWs that aquifer levels, river levels, and water temperature all have a significant impact on the ability to withdraw water. Table 4-15 below shows the variability of withdrawal rates based the parameters identified above.

Table 4-15. Horizontal Collector Wells Hydrogeological Capacity

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(gpm/ft)</td>
<td>(MGD [gpm])</td>
<td>(MGD [gpm])</td>
<td>(MGD [gpm])</td>
<td>(MGD [gpm])</td>
<td>(MGD [gpm])</td>
</tr>
<tr>
<td>HCW 1</td>
<td>755.3</td>
<td>727.6</td>
<td>27.7</td>
<td>250</td>
<td>5.8 [4,000]</td>
<td>10.9 [7,600]</td>
<td>9.95 [6,912]</td>
<td>15.0 [10,417]</td>
<td>15.1 [10,500]</td>
</tr>
<tr>
<td>HCW 2</td>
<td>748.1</td>
<td>727.4</td>
<td>20.6</td>
<td>140</td>
<td>2.0 [1,400]</td>
<td>5.2 [3,600]</td>
<td>4.16 [2,888]</td>
<td>12.0 [8,333]</td>
<td>10.1 [7,000]</td>
</tr>
<tr>
<td>HCW 3</td>
<td>754.6</td>
<td>733.4</td>
<td>21.1</td>
<td>190</td>
<td>3.9 [2,700]</td>
<td>6.5 [4,500]</td>
<td>5.78 [4,015]</td>
<td>0.0 [0.0]</td>
<td>10.1 [7,000]</td>
</tr>
</tbody>
</table>
As shown in Table 4-14 and 4-15, the estimated total reliable hydrogeological capacity of both well fields is 18,634 gpm (26.8 MGD).

Figure 4-7 shows the estimated total capacity of the well fields compared to the projected maximum day demands presented in Section 3 of this Water Master Plan Update. As of 2015, with a projected demand of 29.5 MGD, there is a supply deficit of approximately 2.7 MGD.

As discussed above, the projected deficit assumes a conservative drought scenario of low river flows and minimal precipitation combined with the 2014 and 2015 observed specific capacities in the VWs and HCWs, respectively. During average conditions (non-drought), the wells will likely be capable of higher capacities.

![Figure 4-7. Existing Raw Water Supply Capacity](image)

Although this is projected as a drought scenario, the projected supply deficit will continue to grow over time as the system demands grow.

The capacity of the VWs, however, may be increased by pumping into the screens. While this is generally not recommended due to fouling and increased maintenance costs, this is a method in which the supply deficit may be decreased for short periods to meet peak demands. This should only be done if extensive drought conditions dictate a need.

The long term approach to addressing an increasing supply deficit includes continued maintenance of the well fields and expansion of water supply infrastructure. The
following sections will present the alternatives considered as part of this Master Plan for increasing the raw water supply capacity.

4.6 Raw Water Supply Improvement Alternatives

The purpose of this section is to identify and discuss potential improvement alternatives for increasing the City’s raw water supply. The alternatives presented in this section include:

- Maintenance on existing wells;
- Replacement of existing wells;
- Construction of additional wells; and
- Induced infiltration improvements to the River/Aquifer Interface.

It should be noted, the City’s current primary source of water supply is groundwater under the influence of surface water (produced from HCWs) and groundwater (produced from VWs); for this reason direct surface water intakes and other alternative sources of supply have not been considered as alternatives for water supply expansion as part of this Water Master Plan Update.

4.6.1 Maintenance on Existing Wells

All City wells (VWs and HCWs) should be inspected and cleaned periodically. The following subsection presents the recommended frequency of inspection and cleaning of the City’s wells and the estimated gain in capacity from these maintenance activities.

**Vertical Wells.** A typical frequency for major maintenance activities of municipal VWs constructed in alluvial environments is between two and five years. The frequency of well maintenance and cleaning should be based on current monitoring data. If the specific capacity of a well declines by more than 30-percent then maintenance should be performed. Regardless of the specific capacity, an alluvial well should be inspected every five (5) years at a minimum.

As presented in Table 4-8, four of the eight active VWs have declined more than 30-percent in specific capacity (VWs 5, 6, 9, and 11) indicating cleaning and maintenance is required. Additionally, three of the eight active wells (VWs 7, 9 and 11) have pumping water levels below the top of the screen. As discussed previously, the recommended practice of vertical wells pumping is that the water level should be maintained above the top of the screen. Pulling the water level into the screen, introduces air to the screen and can accelerate biological fouling of the screen and chemical deposition of iron and manganese in the well and pump. These issues will reduce a well’s capacity over time. It is essential that VWs 7, 9, and 11 be continually inspected and frequently cleaned while being operated at water levels below the top of screens.

It is recommended that all five of the eight active wells (VWs 5, 6, 7, 9, and 11) be prioritized for inspection, maintenance, and cleaning. Table 4-16 presents the estimated potential capacities likely achievable post-maintenance based on historical specific capacities achieved following cleaning.
It should be noted the potential capacities for VWs 7, 9, and 11 are shown as 0 gpm in Table 4-16. This is due to pumping water levels below the top of the screens. Although some production may be realized from operating the wells in this manner it is not reliable and, thus, not recommended as a long term practice. However, the wells may be utilized during periods of drought or high demand.

It is recommended that VWs 9 and 10 be further investigated to determine the cause of the production capacity issues that have been observed.

### Table 4-16. Vertical Wells Potential Capacity

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Static Water Level (ft)</th>
<th>Top of Screen Elevation (ft)</th>
<th>Available Drawdown (ft)</th>
<th>Potential Specific Capacity (gpm/ft)</th>
<th>Available Capacity (gpm)</th>
<th>Potential Capacity (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>VW 3R</td>
<td>744.0</td>
<td>737.5</td>
<td>6.5</td>
<td>110</td>
<td>614</td>
<td>715</td>
</tr>
<tr>
<td>VW 4R</td>
<td>744.2</td>
<td>736.0</td>
<td>8.2</td>
<td>110</td>
<td>555</td>
<td>775</td>
</tr>
<tr>
<td>VW 5</td>
<td>740.4</td>
<td>739.0</td>
<td>1.4</td>
<td>100</td>
<td>88</td>
<td>140</td>
</tr>
<tr>
<td>VW 6</td>
<td>740.8</td>
<td>739.0</td>
<td>1.8</td>
<td>120</td>
<td>123</td>
<td>216</td>
</tr>
<tr>
<td>VW 7</td>
<td>746.1</td>
<td>750.1</td>
<td>-4.0</td>
<td>125</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>VW 9</td>
<td>747.0</td>
<td>748.5</td>
<td>-1.5</td>
<td>100</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>VW 10</td>
<td>743.0</td>
<td>742.8</td>
<td>0.2</td>
<td>70</td>
<td>11</td>
<td>14</td>
</tr>
<tr>
<td>VW 11</td>
<td>740.8</td>
<td>742.8</td>
<td>-2.0</td>
<td>135</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>1,391</strong></td>
<td><strong>1,860</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. Based on a Kansas River stage of 757.55 ft at De Soto gage (river flow of 750 cfs) less hydraulic losses to the well.
2. Estimated potential specific capacities achievable by cleaning (corrected to 60 degrees Fahrenheit).
3. The estimated available capacity based on available drawdown and specific capacity for VW 4R is 673 gpm. However, the capacity is limited by the rated capacity of the pump of 555 gpm (see Table 4-12).
4. The estimated potential specific capacity for VW 4R is 902 gpm. However, the capacity is limited by the apportioned maximum instantaneous rate of the pump of 775 gpm (see Table 4-1) assuming the existing pump is upgraded.

The estimated combined capacity that may be gained by cleaning all VWs is nearly 0.7 MGD (469 gpm). In order for VW 4R to achieve the potential capacity depicted in Table 4-16 the existing pump must be upgraded with a higher rated capacity.

It should be noted that potential capacity may be negatively impacted by the electrical and mechanical reliability of the VW pumps. Therefore, it is important to install a redundant power feed loop to the VW field as recommended previously. The potential of pump failure has not been considered as part of the estimated potential capacity presented in Table 4-16.

Table 4-17 presents the proposed cleaning schedule for the City’s VWs prioritized based on observed specific capacity decline, potential capacity to be gained, and whether the well is currently pumped into the screen. It is recommended the City perform maintenance on four VWs per year. It is estimated to cost approximately $25,000 per well for inspection, maintenance and cleaning. It is recommended the CIP budget should contain $100,000 per year for two years to clean all the VWs. The City currently
includes funds for inspections of VWs annually and repair/cleaning as needed in the Operating Budget.

Once the VW field has been cleaned, then the specific capacity decline and other prioritization criteria should be confirmed prior to initiating future maintenance. The schedule in Table 4-17 assumes the existing VWs decline at 10-percent per year requiring cleaning approximately every three years.

### Table 4-17. Vertical Well Cleaning Schedule

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Last Cleaned</th>
<th>2017</th>
<th>2018</th>
<th>2019</th>
<th>2020</th>
<th>2021</th>
<th>2022</th>
</tr>
</thead>
<tbody>
<tr>
<td>VW 3</td>
<td>2009</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>VW 4</td>
<td>2012</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>VW 5</td>
<td>2006</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>VW 6</td>
<td>2013</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>VW 7</td>
<td>2013</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>VW 9</td>
<td>2013</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>VW 10</td>
<td>2013</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>VW 11</td>
<td>2012</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

The estimated capacities gained from a VW Cleaning Program assume cleaning is performed at the recommended locations and at a frequency as presented above in Table 4-17. The capacities assume that the VWs decline at a rate of 10-percent per year following cleaning and that the well cleanings are initiated in 2017/2018 and 2021/2022. Following the 2021/2022 cleaning cycles the VWs are assumed to continually decline at 10-percent per year with no cleanings based on the potential future installation of replacement VWs. This is presented in further detail in the following subsection.

**Horizontal Collector Wells.** Typically, HCWs should be cleaned every 7 to 10 years. The City’s HCWs may require cleaning on a more frequent basis (every 5 years) given capacity decline observed in historic production data. Monitoring at the piezometers located near the collector well laterals indicate that most capacity lost due to clogged laterals was regained during the 2011 to 2013 cleaning.

Lost capacity due to clogged laterals can typically be regained through regular cleaning of the laterals. Eventually, the existing laterals will stop responding to cleaning, much like vertical well screens do with age.

As presented in Table 4-10, two of the four active HCWs have declined more than 20-percent in specific capacity (HCWs 2 and 4) indicating cleaning and maintenance is required. Furthermore, the remaining two wells (HCWs 1 and 3) indicate a decline of 19-percent in specific capacity. Table 4-18 presents an estimate of the potential capacity that may be gained in the HCWs with cleaning.
Table 4-18. Horizontal Collector Wells Potential Capacity

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Static Water Level 1 (ft)</th>
<th>Minimum Pumping Elevation 2 (ft)</th>
<th>Available Drawdown (ft)</th>
<th>Potential Specific Capacity 3 (gpm/ft)</th>
<th>Available Capacity (gpm)</th>
<th>Potential Capacity (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 1</td>
<td>755.3</td>
<td>727.6</td>
<td>27.7</td>
<td>310</td>
<td>6,912</td>
<td>8,571</td>
</tr>
<tr>
<td>HCW 2</td>
<td>748.1</td>
<td>727.4</td>
<td>20.6</td>
<td>175</td>
<td>2,888</td>
<td>3,610</td>
</tr>
<tr>
<td>HCW 3</td>
<td>754.6</td>
<td>733.4</td>
<td>21.1</td>
<td>235</td>
<td>4,015</td>
<td>4,966</td>
</tr>
<tr>
<td>HCW 4</td>
<td>755.6</td>
<td>731.1</td>
<td>24.5</td>
<td>220</td>
<td>3,427</td>
<td>5,386</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>17,243</strong></td>
<td><strong>22,533</strong></td>
<td></td>
</tr>
</tbody>
</table>

1 Based on a Kansas River stage of 757.55 ft at De Soto gage (river flow of 750 cfs) less hydraulic losses to the well.
2 Established based on 5 feet above the centerline of the lateral elevation.
3 Estimated potential specific capacities achievable by cleaning of laterals (corrected to 60 degrees Fahrenheit).

The estimated combined capacity that may be gained by cleaning all HCWs is nearly 7.6 MGD (5,290 gpm).

It should be noted that potential capacity may be negatively impacted by the electrical and mechanical reliability of the HCW pumps. Therefore, it is important that the lightening protection and backup power projects be included in the CIP as previously presented in Section 4.4.2. The potential of pump failure has not been considered as part of the estimated potential capacity presented in Table 4-18.

Table 4-19 presents the proposed cleaning schedule for the City’s HCWs prioritized based on observed specific capacity decline and potential capacity to be gained. It is recommended the City perform maintenance on one HCW per year over four years. It is estimated to cost approximately $220,000 per HCW for inspection and cleaning. However, if cleanings are packaged with multiple wells some savings for mobilization and demobilization may be realized.

The HCW specific capacity decline and other prioritization criteria should be confirmed prior to initiating maintenance again in 2023. The City has indicated that cleaning one HCW per year between 2018 and 2020 has been included in the Operating Budget.

Table 4-19. Horizontal Collector Well Cleaning Schedule

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Last Cleaned</th>
<th>2017</th>
<th>2018</th>
<th>2019</th>
<th>2020</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 1</td>
<td>Oct 2011</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW 2</td>
<td>Nov 2012</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>HCW 3</td>
<td>Jan 2013</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>HCW 4</td>
<td>Nov 2011</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Each HCW was originally constructed with 1 or 2 spare lateral ports (see Table 4-9). These spare ports may be used to install additional laterals and increase the capacity of the existing well. Additional laterals allow more water to be withdrawn for the same amount of drawdown. New laterals should be reserved for when existing laterals no
longer respond to cleaning, as there are limited positions for new laterals and once a new lateral is constructed it will begin aging.

Spare ports in HCW 1 were constructed 2.5 feet above the centerline of the existing laterals. This will impact the additional capacity that may be realized by utilizing the additional laterals.

The estimated capacities gained from a HCW Cleaning Program assume cleaning is performed at the recommended locations and at a frequency as presented above in Table 4-19. The capacities assume that the HCWs decline at a rate of 4-percent per year following cleaning and that the well cleaning cycles are initiated in 2017, 2023, and 2029. The estimated capacities in 2029 assume the HCWs only recover to 96% of that capacity achieved by cleaning in 2017 and 2023 due to age of the infrastructure.

**Estimated Combined Capacity.** The estimated capacities gained through maintenance of the HCWs have been added to those capacities presented in Figure 4-7. The estimated combined capacity gained through maintenance of the existing VWs and HCWs is presented in Figure 4-8. The Figure 4-8 shows the projected variability in capacity available for production of the time period. All future figures will show and average capacity gained by cleaning the HCWs after the initial two years of cleaning has been implemented.
Figure 4-8. Raw Water Supply – Maintenance on Existing Wells

- **VW Cleaning Program**: 4 VWs per year for 2 years (frequency every 3 years). Assumes VWs decline at a rate of 10-percent per year.
- **HCW Cleaning Program**: 1 HCW per year for 4 years (frequency every 5 years). Assumes HCWs decline at a rate of 4-percent per year.
4.6.2 Vertical Well Replacement

VWs are not as dependent on river flow and temperature for maximum production capability as HCWs and can be an important source during drought or water quality impacts to the Kansas River. This subsection considers the potential capacity that may be realized by constructing replacement VWs within the existing well field and increasing the number of diversion points.

The City’s vertical well field is the only raw water source that is covered by a single groundwater right, and does not need a matching surface water right in order to be produced. Although the wells are within ¼ mile of the river and the water right is backed up by KRWAD #1 in-stream flow, the water right on the VWs has a priority date older than the right associated with KRWAD #1. As such, the ability to maximize production under this water right is crucial during extreme drought or during limited river flow conditions.

Furthermore, maximizing use of the existing VWs assists with the required perfection of the combined water right limits on HCWs 2, 3, and 4. These HCWs have both an annual water production limitation on the individual HCW and a combined limit in combination with previous rights. In order to perfect the combined limit to its maximum potential, the wells with previous rights need to be in good operating condition.

The replacement wells should be constructed according to current vertical well construction materials, which allow for shorter screen intervals. It is recommended that new replacement wells be designed with seven to twelve foot long, 36-inch diameter well screens to meet inflow velocity requirements for DWR maximum pumping rates and to have sufficient available drawdown above the top of the well screen. Shorter screens will result in additional available drawdown above the top of the screen, resulting in additional capacity. It is not recommended to screen less than 33-percent of the saturated thickness of the aquifer.

This alternative involves discontinuing VW 6, installing new diversion points for VW 1 and 2, relocating VW 9 and offsetting VWs 3R, 4R, 5, 7, 10 and 11. The proposed changes to the existing vertical well field are shown in Figure 4-9.

The estimated potential capacity to be gained at each of the replacement VWs is shown in Table 4-20. DWR has indicated that the City may increase the number of diversion points but may not exceed the combined instantaneous rate of 5,215 gpm (7.5 MGD). Therefore, Table 4-20 presents the distribution of the combined instantaneous rate among replacement wells.

VW 6 will be reclassified as a standby well due to its proximity to HCW 2. Permitting with DWR will be required to relocate the point of diversion of VW 6 to the vicinity of abandoned well VW1. A new VW 1 can be constructed in this area of the aquifer known to have good production capacity. This new well will be more than 300 feet from the existing VW 6 and will require approval from DWR.

A new VW 2, located at the historical location of VW 2 in the southern portion of the well field, will add a 9th point of diversion to the well field, where there are currently only 8 points of diversion. As a result, the allowable instantaneous rate will be revised through
permitting with DWR for each point of diversion such that the total allowable diversion rate of all wells combined is 6,990 gpm. The combination of wells will still be subject to the limitation of the maximum instantaneous pumping rate of 5,215 gpm.

Test hole drilling will be required as part of this project to identify areas of high production potential within the existing well field. Based on the test hole drilling, VW 9 may be relocated to the southern portion of the well field. It is expected VWs 3R, 4R, 5, 7, 10 and 11 will be offset by approximately 50-feet, adjacent to existing well locations. VWs 7 and 9 are located in the floodway. Offsetting these wells is anticipated to have additional permitting requirements associated with construction in the floodway. It should be noted KDHE does require the owner of a public water supply well have either ownership or a perpetual easement for the area within a 100-foot radius of the well.

Additional operational variability should be considered with the replacement of the VWs. The VW water quality is generally of higher hardness than the HCWs due to its supply being a less significant proportion of the River. The new VW pumps should be affixed with VFDs to provide flexibility in operations. Lastly, a higher amount of lime is required as part of the water treatment process to achieve the desired water hardness goal.

### Table 4-20. Vertical Well Replacement Capacities

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Static Water Level 1 (ft)</th>
<th>Existing Well</th>
<th>Replacement Well</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Specific Capacity (gpm/ft)</td>
<td>Top of Screen Elev (ft msl)</td>
<td>Available Capacity (gpm)</td>
</tr>
<tr>
<td>VW 1</td>
<td>743.00</td>
<td>Abandoned</td>
<td>63</td>
</tr>
<tr>
<td>VW 2</td>
<td>743.00</td>
<td>Abandoned</td>
<td>63</td>
</tr>
<tr>
<td>VW 3R</td>
<td>744.00</td>
<td>94.5</td>
<td>737.50</td>
</tr>
<tr>
<td>VW 4R</td>
<td>744.20</td>
<td>82.1</td>
<td>736.00</td>
</tr>
<tr>
<td>VW 5</td>
<td>740.40</td>
<td>63.1</td>
<td>739.00</td>
</tr>
<tr>
<td>VW 6 a</td>
<td>740.80</td>
<td>68.1</td>
<td>739.00</td>
</tr>
<tr>
<td>VW 7</td>
<td>746.10</td>
<td>109.3</td>
<td>750.10</td>
</tr>
<tr>
<td>VW 8</td>
<td>747.00</td>
<td>Abandoned</td>
<td></td>
</tr>
<tr>
<td>VW 9</td>
<td>747.00</td>
<td>83.6</td>
<td>748.50</td>
</tr>
<tr>
<td>VW 10</td>
<td>743.00</td>
<td>54.5</td>
<td>742.80</td>
</tr>
<tr>
<td>VW 11</td>
<td>740.80</td>
<td>59.5</td>
<td>742.80</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>1,509</td>
<td>6,990</td>
<td>9,241</td>
</tr>
<tr>
<td>Firm Capacity</td>
<td>777</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 Based on a Kansas River stage of 757.55 ft at De Soto gage (river flow of 750 cfs) less hydraulic losses to the well.
2 Assuming long-term firm yield at 70-percent well efficiency (based on typical allowed decline of 30-percent between cleanings).
3 Assuming VW 6 is converted to a Standby well. VW 6 may not achieve 966 gpm while HCW 2 is pumping.
Designate Well #6 as a backup well

Re-establish Wells #1 and #2 as points of diversion.

Map Feature Key
- Olathe City Limits
- City Owned Property
- Raw Water Mains
- Existing Olathe Vertical Well

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Figure 4-9. Proposed Improvements to Existing Vertical Well Field
The cost estimate of replacement of the VWs is presented in Table 4-21. For the purposes of this Master Plan, the replacement of existing VWs assumes that all 9 VWs are replaced as part of one Project. However, there is flexibility to phase the construction of VWs based on supply need and cost.

Table 4-21. Vertical Well Replacement Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Qty</th>
<th>Unit</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Well, with Pumps and Access Platform</td>
<td>9</td>
<td>EA</td>
<td>$300,000</td>
<td>$2,700,000</td>
</tr>
<tr>
<td>VFDs, 100 HP, Outdoor Rated</td>
<td>9</td>
<td>EA</td>
<td>$60,000</td>
<td>$540,000</td>
</tr>
<tr>
<td>12&quot; DIP ¹</td>
<td>2,700</td>
<td>LF</td>
<td>$220</td>
<td>$594,000</td>
</tr>
<tr>
<td>8&quot; Flow Meters</td>
<td>9</td>
<td>EA</td>
<td>$8,000</td>
<td>$72,000</td>
</tr>
<tr>
<td>Flow Meter Vaults with Valves</td>
<td>9</td>
<td>EA</td>
<td>$50,000</td>
<td>$450,000</td>
</tr>
<tr>
<td>Piping Tie-Ins</td>
<td>9</td>
<td>EA</td>
<td>$5,000</td>
<td>$45,000</td>
</tr>
<tr>
<td>Stand-by Generator</td>
<td>1</td>
<td>LS</td>
<td>$350,000</td>
<td>$350,000</td>
</tr>
<tr>
<td>Site Work (5%)</td>
<td>1</td>
<td>LS</td>
<td>$217,000</td>
<td>$221,000</td>
</tr>
<tr>
<td>Electrical and Instrumentation (15%)</td>
<td>1</td>
<td>LS</td>
<td>$736,000</td>
<td>$746,000</td>
</tr>
<tr>
<td>General Requirements (5%)</td>
<td>1</td>
<td>LS</td>
<td>$282,000</td>
<td>$286,000</td>
</tr>
</tbody>
</table>

|                     |    |      | Subtotal  | $6,004,000 |
|                     |    | Contingency (30%) | $1,802,000 |
| **Total Construction Cost**                             | $7,806,000 |
| Engineering, Legal, Admin (20%)                        | $1,562,000 |
| **Total Project Cost (2016 Dollars)**                  | $9,368,000 |

¹ Assumes replacement well constructed within 300 feet of existing well.

The implementation schedule for replacement of the existing VWs is presented in Table 4-22. The CIP prioritization including new vertical wells is shown in Figure 4-10. The schedule assumes DWR water rights approval, design of the replacement wells, and KDHE approval of design and specifications will require approximately 12 months. Bidding the Project will require 3 months. Construction of all 9 replacement wells will require approximately 12 months. The estimated Project duration will be 27 months. It is anticipated the replacement vertical wells will be required by the end of 2020; therefore, construction will need to begin in 2018. This is presented in further detail in Section 4.7.

Table 4-22. Replacement of Vertical Wells Implementation Schedule

<table>
<thead>
<tr>
<th>Replace Existing Vertical Wells</th>
<th>2018</th>
<th>2019</th>
<th>2020</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st Qtr</td>
<td>2nd Qtr</td>
<td>3rd Qtr</td>
</tr>
<tr>
<td>Vertical Wells</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design and Permitting</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bidding</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Construction</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4.6.3 Construction of Additional Horizontal Collector Wells

Construction of additional HCWs is another option for additional water supply development. Several studies have been completed previously for developing additional HCWs. An overview of these previous studies is provided in Appendix E.

The City’s current plan for meeting future demands is to construct additional HCWs 5, 6, and 7 (see Figure 4-11). The City has purchased property north of the Kansas River and has applied for water rights for HCW 5. The water rights have been approved pending construction of the well.

Table 4-23 presents the estimated yield for each of the three sites given Olathe’s design and operational criteria, and using the Hantush and Papadopulos equation with the aquifer information proposed by MKEC parameters (see Appendix E).
Table 4-23. Estimated Future Horizontal Collector Well Yields

<table>
<thead>
<tr>
<th>Proposed Well ID</th>
<th>Potential Yield ¹ (MGD)</th>
<th>Estimated Percent Decline from Seasoning</th>
<th>Potential Yield ² (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 5</td>
<td>12.6</td>
<td>45%</td>
<td>6.9</td>
</tr>
<tr>
<td>HCW 6</td>
<td>6.8</td>
<td>35%</td>
<td>4.4</td>
</tr>
<tr>
<td>HCW 7</td>
<td>5.6</td>
<td>35%</td>
<td>3.6</td>
</tr>
</tbody>
</table>

¹ Yields shown are based on a summer low flow river condition, laterals at 7.5 feet above bedrock, minimum water level at 5 feet above laterals, assuming 10% decline in yield based on screen clogging.

² Yields adjusted for decline due to seasoning. HCW 5 based on similar seasoning in HCW 1. HCW 6 and 7 based on seasoning that occurred in HCW 3 and 4.

Construction of HCWs on the north side of the Kansas River will necessitate a transmission pipeline crossing the Kansas River. In 2009, a 48-inch raw water transmission main was constructed from WTP2 to the vicinity of HCW 1. Some preliminary engineering investigation was completed by Black & Veatch in 2009 as part of that Project. The Draft Memorandum⁴ recommends a river crossing installed via horizontal directional drilling (HDD) in bedrock with an entry angle of 10 degrees, exit angle of 8 degrees, and radius of curvature of 2,000 feet. The purpose for recommendation of installation in bedrock is to avoid the risk of inadvertent returns of the drilling fluid into the river or failure of the crossing in the coarse-grained strata that exists above bedrock. Figure 4-12 depicts a schematic of the soil strata under the Kansas River.

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⁴ 48-inch Raw Water Transmission Main, Phase 1 – Alignment Study Services, City Project No. 5-C-009-09, Horizontal and Vertical Alignment Memorandum, November 24, 2009 and Construction Methods Memorandum, November 24, 2009.
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Figure 4-11. Proposed Additional HCWs

*Note: Locations of existing wells and test sites are approximate.*
Based on the information available, HDD in the coarse-grained layer may experience failure and inadvertent returns of the drilling fluid. The very materials that make for a productive well field (gravel, coarse sand) also make HDD difficult. HDD in bedrock, as proposed in the Draft Memorandum, can experience failures on the entry and exit sides through those materials. Therefore, tunneling through bedrock may be the preferred solution. This can be a more expensive option. WaterOne has installed a 60-inch bedrock tunnel crossing of the Kansas River in 2008 approximately 10 miles downstream of the City's proposed crossing. The reported cost of the project was $6.7 million; approximately $110 per inch diameter per linear foot. It is recommended that a detailed geotechnical and geophysical evaluation of the soil strata be performed including a review by large-diameter HDD and tunneling contractors.

Alternative cost estimates were developed for the transmission pipeline crossing the Kansas River. Each alternative cost requires a carrier pipeline, tunnel shafts, and a river crossing tunnel. The alternatives considered include:

- Alternative 1: Single 24-inch pipeline to serve HCW 5 only;
- Alternative 2: Single 36-inch carrier pipeline to serve HCWs 5, 6, and 7; and
- Alternative 3: Two 24-inch pipelines to serve HCWs 5, 6, and 7.

The cost estimates for each alternative are presented in Table 4-24, 4-25, and 4-26.
### Table 4-24. Alternative 1: Single 24-inch Carrier Pipe Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Qty</th>
<th>Unit</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kansas River Bank Stabilization (Rip-Rap)</td>
<td>50</td>
<td>CY</td>
<td>$150</td>
<td>$8,000</td>
</tr>
<tr>
<td>24&quot; Carrier Pipe for HCW 5 only</td>
<td>5,500</td>
<td>LF</td>
<td>$350</td>
<td>$1,925,000</td>
</tr>
<tr>
<td>River Crossing Tunnel Shafts</td>
<td>140</td>
<td>VF</td>
<td>$35,000</td>
<td>$4,900,000</td>
</tr>
<tr>
<td>36&quot; River Crossing Tunnel (^1)</td>
<td>1,800</td>
<td>LF</td>
<td>$2,000</td>
<td>$3,600,000</td>
</tr>
<tr>
<td>General Requirements (7%)</td>
<td>1</td>
<td>LS</td>
<td>$731,000</td>
<td>$731,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>$11,164,000</strong></td>
</tr>
<tr>
<td>Contingency (30%)</td>
<td></td>
<td></td>
<td></td>
<td><strong>$3,350,000</strong></td>
</tr>
<tr>
<td><strong>Total Construction Cost</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>$14,514,000</strong></td>
</tr>
<tr>
<td>Engineering, Legal, Admin (20%)</td>
<td></td>
<td></td>
<td></td>
<td><strong>$2,903,000</strong></td>
</tr>
<tr>
<td><strong>Total Project Cost</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>$17,417,000</strong></td>
</tr>
</tbody>
</table>

\(^1\) Assumes tunneling through a bed rock is completed using Micro-Tunnel Boring Machine (MTBM).

### Table 4-25. Alternative 2: Single 36-inch Carrier Pipe Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Qty</th>
<th>Unit</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kansas River Bank Stabilization (Rip-Rap)</td>
<td>50</td>
<td>CY</td>
<td>$150</td>
<td>$8,000</td>
</tr>
<tr>
<td>36&quot; Raw Water Line for HCWs 5, 6, 7</td>
<td>5,500</td>
<td>LF</td>
<td>$450</td>
<td>$2,475,000</td>
</tr>
<tr>
<td>River Crossing Tunnel Shafts</td>
<td>140</td>
<td>VF</td>
<td>$35,000</td>
<td>$4,900,000</td>
</tr>
<tr>
<td>48&quot; River Crossing Tunnel (^1)</td>
<td>1,800</td>
<td>LF</td>
<td>$3,000</td>
<td>$5,400,000</td>
</tr>
<tr>
<td>General Requirements (7%)</td>
<td>1</td>
<td>LS</td>
<td>$958,000</td>
<td>$958,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>$13,678,000</strong></td>
</tr>
<tr>
<td>Contingency (30%)</td>
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<td></td>
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<td><strong>$4,104,000</strong></td>
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<tr>
<td><strong>Total Construction Cost</strong></td>
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<td></td>
<td></td>
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<tr>
<td>Engineering, Legal, Admin (20%)</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td><strong>$21,339,000</strong></td>
</tr>
</tbody>
</table>

\(^1\) Assumes tunneling through a bed rock is completed using Micro-Tunnel Boring Machine (MTBM).
Table 4-26. Alternative 3: Two, 24-inch Carrier Pipes Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Qty</th>
<th>Unit</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kansas River Bank Stabilization (Rip-Rap)</td>
<td>50</td>
<td>CY</td>
<td>$150</td>
<td>$8,000</td>
</tr>
<tr>
<td>24&quot; Carrier Pipe for HCW 5</td>
<td>5,500</td>
<td>LF</td>
<td>$350</td>
<td>$1,925,000</td>
</tr>
<tr>
<td>24&quot; Carrier Pipe for HCW 6 and 7</td>
<td>5,500</td>
<td>LF</td>
<td>$350</td>
<td>$1,925,000</td>
</tr>
<tr>
<td>River Crossing Tunnel Shafts</td>
<td>140</td>
<td>VF</td>
<td>$35,000</td>
<td>$4,900,000</td>
</tr>
<tr>
<td>60&quot; River Crossing Tunnel</td>
<td>1,800</td>
<td>LF</td>
<td>$3,500</td>
<td>$6,300,000</td>
</tr>
<tr>
<td>General Requirements (7%)</td>
<td>1</td>
<td>LS</td>
<td>$1,055,000</td>
<td>$1,055,000</td>
</tr>
</tbody>
</table>

Subtotal $16,113,000

Contingency (30%) $4,834,000

Total Construction Cost $20,947,000

Engineering, Legal, Admin (20%) $4,190,000

Total Project Cost $25,137,000

---

1 One 24-in carrier pipe will be capped until HCWs 6 and 7 are brought online.
2 Assumes tunneling through a bed rock is completed using Micro-Tunnel Boring Machine (MTBM).

Although Alternative 1 is of lower cost it will serve HCW 5 only. In order to serve HCWs 6 and 7, it will require construction of a separate 24-inch carrier pipe through a separate 36-inch tunnel. Therefore, to serve all HCWs the total cost is nearly $35 million. Alternative 1 may be a viable alternative as the construction of HCWs 6 and 7 may be delayed past 2050, as presented in further detail in Section 4.8 below, thereby offsetting the additional cost until further into the future.

Alternative 2, however, is recommended for the river crossing transmission line as it is the lowest cost alternative to construct a transmission line that will serve all three HCWs 5, 6, and 7. This recommendation and timing of construction should be reevaluated in any subsequent Master Plan Updates.

The cost estimates for construction of HCW 5, 6, and 7 are presented in Table 4-27, 4-28, and 4-29, respectively.
Table 4-27. HCW 5 Construction Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Qty</th>
<th>Unit</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 5 ¹,²</td>
<td>1</td>
<td>LS</td>
<td>$5,400,000</td>
<td>$5,400,000</td>
</tr>
<tr>
<td>Stand-by Generator</td>
<td>1</td>
<td>LS</td>
<td>$460,000</td>
<td>$460,000</td>
</tr>
<tr>
<td>General Requirements (7%)</td>
<td>1</td>
<td>LS</td>
<td>$411,000</td>
<td>$411,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td></td>
<td></td>
<td></td>
<td>$6,271,000</td>
</tr>
<tr>
<td>Contingency (30%)</td>
<td></td>
<td></td>
<td></td>
<td>$1,882,000</td>
</tr>
<tr>
<td><strong>Total Construction Cost</strong></td>
<td></td>
<td></td>
<td></td>
<td>$8,153,000</td>
</tr>
<tr>
<td>Engineering, Legal, Admin (20%)</td>
<td></td>
<td></td>
<td></td>
<td>$1,631,000</td>
</tr>
<tr>
<td><strong>Total Project Cost (2016 Dollars)</strong></td>
<td></td>
<td></td>
<td></td>
<td>$9,784,000</td>
</tr>
</tbody>
</table>

¹ Cost includes site work and electrical.
² Assumes river crossing transmission pipeline will connect to HCW 5.

Table 4-28. HCW 6 Construction Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Qty</th>
<th>Unit</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 6 ¹</td>
<td>1</td>
<td>LS</td>
<td>$3,600,000</td>
<td>$3,600,000</td>
</tr>
<tr>
<td>Kansas River Bank Stabilization (Rip-Rap)</td>
<td>50</td>
<td>CY</td>
<td>$150</td>
<td>$8,000</td>
</tr>
<tr>
<td>Stand-by Generator</td>
<td>1</td>
<td>LS</td>
<td>$402,500</td>
<td>$402,500</td>
</tr>
<tr>
<td>24” Raw Water Line</td>
<td>1,000</td>
<td>LF</td>
<td>$340</td>
<td>$340,000</td>
</tr>
<tr>
<td>General Requirements (7%)</td>
<td>1</td>
<td>LS</td>
<td>$305,000</td>
<td>$305,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td></td>
<td></td>
<td></td>
<td>$4,655,500</td>
</tr>
<tr>
<td>Contingency (30%)</td>
<td></td>
<td></td>
<td></td>
<td>$1,397,000</td>
</tr>
<tr>
<td><strong>Total Construction Cost</strong></td>
<td></td>
<td></td>
<td></td>
<td>$6,052,500</td>
</tr>
<tr>
<td>Engineering, Legal, Admin (20%)</td>
<td></td>
<td></td>
<td></td>
<td>$1,211,000</td>
</tr>
<tr>
<td><strong>Total Project Cost (2016 Dollars)</strong></td>
<td></td>
<td></td>
<td></td>
<td>$7,263,500</td>
</tr>
</tbody>
</table>

¹ Cost includes site work and electrical.

Table 4-29. HCW 7 Construction Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Qty</th>
<th>Unit</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 7 ¹</td>
<td>1</td>
<td>LS</td>
<td>$3,600,000</td>
<td>$3,600,000</td>
</tr>
<tr>
<td>Kansas River Bank Stabilization (Rip-Rap)</td>
<td>50</td>
<td>CY</td>
<td>$150</td>
<td>$8,000</td>
</tr>
<tr>
<td>Stand-by Generator</td>
<td>1</td>
<td>LS</td>
<td>$402,500</td>
<td>$402,500</td>
</tr>
<tr>
<td>24” Raw Water Line</td>
<td>3,000</td>
<td>LF</td>
<td>$340</td>
<td>$1,020,000</td>
</tr>
<tr>
<td>General Requirements (7%)</td>
<td>1</td>
<td>LS</td>
<td>$353,000</td>
<td>$353,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td></td>
<td></td>
<td></td>
<td>$5,383,500</td>
</tr>
<tr>
<td>Contingency (30%)</td>
<td></td>
<td></td>
<td></td>
<td>$1,616,000</td>
</tr>
<tr>
<td><strong>Total Construction Cost</strong></td>
<td></td>
<td></td>
<td></td>
<td>$6,999,500</td>
</tr>
<tr>
<td>Engineering, Legal, Admin (20%)</td>
<td></td>
<td></td>
<td></td>
<td>$1,400,000</td>
</tr>
<tr>
<td><strong>Total Project Cost (2016 Dollars)</strong></td>
<td></td>
<td></td>
<td></td>
<td>$8,399,500</td>
</tr>
</tbody>
</table>

¹ Cost includes site work and electrical.
The implementation schedule for the construction of HCW 5 is presented in Table 4-30. The CIP prioritization including new collector wells is shown in Figure 4-13. The schedule assumes design and permitting for the transmission line and well occur in parallel. Design and permitting will require 9 months and 12 months, respectively. Bidding each Project will require 3 months. Construction of the transmission line is anticipated to require approximately 18 months. Construction of the HCW is anticipated to require approximately 12 months. The total estimated Project duration will be 30 months.

HCW 5 and the river crossing transmission pipeline is currently planned to begin construction in 2028. The existing CIP also includes construction of HCWs 6 and 7 on the pending project list. This recommendation and timing of construction should be reevaluated in any subsequent Master Plan Updates.

This is presented in further detail in Section 4.7.

Table 4-30. Construction of HCW 5 Implementation Schedule

<table>
<thead>
<tr>
<th>Construct HCW5</th>
<th>2028</th>
<th>2029</th>
<th>2030</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st Qtr</td>
<td>2nd Qtr</td>
<td>3rd Qtr</td>
</tr>
<tr>
<td>Kansas River Raw Water Crossing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design and Permitting</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bidding</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Construction</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HCW5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design and Permitting</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bidding</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Construction</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 4-13. CIP Prioritization Including HCW No. 5
4.6.4 Induced Infiltration Improvements to the River/Aquifer Interface

Additional capacity may be gained for the HCWs through streambed modifications. It is expected that the impacts of streambed clogging (i.e., “seasoning”) can potentially be remediated by removing or breaking up the uppermost layer of the streambed adjacent to HCWs 1, 3 and 4. The removal will involve contracting a local in-stream sand and gravel dredger. This remedial action will require approval by the U.S. Army Corps of Engineers (USACE).

The USACE has recently developed a draft Environmental Impact Statement (EIS) addressing in-stream dredging in the Kansas River. Within this EIS the USACE proposes to continue permitting commercial dredging, which will likely have a much greater impact than simply removing the upper layer of sediment. Removal of the uppermost sand layer is not expected to result in permanent improvements, as the freshly uncovered streambed will continue the process of seasoning, requiring additional removal in another 2 to 4 years.

As discussed in Section 4.6.4, it is hypothesized horizontal clay or silt layers may exist east of HCW 2 impeding recharge from the riverbed to the collector well laterals. Very little geologic data is available east of HCW 2. Pumping tests at both VW 6 and HCW 2 indicate that the effective A-distance to recharge is much further than expected. In 2006, the City drilled three test holes east and southeast of HCW 2, with only one deeper than 15 feet. In the deeper test hole, a ten-foot layer of clayey sand was encountered at an elevation of approximately 726 to 736 feet msl. If this layer has significant lateral extent, it would impede recharge from the river to the HCW 2 laterals at 722.4 ft msl. Additional test hole drilling will be required to verify the extent of this clayey sand layer. Dredging within the river adjacent to HCW 2 may remove this clayey layer.

Conversations with Master’s Dredging personnel indicate that Master’s has only dredged adjacent to the north end of the vertical well field. Currently, Master’s Dredging has applied with the USACE to dredge 300,000 tons of sand and gravel annually from River Mile 26.1 to 27.1. It is not known where within the mile stretch operations would occur. The City has requested that the dredging is performed more than 500-ft from the HCW 2 laterals.

Several communities outside of the State of Kansas with riverbank infiltration well fields use recharge channels on the backside of wells to reduce drawdown in the aquifer away from the river. The recharge channels will then be filled via a surface water pump station. In the case of collector wells, additional laterals can be installed in the direction of the recharge channels to further increase well production rates. Additional water rights will be required for the river water pumped to the recharge channels. Recharge channels can clog much like with streambed seasoning. For recharge channels constructed with the base of the channel above static water level, the upper layer of aggregate in the channel can be removed and replaced with new aggregate.
4.7 Raw Water Transmission Assessment

A hydraulic model of the raw water transmission system was developed and calibrated using Innovyze InfoWater Executive Suite 12.2. The development process is described in detail in Appendix F. The model was used to assess the performance of the existing infrastructure for the current water supply and to assess the performance of the system for the projected future supply.

4.7.1 Existing Infrastructure and Operation

The raw water system includes HCWs, VWs, and transmission mains from the wells to the head of WTP2. As presented in Section 4.5, the estimated existing hydrogeological capacity based on available drawdown and the most current recorded specific capacities is approximately 26.8 MGD (see Table 4-14 and Table 4-15). These estimated hydrogeological capacities were used for the existing conditions model scenarios with the exception of VW 4R. VW 4R is limited by its rated pumping capacity of 555 gpm rather than the estimated hydrogeological capacity of 673 gpm.

Based on the model, the existing raw water transmission system was shown to have sufficient capacity to convey the existing capacity from all raw water supply wells.

4.7.2 Future Capacity Analysis

As presented in Figure 4-7, the existing well capacity is deficient in meeting the future maximum day demands without further maintenance and improvements. Therefore, the future model scenarios assume that future improvements are implemented to meet the future 2055 maximum day demand of 40.9 MGD.

The higher flows generated from meeting the maximum day demands increases pressures in the existing raw water supply transmission system by approximately 10 to 22 psi. Nearly half of the pressure increase is due to smaller piping in the plant downstream of the raw water flow meter. The model indicated the existing pumps performed within the existing pump curves even with the pressure increase.

As detailed in Section 4, the construction of HCW 5 is projected to serve a significant portion of the supply enabling the City to meet the future maximum day demand. Construction of a raw water transmission main crossing the Kansas River is necessary as part of this CIP Project. The raw water transmission assessment reviewed alternative sizing of a transmission line crossing the river to convey flow from the future HCWs. Alternative 2, is the recommended alternative for best value, however, that recommendation and timing of construction should be reevaluated in any subsequent Master Plan Updates.

4.7.3 Recommendations and Cost Estimates

Additional or upsized piping inside the plant beyond the raw water meter is recommended in parallel with the construction of HCW 5 to minimize negative impacts to WTP2 due to the increased flows. The estimated cost for this improvement is included.
with the Lime Feed and Basin Modifications Project cost estimate being completed by HDR, Inc. at the time of this report.

No other improvements to the existing raw water transmission system are expected based on the future demand projections outlined in this Water Master Plan Update.

4.8 Raw Water Supply CIP

The recommended Capital Projects to meet water the current raw water supply deficit through 2055 are presented in Table 4-31 and presented in Figure 4-14.

Table 4-31. Raw Water Supply Capital Improvements Plan

<table>
<thead>
<tr>
<th>Capital Project</th>
<th>Project Duration</th>
<th>Cost ¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maintenance of Existing Wells</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cleaning VWs ¹</td>
<td>2017</td>
<td>$200,000</td>
</tr>
<tr>
<td>Cleaning HCWs ²</td>
<td>2017</td>
<td>$880,000</td>
</tr>
<tr>
<td>Replacement of VWs</td>
<td>2018-2020</td>
<td>$9,650,000</td>
</tr>
<tr>
<td>River Crossing Transmission Pipeline ⁴</td>
<td>2028-2030</td>
<td>$21,339,000</td>
</tr>
<tr>
<td>Construction of HCW 5 ⁴</td>
<td>2028-2030</td>
<td>$9,784,000</td>
</tr>
<tr>
<td>Construction of HCW 6 ⁵</td>
<td>2055+</td>
<td>$7,482,000</td>
</tr>
<tr>
<td>Construction of HCW 7 ⁵</td>
<td>2055+</td>
<td>$8,652,000</td>
</tr>
<tr>
<td>Total Capital Cost</td>
<td></td>
<td>$57,987,000</td>
</tr>
</tbody>
</table>

Note: Cost estimate does not include costs for City of Olathe staff.

¹ Total costs presented in 2017 dollars.
² Total cost for cleaning all VWs ($100,000 per year for two years).
³ Total cost for cleaning all HCWs ($220,000 per year for four years).
⁴ Construction planned to begin in 2019 on HCW 5 and transmission pipeline. Project is included in the City’s 2016 to 2020 CIP.
⁵ Assumes Alternative 2 with a 36-in carrier pipe and 48-in tunnel. This recommendation and timing of construction should be reevaluated in any subsequent Master Plan Updates.

Additional Capital Projects already included the City’s current 2016 to 2020 CIP that have been addressed in the Water Treatment Plant 2 Rehabilitation Report by Black & Veatch and referenced in this Water Master Plan Update include:

- WTP2: Electrical/Backup Power – Chemical Feed and Electrical Modifications: recommends generators be installed at HCW 3 and 4.
- Remote Facilities Improvements: recommends installation of lightning protection at the HCW and VW field scheduled for 2020 and beyond.

It is recommended the City complete the following activities in addition of the Capital Projects detailed above to ensure the City can meet the current raw water supply deficit:
• A detailed geotechnical and geophysical evaluation of the soil strata be performed as part of the HCW 5 Transmission Pipeline Design including a review by large-diameter HDD and tunneling contractors.

• The lateral centerline elevations and minimum pumping level elevations currently set in SCADA should be confirmed to ensure the City is utilizing the HCWs most efficiently.

• A redundant power feed loop should be installed for the VW field.

• Perfection and certification of the City’s existing wells should continue as detailed in Section 4.1.3.
Figure 4-14. Raw Water Supply – 30-year Improvements Plan
5 Water Distribution System Assessment

A hydraulic model of the water distribution system was developed and calibrated using Innovyze InfoWater Executive Suite 12.2. The development process is described in detail in Appendix F. The model was used to assess the performance of the existing system and the future system as demands increase into the future.

The water distribution system assessment was performed based on two planning periods: existing conditions (2015) and future conditions (2020 to 2035). The ultimate build-out (2055) of the system was assessed to inform the need of system improvements for long term benefit. Based on the modeling results, it was determined that the improvements recommended through 2035, as presented below, address the projected demands through the 2055.

Evaluation criteria have been established for analyzing the distribution system performance of the under existing and future conditions (see Table 5-1).

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pressure</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum</td>
<td>psi</td>
<td>40</td>
</tr>
<tr>
<td>Working</td>
<td>psi</td>
<td>60</td>
</tr>
<tr>
<td>Maximum</td>
<td>psi</td>
<td>120</td>
</tr>
<tr>
<td><strong>Head Loss</strong></td>
<td>ft/1000 ft</td>
<td>5</td>
</tr>
<tr>
<td><strong>Velocity</strong></td>
<td>ft/s</td>
<td>7</td>
</tr>
<tr>
<td><strong>Fire Flow</strong></td>
<td>gpm</td>
<td></td>
</tr>
<tr>
<td>1 and 2 Family Residential</td>
<td>gpm</td>
<td>1000</td>
</tr>
<tr>
<td>Multi-Family, Commercial, Industrial</td>
<td>gpm</td>
<td>3500</td>
</tr>
<tr>
<td><strong>Water Age</strong></td>
<td>days</td>
<td>10</td>
</tr>
</tbody>
</table>

5.1 Existing System Performance

5.1.1 Pressure Evaluation

As detailed in Section 2.4, the City’s existing water distribution system is made up of two pressure zones: the Main Pressure Zone and the Southeast Pressure Zone. The model indicates the existing system is able to maintain the minimum 40 pounds per square inch (psi) pressure in the majority of the distribution system with the exception of the far
southwest portion of the city during peak hour, see Figure 5-1. This is due to the lack of transmission main capacity serving three large wholesale customers (RWD #6, RWD #7 and New Century) located within this service area.

Currently, the City has proposed the 16-in Lone Elm Main and Booster Station as a CIP Project (2017 to 2019) to increase pressures within the southwest service area. The proposed Lone Elm Main and Booster Station will maintain peak hour pressures above 40 psi until 2020 demands are reached and increase fire capacity within this area to an estimated 2,500 gpm for 12-inch and larger mains. The transmission main will be approximately 20,000 LF of 16-inch main from South Lone Elm Road and I-35 west along 159th Street to New Century Parkway. The Booster Station is to be located near South Lone Elm Road and I-35. The estimated cost for the Lone Elm Project including the City’s engineering and contingencies as developed by City staff is approximately $15,266,000.

In lieu of the proposed Lone Elm Main and Booster Station, a new 16-inch main along old Highway 56, from the Hedge Lane and Dennis Avenue area extending southwest, can also serve to increase pressures within the southwest service area. The selected alternative will be based on which southwest service areas of the City develop first. The cost estimate for the Old Highway 56 Transmission Main is presented in Table 5-2.

In addition to the improvements to serve the southwest area, it is recommended that a new 36-inch transmission main be extended south from the Hedge Lane Pump Station. The Hedge Lane Main Extension can be completed in two phases. Phase 1 will involve an extension from Hedge Lane Pump Station to the Curtis Street area as part of the 5-year CIP (2017 to 2022). This will allow the improvements in the southwest service area discussed above to provide required pressures beyond 2020 demands and allow the decommissioning of Curtis Street Reservoir and Pump Station. Phase Two will continue the Hedge Lane 36-in transmission main, beyond Curtis Street, connecting to the Blackbob Pump Station discharge main within the southeast service area of the system. Phase Two is presented in further detail in Section 5.4. The cost estimates for Phase One and Two of the Hedge Lane Transmission Main Extension is presented in Table 5-3 and Table 5-4, respectively.

AMI data indicates New Century withdraws water at exceptionally high rates for short periods causing high pressure fluctuations in the service area. City staff indicates the New Century meter has a pressure sustaining valve. The sustaining valve may be either not set correctly or is malfunctioning. It is recommended the pressure sustaining valve be repaired or replaced as soon as possible to minimize pressure fluctuations while maintaining New Century demands at contracted volumes. The estimated cost to replace the existing valve is approximately $10,000, assuming the existing valve vault will be utilized. Furthermore, it is recommended that AMI data for large wholesale customers should be regularly monitored to determine any potential adverse impacts to the system.
CITY OF OLATHE, KANSAS
WATER SYSTEM PRESSURE CONTOUR MAP (2016 PEAK HOUR)

LEGEND
PRESSURE ZONE
PEAK HOUR WATER PRESSURE (PSI)
20
30
40
50
60
75
100
125
150
MAJOR HIGHWAYS
MAJOR ROADS
CITY LIMITS

PATH: W:\010221\000000000269268\GIS\MAPS\FIGURE 5-1_OLATHE - WATER SYSTEM PRESSURE CONTOUR MAP - 2016 PEAK HOUR.MXD  -  USER: P KEYHILL  -  DATE: 10/3/2017
Table 5-2. Old Highway 56 Transmission Main Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit</th>
<th>Total Price</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobilization</td>
<td>1</td>
<td>LS</td>
<td>$50,000</td>
</tr>
<tr>
<td>16-inch DIP Class 350psi – Zinc Coated &amp; Polywrap</td>
<td>13,600</td>
<td>LF</td>
<td>$1,360,000</td>
</tr>
<tr>
<td>ise</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16-inch DIP Class 350psi Restrained Joint Pipe- Zinc Coated &amp; Polywrap</td>
<td>3,500</td>
<td>LF</td>
<td>$437,500</td>
</tr>
<tr>
<td>16-inch Valves</td>
<td>10</td>
<td>LF</td>
<td>$40,000</td>
</tr>
<tr>
<td>24-inch Highway/Road Bore and Casing (Lakeshore Dr &amp; 151st St &amp; Dennis Ave)</td>
<td>375</td>
<td>LF</td>
<td>$262,500</td>
</tr>
<tr>
<td>Creek Crossings</td>
<td>250</td>
<td>LF</td>
<td>$100,000</td>
</tr>
<tr>
<td>Open Cut Road Crossing with 24” Casing &amp; Pavement Restoration</td>
<td>8</td>
<td>EA</td>
<td>$120,000</td>
</tr>
<tr>
<td>Connection to Existing Main</td>
<td>2</td>
<td>EA</td>
<td>$40,000</td>
</tr>
<tr>
<td>Hydrant/Flushing Assembly</td>
<td>5</td>
<td>EA</td>
<td>$22,500</td>
</tr>
<tr>
<td>Air Release Valve and Vault</td>
<td>5</td>
<td>LF</td>
<td>$60,000</td>
</tr>
<tr>
<td>Site Restoration, Seeding, Landscaping, Sidewalk Replacement, Etc.</td>
<td>1</td>
<td>LS</td>
<td>$50,000</td>
</tr>
</tbody>
</table>

**Subtotal** $2,542,500  
**Contingency (15%)** $381,375  
**Total Construction Estimate** $2,923,875

Easement Acquisition including Temporary Construction Easements, Administration, Legal, Permitting (including Highway Permits), Survey, Design, Plans, Specifications & Contract Documents, Bidding, Construction Administration, Construction Coordination of Tie-ins/Shutdowns, Construction Observation, Conforming to Construction Records (40%) $1,176,125

**Total Probable Project Cost** $4,100,000

Note: Cost estimate does not include costs for City of Olathe staff.
### Table 5-3. Phase One Hedge Lane Transmission Main (Extension to Curtis St) Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit</th>
<th>Unit Price</th>
<th>Total Price</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobilization</td>
<td>1</td>
<td>LS</td>
<td>$75,000</td>
<td></td>
</tr>
<tr>
<td>36-inch DIP Class 150 psi - Zinc Coated &amp; Polywrap</td>
<td>16,200</td>
<td>LF</td>
<td>$240</td>
<td>$3,888,000</td>
</tr>
<tr>
<td>36-inch DIP Class 150 psi Restrained Joint Pipe - Zinc Coated &amp; Polywrap</td>
<td>4,500</td>
<td>LF</td>
<td>$350</td>
<td>$1,575,000</td>
</tr>
<tr>
<td>36-inch Valves</td>
<td>10</td>
<td>LF</td>
<td>$10,000</td>
<td>$100,000</td>
</tr>
<tr>
<td>48-inch Highway/Road Bore and Casing (135th St, 7 HWY, Harrison St)</td>
<td>450</td>
<td>LF</td>
<td>$1,200</td>
<td>$540,000</td>
</tr>
<tr>
<td>48-inch Railroad Bore and Casing</td>
<td>150</td>
<td>LF</td>
<td>$1,400</td>
<td>$210,000</td>
</tr>
<tr>
<td>Open Cut Road Crossing with 48&quot; Casing &amp; Pavement Restoration</td>
<td>25</td>
<td>EA</td>
<td>$20,000</td>
<td>$500,000</td>
</tr>
<tr>
<td>Connection to Existing Main</td>
<td>2</td>
<td>EA</td>
<td>$20,000</td>
<td>$40,000</td>
</tr>
<tr>
<td>Hydrant/Flushing Assembly</td>
<td>4</td>
<td>EA</td>
<td>$4,500</td>
<td>$18,000</td>
</tr>
<tr>
<td>Air Release Valve and Vault</td>
<td>4</td>
<td>LF</td>
<td>$12,000</td>
<td>$48,000</td>
</tr>
<tr>
<td>Site Restoration, Seeding, Landscaping, Sidewalk Replacement, Etc.</td>
<td>1</td>
<td>LS</td>
<td>$125,000</td>
<td>$125,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td></td>
<td></td>
<td><strong>$7,119,000</strong></td>
<td></td>
</tr>
<tr>
<td>Contingency (15%)</td>
<td></td>
<td></td>
<td></td>
<td><strong>$1,067,850</strong></td>
</tr>
<tr>
<td><strong>Total Construction Estimate</strong></td>
<td></td>
<td></td>
<td><strong>$8,186,850</strong></td>
<td></td>
</tr>
<tr>
<td>Easement Acquisition including Temporary Construction Easements,</td>
<td></td>
<td></td>
<td></td>
<td><strong>$3,273,150</strong></td>
</tr>
<tr>
<td>Administration, Legal Permitting (including Highway and Railroad</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permits), Survey, Design, Plans, Specifications &amp; Contract Documents,</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bidding, Construction Administration, Construction Coordination of</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tie-ins/Shutdowns, Construction Observation, Conforming to Construction Records (40%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total Probable Project Cost</strong></td>
<td></td>
<td></td>
<td><strong>$11,460,000</strong></td>
<td></td>
</tr>
</tbody>
</table>

Note: Cost estimate does not include costs for City of Olathe staff.
<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Total Price</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobilization</td>
<td>1</td>
<td>LS</td>
<td>$75,000</td>
</tr>
<tr>
<td>36-inch DIP Class 150 psi - Zinc Coated &amp; Polywrap</td>
<td>12,500</td>
<td>LF</td>
<td>$3,000,000</td>
</tr>
<tr>
<td>36-inch DIP Class 150 psi Restained Joint Pipe - Zinc Coated &amp; Polywrap</td>
<td>3,500</td>
<td>LF</td>
<td>$1,225,000</td>
</tr>
<tr>
<td>36-inch Valves</td>
<td>8</td>
<td>LF</td>
<td>$80,000</td>
</tr>
<tr>
<td>48-inch Highway/Road Bore and Casing (I-35, 7 Murlen Ln)</td>
<td>650</td>
<td>LF</td>
<td>$780,000</td>
</tr>
<tr>
<td>48-inch Railroad Bore and Casing</td>
<td>550</td>
<td>LF</td>
<td>$770,000</td>
</tr>
<tr>
<td>Open Cut Road Crossing with 48” Casing &amp; Pavement Restoration</td>
<td>21</td>
<td>EA</td>
<td>$420,000</td>
</tr>
<tr>
<td>Connection to Existing Main</td>
<td>2</td>
<td>EA</td>
<td>$40,000</td>
</tr>
<tr>
<td>Hydrant/Flushing Assembly</td>
<td>4</td>
<td>EA</td>
<td>$18,000</td>
</tr>
<tr>
<td>Air Release Valve and Vault</td>
<td>4</td>
<td>LF</td>
<td>$48,000</td>
</tr>
<tr>
<td>Site Restoration, Seeding, Landscaping, Sidewalk Replacement, Etc.</td>
<td>1</td>
<td>LS</td>
<td>$100,000</td>
</tr>
</tbody>
</table>

Subtotal $6,556,000

Contingency (15%) $983,400

Total Construction Estimate $7,539,400

Easement Acquisition including Temporary Construction Easements, Administration, Legal, Permitting (including Highway and Railroad Permits), Survey, Design, Plans, Specifications & Contract Documents, Bidding, Construction Administration, Construction Coordination of Tie-ins/Shutdowns, Construction Observation, Conforming to Construction Records (40%) $3,000,600

Total Probable Project Cost $10,540,000

Note: Cost estimate does not include costs for City of Olathe staff.
5.1.2 Fire Flow Evaluation

The model was used to evaluate the maximum available fire flow assuming a minimum 20 psi pressure is maintained in the distribution system. As detailed in Table 5-1, the City fire flow targets are 1,000 gpm for residential areas and 3,500 gpm for commercial and industrial areas. The existing system is not able to provide required fire flow to the far southeast portion of the distribution system near the executive airport due to the lack of transmission main capacity, see Figure 5-2. If the Phase One Hedge Lane improvements are completed, modeling indicates the system will be able to provide the necessary fire flows, with the exception of some isolated dead end mains.

Isolated dead end mains may have fire flows at or below targets due to the limited diameter and length. These dead end locations will require upsizing of the dead end main or looping to increase fire flow capacity. It is recommended the City performs further investigation of dead end mains to determine if upsizing or looping is required.

5.1.3 Water Age Evaluation

A 15-day water age scenario was run for existing conditions (2015). The City’s maximum water age target is 10 days (240 hours). Most areas were able to meet the target water age given that all tanks are regularly cycled daily at a minimum of 25-percent of the tank volume, see Figure 5-3. Areas near the boundary of the two pressure zones and throughout the Southeast Pressure Zone have higher water age – approximately 120 to 180 hours (5 to 7.5 days). Consolidating the system into a single pressure zone will improve water age throughout the system to approximately 48 to 72 hours (2 to 3 days). The benefits of a single pressure zone are presented in further detail in Section 5.4.

It can be time intensive to model demand loads on every single system node; therefore, certain isolated dead end mains were modeled with no demand. As a result the model indicates that these locations exceeded the target water age. Therefore, it is recommended that isolated dead end mains be further reviewed on a case by case basis to determine if the dead end requires any necessary improvements to decrease water age.
LEGEND

PRESSURE ZONE

AVERAGE DAY WATER AGE (HRS)

12
24
48
72
120
180
240
MAJOR HIGHWAYS
MAJOR ROADS
CITY LIMITS

CITY OF OLATHE, KANSAS
WATER SYSTEM WATER AGE CONTOUR MAP (2016 AVERAGE DAY)

FIGURE 5-3
5.2 Future System Performance (2020 to 2035)

5.2.1 Pressure Evaluation

The model indicates the system can maintain the minimum 40 psi pressure for the future planning period given the recommended Lone Elm Main and Booster Station and the Phase One Hedge Lane Main Extension are completed, as presented in Section 5.1.1, Figure 5-4. Construction of the Lone Elm Main and Booster Station (or Highway 56 Main Extension) will provide improved service to the southwest service area of the City until 2020 demands are reached. Once the projected 2020 demands are reached, the proposed Phase One Hedge Lane Transmission Main to Curtis Street will need to be completed in order to maintain adequate pressures within the southwest area in the future.

5.2.2 Fire Flow Evaluation

Fire supply scenarios were evaluated for the future planning period using a maximum day steady state application to confirm a minimum 20 psi pressure is maintained in the distribution system. If the proposed capital improvements are completed, as discussed previously in Section 5.1.1, the system can provide required fire flows under future conditions, see Figure 5-5. Similar to the existing conditions, some isolated dead end mains may remain at or below targets due to the limited diameter and length. It is recommended the City performs further investigation of dead end mains to determine if upsizing or looping is required.

5.2.3 Water Age Evaluation

A 15-day water age scenario was run for future conditions (2020 to 2035). The City’s maximum water age target is 10 days (i.e., 240 hours). Similar to the existing conditions, the model indicates for future conditions that most areas were able to meet the target water age given that all tanks are regularly cycled daily at a minimum of 25-percent of the tank volume, see Figure 5-6. Areas near the boundary of the Southeast and Main Pressure Zones will continue to have higher water age. Consolidating the system into a single pressure zone will improve water age throughout the system. Figure 5-7 presents the improved water age assuming conversion to a single pressure zone.

Similar to the existing conditions, isolated dead end mains will require additional review on a case by case basis to determine if the dead end main requires flushing or other improvements to improve water age.
FIGURE 5-7
WATER SYSTEM WATER AGE CONTOUR MAP (2035 AVERAGE DAY WITH SINGLE ZONE)
5.3 Storage Evaluation

Storage within a water distribution system must consist of equalization storage, fire storage and emergency storage.

Equalization Storage. Equalization storage is the amount of storage utilized to meet diurnal variations in demands. Equalization storage in elevated water tanks is considered the volume above the elevation that will maintain the minimum desirable pressure under normal conditions (e.g., 40 psi). Equalization storage in ground storage tanks with booster pumps can be considered to be the full depth, or a portion thereof, since it is pumped.

Fire Storage. Fire storage is the amount of storage required for fighting fires determined based on the maximum fire flow rate of 3,500 gpm for a duration of 3 hours. This flow and duration results in 0.63 MG of available volume required within each pressure zone. Fire storage in elevated tanks is the volume below the equalization volume but above the elevation that will maintain a minimum pressure of 20 psi within the zone or system. Similar to equalization storage, the full depth of a ground storage tank can be used for fire storage.

Emergency Storage. Emergency storage is the amount of storage held in reserve for extended disruptions in water supply. This amount of storage varies from community to community depending on the reliability of water supply, frequency of storage tank outages, and other factors. If there was an extended outage at WTP2 such that the City could not supply its customers, the following actions are taken by the City:

- The City will issue a Water Emergency Notice, prohibiting all outdoor watering with potable water and promoting smart indoor water use. This is estimated to have an effect of reducing the average daily demand by 50-percent; and
- The City will purchase water from WaterOne.

As presented in Section 2.4, there is currently 15 MG of storage within the water distribution system. As part of the Alternative Disinfection Project at WTP2, the system storage volume will likely increase. Storage requirements for existing, future, and ultimate conditions are presented in Table 5-5. Currently, there is a surplus of storage within the system. From a volume perspective, no additional storage will be required through the planning periods.
Table 5-5. Storage Requirements

<table>
<thead>
<tr>
<th>Storage Component</th>
<th>Storage Requirements (MG)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2015</td>
</tr>
<tr>
<td><strong>Main Zone</strong></td>
<td></td>
</tr>
<tr>
<td>Equalization</td>
<td>3.34</td>
</tr>
<tr>
<td>Fire Flow</td>
<td>0.63</td>
</tr>
<tr>
<td>Emergency</td>
<td>1.67</td>
</tr>
<tr>
<td>Subtotal</td>
<td>5.65</td>
</tr>
<tr>
<td>Existing</td>
<td>8.5</td>
</tr>
<tr>
<td><strong>Southeast Zone</strong></td>
<td></td>
</tr>
<tr>
<td>Equalization</td>
<td>1.66</td>
</tr>
<tr>
<td>Fire Flow</td>
<td>0.63</td>
</tr>
<tr>
<td>Emergency</td>
<td>0.83</td>
</tr>
<tr>
<td>Subtotal</td>
<td>3.11</td>
</tr>
<tr>
<td>Existing</td>
<td>6.5</td>
</tr>
<tr>
<td><strong>Summary</strong></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>8.76</td>
</tr>
<tr>
<td>Existing</td>
<td>15.0</td>
</tr>
</tbody>
</table>

Currently, there is no elevated water storage (i.e., floating storage) within the distribution system with the exception of temporary elevated storage at Blackbob. The Main Pressure Zone has temporary elevated storage when the Blackbob fill valve is fully open allowing two-way flow in and out of the Blackbob Storage Tank. In situations in which the Blackbob valve is closed, the Main Pressure Zone becomes a closed system with no elevated storage. The Southwest Pressure Zone does not have elevated storage and is thus a closed system.

Most commonly, water must be pumped from storage into the City’s system. This type of system is considered a closed system. A benefit of pumped storage is the full depth of the storage tank can be used for equalizing demands and for providing water for fires and other emergencies. However, there are certain negative considerations of closed systems, as follows:

- Pressure surges have nowhere to relieve pressure within the system resulting in higher potential for main breaks.
- Maintaining relatively constant pressures within the system requires continuous operator attention to adjust pump speeds. Alternatively, if the pump station is set at a constant speed and not adjusted as demands in the system vary, large pressure fluctuations can occur. Due to the continuous operator attention required, the system can not be effectively automated.
• Pumps within closed systems must be sized to meet fire flow demands resulting in characteristically larger and higher horsepower pumps than those systems with elevated storage. These oversized pumps can run constantly at lower speeds, likely operating less efficiently along the pump curve.

• Pump stations that pump from ground storage are subject to power failures that affect the ability to supply water, whereas systems that rely on elevated storage are not impacted by power for shorter durations.

• Maintenance costs for pumps in a closed system are greater; elevated storage will eliminate the need of some of the pump stations required for pumping out of storage.

Long term operational benefits can be realized by moving from a closed system by converting to single pressure zone. This is presented in further detail in Section 5.4.

### 5.4 Operational Enhancements

#### 5.4.1 Existing System Operational Enhancements

The existing valve and piping arrangement at the Blackbob 2 Pump Station does not currently allow for bypassing of the Blackbob 2 Storage Tank for maintenance. It is recommended that a connection be made between the 24-inch Blackbob 2 fill line and the 36-inch pump suction line to allow for tank bypassing as part of the 5-year CIP (2017 to 2022). The recommended pipe size for the connection is 24-inch with a valve to allow the bypass to remain closed during normal operation.

The cost estimate for the Blackbob 2 Bypass Improvements is presented in Table 5-6.

#### Table 5-6. Blackbob 2 Bypass Improvements Cost Estimate

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobilization, Seeding &amp; Site Restoration</td>
<td>1</td>
<td>LS</td>
<td>$30,000</td>
</tr>
<tr>
<td>Connection to Existing 24&quot; Main</td>
<td>2</td>
<td>EA</td>
<td>$20,000</td>
</tr>
<tr>
<td>24&quot; Valve</td>
<td>1</td>
<td>EA</td>
<td>$6,000</td>
</tr>
<tr>
<td>24&quot; Bends and Fittings</td>
<td>1</td>
<td>LS</td>
<td>$10,000</td>
</tr>
<tr>
<td>24&quot; DIP Class 250 psi Restrained Joint Pipe</td>
<td>50</td>
<td>LF</td>
<td>$200</td>
</tr>
<tr>
<td>Hydrant/Blow Off Assembly/Access Point for Air Release, Flushing and Chlorination</td>
<td>1</td>
<td>LF</td>
<td>$4,500</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td></td>
<td></td>
<td><strong>$100,500</strong></td>
</tr>
<tr>
<td>Contingency (15%)</td>
<td></td>
<td></td>
<td>$15,075</td>
</tr>
<tr>
<td><strong>Total Construction Estimate</strong></td>
<td></td>
<td></td>
<td><strong>$115,575</strong></td>
</tr>
<tr>
<td>Administration, Legal, Survey, Design, Plans, Specifications &amp; Contract Documents, Bidding, Construction Administration, Construction Coordination of Tie-ins/Shutdowns, Construction Observation, Conforming to Construction Records, Permitting (45%)</td>
<td></td>
<td></td>
<td>$52,425</td>
</tr>
<tr>
<td><strong>Total Probable Project Cost</strong></td>
<td></td>
<td></td>
<td><strong>$168,000</strong></td>
</tr>
</tbody>
</table>

Note: Cost estimate does not include costs for City of Olathe staff.
5.4.2 Future System Operational Enhancements (2020 to 2035)

Long term operational benefits can be realized by moving from a closed system and by converting to a single pressure zone. The range of elevations within the distribution system indicates a single pressure zone is possible. The highest elevations within the Southeast zone are approximately 1090 feet, which is not significantly greater than the high elevation within the main zone of 1080 feet. Consolidating the system into a single pressure zone will improve necessary operational oversight and water age throughout the system.

If the distribution system is converted into a single zone system then the valving between the Blackbob supply and discharge lines will be open during normal operations. The existing fill valve for the Blackbob 2 Storage Tank will still maintain the ability to remotely shut to allow occasional drawdown of the tank for water quality purposes. The Blackbob supply and discharge mains are 36-inch diameter but are only connected by a 16-inch pump station bypass. It is recommended that the 16-inch bypass piping be upsized to a 36-inch pipe as part of the single zone conversion improvements to eliminate the bottleneck caused by the existing bypass.

The cost estimate for the Blackbob 2 Piping Improvements is presented in Table 5-7.

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Total Price</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobilization, Seeding &amp; Site Restoration</td>
<td>1</td>
<td>LS</td>
<td>$30,000</td>
</tr>
<tr>
<td>Connection to Existing Main</td>
<td>2</td>
<td>LS</td>
<td>$50,000</td>
</tr>
<tr>
<td>36&quot; Valve</td>
<td>1</td>
<td>EA</td>
<td>$10,000</td>
</tr>
<tr>
<td>36&quot; Bends and Fittings</td>
<td>1</td>
<td>LS</td>
<td>$7,500</td>
</tr>
<tr>
<td>36&quot; DIP Class 150 psi Restrained Joint Pipe - Zinc Coated &amp; Polywrap</td>
<td>250</td>
<td>LF</td>
<td>$87,500</td>
</tr>
<tr>
<td>Hydrant/Blow Off Assembly/Access Point for Air Release, Flushing and Chlorination</td>
<td>1</td>
<td>LF</td>
<td>$4,500</td>
</tr>
</tbody>
</table>

| Subtotal                                                 |          |            | $189,500    |
| Contingency (15%)                                        |          |            | $28,425     |
| Total Construction Estimate                              |          |            | $217,925    |

| Administration, Legal, Survey, Design, Plans, Specifications & Contract Documents, Bidding, Construction Administration, Construction Coordination of Tie-ins/Shutdowns, Construction Observation, Conforming to Construction Records, Permitting (45%) | | | $97,075    |

| Total Probable Project Cost                              |          |            | $315,000    |

Note: Cost estimate does not include costs for City of Olathe staff.

In addition, it is recommended that elevated storage be added to the system. If the system is converted into a single pressure zone, as a method of simplifying operations throughout the system, a single 1-million gallon (MG) elevated storage tank can be constructed. The estimated cost for the single tank is $2.75 million including engineering and contingencies. This tank can be installed at the time the proposed transmission main
from Hedge Lane to Blackbob is completed. If the single elevated storage tank has an overflow equal to the Blackbob 2 tank overflow, then both tanks may operate as elevated storage allowing occasional drawdown of Blackbob 2 for water quality purposes. If two pressure zones are maintained then a 1-MG elevated storage tank will be required in each pressure zone. The estimated cost for the two tanks is $5.5 million including engineering and contingencies.

5.5 Water Distribution System CIP

The recommended CIP Projects to address pressure, fire flow, and water age issues throughout the distribution system are presented in Table 5-8. The proposed CIP Projects which involve transmission main extensions are presented in Figure 5-8.

Table 5-8. Water Distribution System Capital Improvements Plan

<table>
<thead>
<tr>
<th>Capital Project</th>
<th>Cost ¹</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>5-year CIP (2017 to 2022)</strong></td>
<td></td>
</tr>
<tr>
<td>New Century Pressure Sustaining Valve Replacement</td>
<td>$10,000</td>
</tr>
<tr>
<td>Blackbob 2 Bypass Improvements</td>
<td>$168,000</td>
</tr>
<tr>
<td>Lone Elm Main and Booster Station ², ³</td>
<td>$15,266,000</td>
</tr>
<tr>
<td>Phase One Hedge Lane Transmission Main – Extension to Curtis Street</td>
<td>$11,460,000</td>
</tr>
<tr>
<td><strong>Future Operational Enhancements (2035)</strong></td>
<td></td>
</tr>
<tr>
<td>Conversion to Single Pressure Zone</td>
<td></td>
</tr>
<tr>
<td>Phase Two Hedge Lane Transmission Main – Extension to Blackbob</td>
<td>$10,540,000</td>
</tr>
<tr>
<td>Blackbob Piping Improvements</td>
<td>$315,000</td>
</tr>
<tr>
<td>Elevated Storage ⁴</td>
<td></td>
</tr>
<tr>
<td>Option 1 – Single Pressure Zone (1-MG storage tank required)</td>
<td>$2,750,000</td>
</tr>
<tr>
<td>Option 2 – Two Pressure Zones (2, 1-MG storage tanks required)</td>
<td>$4,400,000</td>
</tr>
<tr>
<td><strong>Total Capital Cost</strong></td>
<td>$44,909,000</td>
</tr>
</tbody>
</table>

¹ Presented in 2017 dollars.
² Cost developed by City Staff and includes City contingencies and engineering. All other cost estimates presented in the table do not include costs for City of Olathe staff.
³ Old Hwy 56 Transmission Main may be constructed in lieu of Lone Elm Project depending on which southwest service area develops first.
⁴ Location of the tank(s) to be determined at the time project is required.

It is recommended the City complete the following activities in addition of the Capital Projects detailed above:

- It is recommended that AMI data for large wholesale customers should be regularly monitored to determine any potential adverse impacts to the system.
- It is recommended the City performs a further investigation of dead end mains to determine if flushing, upsizing, or looping is required.
Proposed Lone Elm Booster Station
Hedge Lane Reservoir & Pump Station
Curtis Street Reservoir & Pump Station
BlackBob Standpipe & Pump Station

LEGEND
- Water System Facilities
  - Lone Elm Main and Booster Station
  - Old Hwy 56 Transmission Main (Optional in lieu of Lone Elm)
  - Phase One - Hedge Lane Transmission Main (Extension to Curtis St.)
  - Phase Two - Hedge Lane Transmission Main (Extension to Black Bob)
- Pressure Zone Boundary
- Major Highways
- Major Roads
- Olathe Water Service Area
- Olathe City Limits
6 Water Main Replacement Program

The City of Olathe owns approximately $444 million of water pipeline infrastructure\(^5\) within the water distribution system. As the system continues to age and deteriorate, one of the City’s primary goals is to cost effectively sustain desired service levels. To accomplish this, the City has identified a need to continuously improve the way this infrastructure is managed and renewed to ensure prudent, data-driven investment decisions are made.

The purpose of the Program is to leverage readily available data to make more consistent, transparent, and defensible decisions regarding water main replacements. This will include gaining a better understanding of broad infrastructure performance trends that will be used to estimate useful life, size sustainable renewal budgets, prioritize renewal investments, assess possible break mitigation strategies, and optimize replacement specifications based on cost and useful life expectations. The goal of this Section is to empower the City’s internal staff resources to drive the update, execution, and continuous refinement of this program.

6.1 Existing Program

6.1.1 System Overview

As shown in Section 1, the City’s distribution system consists of a wide range of pipe sizes, ages, and materials. As of the end of 2015, the system had approximately 580 miles of water mains ranging in size from 1 to 48 inches in diameter. The oldest mains in the system were installed in the 1920s; however, most mains are 1950s or later. Water mains that are smaller than 16-inches are generally considered distribution mains; while mains 16-inch and larger are considered transmission mains. Distribution mains comprise approximately 92-percent of the system; approximately 44-percent are 6-inch mains.

At the end of 2015, the water mains in the system have an average age of approximately 26 years. Figure 6-1 shows the average age of the water main in the. While the age of a particular pipe is not an accurate predictor of condition and performance, in general as pipeline infrastructure ages, that infrastructure will generally deteriorate and break more often.

In order to sustain the desired service levels for City’s customers, an optimized Water Main Replacement Program is required. The City’s existing Program and recommendations for improvements to prioritizing water main replacements are the focus of this Section.

\(^5\) Total Cost of Ownership Study, 2012.
Figure 6-1. Average Age of Water Distribution System (1950 – 2015)
6.1.2 Current Project Prioritization Methods

One of the greatest challenges faced by any water utility is how to optimize the use of funds to maintain a safe and reliable distribution system. This becomes more challenging as many of the various pipe materials installed over the years near the end of a useful service live. Recently, the City has developed various methods of prioritizing projects within the water and sewer CIPs. This section summarizes those existing prioritization methods.

Current Water Main Replacement Project Prioritization. To manage water main assets and maintain an acceptable level of service, the City established a Water Main Replacement Program in 2008, which includes prioritizing the water main replacement needs through an asset ranking structure to identify the highest priority replacement projects.

To facilitate this process, the City uses an asset ranking system that results in a condition index for each pipe of the system. The condition index is determined based on evaluation of several criteria, which are weighted according to importance as shown in Table 6-1.

<table>
<thead>
<tr>
<th>Attribute</th>
<th>Maximum Score</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure History</td>
<td>25</td>
<td>20.0%</td>
</tr>
<tr>
<td>Pipe Age</td>
<td>15</td>
<td>15.0%</td>
</tr>
<tr>
<td>Pipe Material</td>
<td>15</td>
<td>12.5%</td>
</tr>
<tr>
<td>Service Criticality</td>
<td>10</td>
<td>12.5%</td>
</tr>
<tr>
<td>Major Road Intersection</td>
<td>5</td>
<td>10.0%</td>
</tr>
<tr>
<td>Railroad Intersection</td>
<td>5</td>
<td>10.0%</td>
</tr>
<tr>
<td>Crew Observations</td>
<td>12</td>
<td>7.5%</td>
</tr>
<tr>
<td>Capacity</td>
<td>5</td>
<td>5.0%</td>
</tr>
<tr>
<td>Fire Flow</td>
<td>5</td>
<td>5.0%</td>
</tr>
<tr>
<td>Other Factors</td>
<td>3</td>
<td>2.5%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>100</strong></td>
<td><strong>100.0%</strong></td>
</tr>
</tbody>
</table>

Each pipe is given a score for each criterion; criteria scoring are shown in Table 6-2. The condition index is then determined according to the following equation:

\[
\text{Condition Index} = 100 \times \sum \frac{\text{Attribute Score}}{\text{Maximum Possible Score}} \times \text{Weight}
\]

The maximum possible score is 100 points, which is the best condition possible. A score near zero indicates poor condition, which represent the top candidates for replacement. Opportunity projects are also considered based on planned roadway projects. When a street reconstruction or widening project is planned, the water main in the area is evaluated for replacement depending on water main age and break history. Coordination of construction or roadway projects and water main replacement projects limits disruptions to the community and provides cost savings.
### Table 6-2. City of Olathe Current Condition Scoring Criteria and Ranking

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Score</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Number of Failures within Past 5 Years</strong></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>25</td>
</tr>
<tr>
<td>1</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>5+</td>
<td>0</td>
</tr>
<tr>
<td><strong>Pipe Age (Years)</strong></td>
<td></td>
</tr>
<tr>
<td>0 to 10</td>
<td>10</td>
</tr>
<tr>
<td>11 to 20</td>
<td>9</td>
</tr>
<tr>
<td>21 to 30</td>
<td>8</td>
</tr>
<tr>
<td>31 to 40</td>
<td>7</td>
</tr>
<tr>
<td>41 to 50</td>
<td>6</td>
</tr>
<tr>
<td>51 to 60</td>
<td>5</td>
</tr>
<tr>
<td>61 to 70</td>
<td>4</td>
</tr>
<tr>
<td>71 to 80</td>
<td>3</td>
</tr>
<tr>
<td>81 to 90</td>
<td>2</td>
</tr>
<tr>
<td>91 to 100</td>
<td>1</td>
</tr>
<tr>
<td>100+ / Unknown</td>
<td>0</td>
</tr>
<tr>
<td><strong>Proximity to Major Road Intersection</strong></td>
<td></td>
</tr>
<tr>
<td>751+</td>
<td>5</td>
</tr>
<tr>
<td>501 to 750</td>
<td>4</td>
</tr>
<tr>
<td>251 to 500</td>
<td>3</td>
</tr>
<tr>
<td>101 to 250</td>
<td>2</td>
</tr>
<tr>
<td>0 to 100</td>
<td>1</td>
</tr>
<tr>
<td><strong>Fire Flow Availability (gpm)</strong></td>
<td></td>
</tr>
<tr>
<td>1500+</td>
<td>5</td>
</tr>
<tr>
<td>1000 - 1499</td>
<td>4</td>
</tr>
<tr>
<td>500 - 999</td>
<td>3</td>
</tr>
<tr>
<td>0 - 499</td>
<td>2</td>
</tr>
<tr>
<td><strong>Capacity</strong></td>
<td></td>
</tr>
<tr>
<td>Diameter less than 6-in</td>
<td>0</td>
</tr>
<tr>
<td>Diameter greater than 6-in</td>
<td>10</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Score</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Crew Observation</strong></td>
<td></td>
</tr>
<tr>
<td>Very good - minor defects</td>
<td>12</td>
</tr>
<tr>
<td>Good - no deterioration</td>
<td>9</td>
</tr>
<tr>
<td>Fair - moderate deterioration</td>
<td>7</td>
</tr>
<tr>
<td>Poor - severe defects</td>
<td>5</td>
</tr>
<tr>
<td>Very Poor - immediate attention</td>
<td>2</td>
</tr>
<tr>
<td><strong>Service Criticality</strong></td>
<td></td>
</tr>
<tr>
<td>Green Space</td>
<td>10</td>
</tr>
<tr>
<td>Single-Family Residential</td>
<td>9</td>
</tr>
<tr>
<td>Multi-Family Residential</td>
<td>8</td>
</tr>
<tr>
<td>Commercial</td>
<td>7</td>
</tr>
<tr>
<td>Heavy Industry</td>
<td>6</td>
</tr>
<tr>
<td>Restaurants</td>
<td>5</td>
</tr>
<tr>
<td>Hotels</td>
<td>4</td>
</tr>
<tr>
<td>Social Services / Schools</td>
<td>3</td>
</tr>
<tr>
<td>Welfare Services / Government</td>
<td>2</td>
</tr>
<tr>
<td>Emergency Services / Medical</td>
<td>1</td>
</tr>
</tbody>
</table>

| Proximity to Railroad                        |       |
| 751+                                          | 5     |
| 501 to 750                                    | 4     |
| 251 to 500                                    | 3     |
| 101 to 250                                    | 2     |
| 0 to 100                                      | 1     |

| **Pipe Material**                             |       |
| HDPE                                          |       |
| DIP, PVC                                      |       |
| CIP, Concrete                                 |       |
| Copper, Steel                                 |       |
| GSS                                           |       |
| Transite                                      |       |
| Unknown                                       |       |

1 Pipe material rankings were not provided by the City.
Using this process, the City has selected approximately 6.5 miles of priority water main replacements for the 5-year CIP (2016 to 2020). The planned replacements are summarized in Table 6-3. As shown, the identified main replacements fall below the 5-year CIP budget amount of $2,875,000. Based on the projects that have already been identified (approximately $2.5 million per Table 6-3), and assuming the remaining budget is used for water main replacements (to total to $2.875 million), the planned system renewal rate is approximately 0.26-percent per year. At this replacement rate, the asset inventory will be renewed every 360 years. The planned replacements for 2016 to 2020 are in line with replacements historically conducted, as shown in Figure 6-2.

Table 6-3. Planned Water Line Replacement Budget (2016 to 2020)

<table>
<thead>
<tr>
<th>Year</th>
<th>Length (ft)</th>
<th>Budget Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>2016</td>
<td>5,450</td>
<td>$479,700</td>
</tr>
<tr>
<td>2017</td>
<td>7,160</td>
<td>$484,773</td>
</tr>
<tr>
<td>2018</td>
<td>9,750</td>
<td>$493,534</td>
</tr>
<tr>
<td>2019</td>
<td>4,750</td>
<td>$513,275</td>
</tr>
<tr>
<td>2020</td>
<td>7,500</td>
<td>$563,919</td>
</tr>
<tr>
<td>Total</td>
<td>34,610</td>
<td>$2,535,201</td>
</tr>
<tr>
<td>Budget</td>
<td></td>
<td>$2,875,000</td>
</tr>
</tbody>
</table>

Figure 6-2. Historical Water Main Replacements
6.1.3 System Component Inspection Procedures

**Main Breaks.** When a main break is reported, City crews collect the following data on hand-held devices that link directly to the City’s Cartegraph:

- GPS location of break;
- Type of break (e.g., blowout, old repair, pin hole, puncture, separation, shear, split);
- Cause of break (e.g., Contractor, ground shift, corrosion, water pressure, flare, bolts);
- Method of repair; and
- Asset type (main, hydrant, valve, lateral, etc.).

**Valves.** The City has 14,717 active valves in the system including hydrant service valves. These valves are in place to isolate sections of the system in the event of a main break or system maintenance. The City has developed inspection standards for the purposes of determining the need for work orders. The inspection standards evaluate the following valve information:

- Valve operation including ease of operation and condition of the operating nut;
- Presence of leakage (a pass or fail evaluation); and
- Valve box condition.

Generally, valves 12-inch and smaller are inspected every five years. City staff has indicated valves 16-inch and larger will be inspected more frequently. These inspections are recorded in Cartegraph.

**Hydrants.** There are 5,708 active hydrants within the distribution system. Hydrants and their associated valves are inspected every four years and then recorded in Cartegraph. The following items are reviewed during a hydrant inspection:

- Quality of hydrant paint;
- Leakage of hydrant;
- Ease of operation of hydrant; and
- Grade of hydrant relative to ground surface.

**Cathodic Protection.** Cathodic protection and anode test stations were installed on the 42-inch finished water transmission main when it was constructed in 2008. The City does not perform any maintenance or inspections on the anode test stations. It is not believed that any other mains in the system have cathodic protection installed other than standard polyethylene encasement.

6.1.4 Summary of Data Provided

**Break Data.** Prudent data driven investment decisions rely on high quality data. The City provided a geodatabase data set of 844 known water main breaks and other system failures through April 30, 2016. Figure 6-3 summarizes the number of breaks by year and miles of active and abandoned pipe by installation year.
The break dataset dates back to 2011, with a few data points in 2009 and 2000. Based on discussions with City staff, there is no data on main breaks prior to 2011 (paper or electronic forms). The following items were removed from the dataset:

- Breaks with an asset-type related to the sewer system;
- Failures associated with water valves; and
- Failures due to separation of bolts or contractor damage.

System Data. The City provided data regarding locations and classifications of active pipes, abandoned pipes, booster station, treatment plant, roadways. Soil characteristic data was obtained from the Web Soil Survey (SSURGO database) on November 2015. The data was prepared by the United States Department of Agriculture’s (USDAs) Natural Resources Conservation Service (NRCS) Soil Survey staff.

The water main data provided included raw water mains, transmission mains, hydrant services, and distribution mains. The data was modified to remove private mains, raw water mains, and proposed mains. Following the data clean-up there were 580 active miles of water mains and 20 miles of abandoned water mains remaining.
Figure 6-3. Summary of Break History
Break and Pipe Association. The spatial location of breaks was provided in GIS; however, breaks were not associated to the specific pipe that broke. To associate the break locations with a particular water main, HDR used a spatial analysis. ESRI’s geoprocessing tools spatially locate the pipe closest to each break point. For this analysis, the following steps were performed:

- Join each break to the nearest active pipe.
- Join each pipe to the nearest abandoned pipe.
- If the break date was not populated but the comment field identified the date of the break, then populate the break date using the comment (applicable to 6 breaks).
- Where the break SDEID is populated and matches a pipe SDEID, the break was associated to that pipe (applicable to 517 breaks).
- If the nearest pipe is active and that pipe was active at the time of the break, associate the break to the active pipe (applicable to 311 breaks).
- If the nearest pipe is inactive and that pipe was active at the time of the break, associate the break to the inactive pipe (applicable to 10 breaks).
- Manually review and override logic based on comments (applicable to 1 break).

Assumptions and Limitations. A lack of substantially complete break data, prior to 2011, limits the ability to measure system performance in age ranges and break counts for pipes that were active prior to 2011. For example, a pipe that was installed in 1960 with six recorded breaks when recorded break data became available in 2011 or later, may have experienced one or many breaks between installation and 2011. Therefore, it is difficult to determine exactly how many breaks each pipe has experienced, only that this particular 1960 pipe has experienced 6 breaks since 2011. This limitation can significantly impact results if there are only several years of break data available. Additionally, the limited data available does not allow the determination of how many breaks have been saved (and therefore dollars saved) as a result of the current Program or to project how much a future Program will potentially save. It is important to recognize this data limitation exists and it may be mitigated in the future as more break data is collected and the City has access to a larger proportion of the complete break history for every pipe. Section 6.9.1 discusses recommendations for data collection improvements.

6.2 Break Density Mapping

The historical breaks based on data provided by the City are shown in Figures 6-4. Figure 6-5 shows the historical hot spots for main breaks for all active and abandoned pipes. The raster image shows the length weighted density of breaks throughout the service area on a red to green scale where areas in red are breaking more often and areas in green are breaking less often. This figure was developed to visually determine areas that have higher than normal water main breaks. Areas of concern show the south part of the City with the highest density of breaks as well as pockets around the rest of the City.
Additional break history characteristics were analyzed, such as break rate, break count, and date of last break. Break history is typically the greatest indicator of future main breaks.
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Water Master Plan Update

Figure 6-4 Historical Water Main Break Locations
Figure 6-5  Historical Length-Weighted Break Density Map (All Pipe)

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Map Feature Key
- Olathe Water Service Area
- Water Facilities
- Transmission
- Distribution Mains

Break Density
- High
- Medium
- Low

Olathe Water Treatment Plant #2
Renner Rd Standpipe & Pump Station
Hedge Lane Reservoir & Pump Station
Black Bob Standpipe & Pump Station
Curtis St Reservoir & Pump Station

City of Olathe
City of Overland Park
City of Lenexa

101 0.5 Miles
±

1 0.5 0 1 Miles

Map Scale
Figure 6-6  Historical Length-Weighted Break Density Map (Active Pipe Only)
6.3 System Benchmarking

As stated above, based on the budget and projects selected in the 5-year CIP (2016 to 2020), the City is replacing approximately 0.26-percent per year and they average 22 breaks per year. Olathe’s average break rate is above the national average break rate of 15 – 20 breaks per year. In comparison to other known communities as shown in Table 6-4, Olathe’s break rate is higher than average and the replacement rate is lower than average.

Compared to WaterOne, also located in Johnson County, Kansas, Olathe has a similar break rate, perhaps due to similarly corrosive soils; however, WaterOne is replacing their system approximately double the rate that the City currently is.

Table 6-4. Water Distribution Replacement Rate of Select Utilities

<table>
<thead>
<tr>
<th>Utility</th>
<th>Miles of Distribution Main</th>
<th>Break Rate (Breaks per Year)</th>
<th>Annual Replacement Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phoenix, Arizona</td>
<td>6,975</td>
<td>15</td>
<td>0.47%</td>
</tr>
<tr>
<td>Des Moines, Iowa</td>
<td>1,555</td>
<td>20</td>
<td>N/A</td>
</tr>
<tr>
<td>Bellevue, Washington</td>
<td>670</td>
<td>3.5</td>
<td>0.75%</td>
</tr>
<tr>
<td>East Bay Municipal Water District, California</td>
<td>1,120</td>
<td>11</td>
<td>0.45%</td>
</tr>
<tr>
<td>Eugene, Oregon</td>
<td>830</td>
<td>8</td>
<td>0.22%</td>
</tr>
<tr>
<td>Lincoln, Nebraska</td>
<td>1,200</td>
<td>13</td>
<td>0.58%</td>
</tr>
<tr>
<td>WaterOne, Johnson County, Kansas</td>
<td>2,650</td>
<td>24</td>
<td>0.60%</td>
</tr>
<tr>
<td>Westminster, Colorado</td>
<td>500</td>
<td>10</td>
<td>0.50%</td>
</tr>
<tr>
<td>Lawrence, Kansas</td>
<td>400</td>
<td>16</td>
<td>0.88%</td>
</tr>
<tr>
<td>Olathe, Kansas (Current)</td>
<td>580</td>
<td>22</td>
<td>0.26%</td>
</tr>
</tbody>
</table>

1 Estimated values based on information from www.waterone.org. In 2015, there were 641 main breaks. In 2016, they have budgeted 43-percent of $14.4 million for water main replacements; at $75/ft (in-house replacement costs) this equates to 16 miles of pipe replaced per year.

2 Information obtained from the City of Lawrence.
6.4 System Deterioration Mechanisms

In this Water Master Plan Update, the term “cohort” refers to a subset of the water main system that has a specific characteristic. In general, the intent of the cohort analysis is to better understand broad infrastructure performance trends that will be used to estimate useful life, size sustainable renewal budgets, prioritize renewal investments, assess possible break mitigation strategies, and optimize replacement specifications based on cost and useful life expectations. In this Section, readily available City pipe data is analyzed at a macro level to:

- Validate that pipe deteriorates over time as infrastructure ages (i.e. do pipes generally break more often as they get older?).
- Determine whether deterioration is nonhomogeneous (i.e. do cohorts deteriorate at different rates?).
- If deterioration over time is nonhomogeneous, quantify which factors drive deterioration and useful life.

6.4.1 Analysis Method and Approach

For the purposes of this Study, pipe deterioration rates were measured as a function of infrastructure age verses break rate (in terms of annual breaks per 100 miles of active pipe). Full year break data was available between 2011 and 2015.

Break Rate Example. The break rate calculation is the number of breaks that occurred at a particular pipe age and the length of active pipe in miles at a particular age when break data were collected (see formula below).

\[
\text{Break Rate} = \frac{100 \times \text{Number of Breaks}}{\text{Miles of Main}}
\]

In order to illustrate how the break rate is determined, the below example considers breaks within the system that were 7 year old pipe at the time of the break. As shown in Table 6-5, there have been four pipes in the system that have had a break occur when the pipe was 7 years old.

Table 6-5. Summary of Breaks on 7-Year Old Pipe

<table>
<thead>
<tr>
<th>System Pipe with Break (4 total)</th>
<th>Installation Year</th>
<th>Break Year</th>
<th>Age When Break Occurred</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe 1</td>
<td>2004</td>
<td>2011</td>
<td>7</td>
</tr>
<tr>
<td>Pipe 2</td>
<td>2004</td>
<td>2011</td>
<td>7</td>
</tr>
<tr>
<td>Pipe 3</td>
<td>2005</td>
<td>2012</td>
<td>7</td>
</tr>
<tr>
<td>Pipe 4</td>
<td>2006</td>
<td>2013</td>
<td>7</td>
</tr>
</tbody>
</table>

Table 6-6 shows miles of pipe that were 7 years old at the time break data was collected between 2011 and 2015. Based on this information, approximately 64.8 miles of the total 580 miles in the system were 7 years old during the period break data was being collected.
Table 6-6. Summary of Pipes 7-Years Old When Break Data Were Collected (2011 – 2016)

<table>
<thead>
<tr>
<th>Age of Pipe</th>
<th>Installation Year</th>
<th>Year Pipe Was 7 Years Old</th>
<th>Pipe Length (Mi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>2004</td>
<td>2011</td>
<td>13.7</td>
</tr>
<tr>
<td>7</td>
<td>2005</td>
<td>2012</td>
<td>9.2</td>
</tr>
<tr>
<td>7</td>
<td>2006</td>
<td>2013</td>
<td>17.7</td>
</tr>
<tr>
<td>7</td>
<td>2007</td>
<td>2014</td>
<td>8.5</td>
</tr>
<tr>
<td>7</td>
<td>2008</td>
<td>2015</td>
<td>15.7</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>64.8</td>
</tr>
</tbody>
</table>

Based on the data collected for 7 year old pipe, the “Number of Breaks” is equal to 4 and the “Miles of Main” is equal to 64.8. Therefore, the break rate at age 7 is calculated as:

Example Break Rate = \( \frac{100 \times 4}{64.8} \) = 6.2 annual breaks per 100 miles

This analysis requires that each pipe analyzed has an installation date and an abandoned date (if abandoned). Any pipes or breaks associated to a pipe without this information was excluded. As discussed above, pipe and breaks associated with large diameter pipe were also removed. As a result, 579 miles of active water mains, 3.2 miles of abandoned water mains, and 664 breaks were analyzed. Figure 6-7 shows a scatter graph of the system length, count of breaks, and break rate by age.

To obtain more statistically relevant information, Figure 6-8 groups data into five-year age ranges, with the exception of age zero. A zero age is separated to better quantify the impact of early failure\(^6\) in the system. Age ranges with less than 10 miles of data have been removed.

The data indicates that the City is experiencing some early failure as the break rate (28 annual breaks per 100 miles) of pipe less than 1 year of age is roughly tens time higher than that of pipe that is 1 to 10 years old (2.9 annual breaks per 100 miles). Based on the number of breaks, deterioration over the first 65 years of life in the City’s system appears to be linearly increasing from approximately 0 breaks at age 1 to 5 and increasing to approximately 70 breaks by age 65. While Figure 6-8 indicates deterioration in the system, some groups of pipes (i.e., cohorts) deteriorate much faster or slower than others. These cohorts, pipes with similar characteristics, will be explored in more detail to better understand how the system is deteriorating, what level of investment is needed to sustain desired service levels, and how to focus those investments to get the greatest return.

---

\(^6\) Water mains often exhibit a failure pattern that can best be modeled by the Bathtub Curve hazard function. This function includes elevated failure rates which decrease with time known as Early Failure. In water mains, if Early Failure exists, it is most often observed in months immediately following installation.
Figure 6-7. Summary of Data Used to Calculate Break Rates
While this composite deterioration rate confirms that the City’s pipes are generally deteriorating as they age, there are likely cohorts of pipes that are deteriorating much faster or much slower than the composite rate. Based on industry experience and institutional knowledge from City staff, the following deterioration factors were analyzed. Factors in **bold** were found to have a strong correlation with break rates in Olathe’s system (i.e., material vintage and soil shrink/swell potential).

- **Material Vintage** – Includes pipe material and installation year which is meant to capture significant changes in the quality of manufacturing, installation, and corrosion protection that occurred over time.
- Diameter
- Soils Characteristics
  - Steel Corrosion Potential
  - Freeze/Thaw Potential (i.e. ground movement loading)
  - **Soil Shrink/Swell Potential** (i.e. ground movement loading)
- Proximity to Pump Stations (i.e. Cyclic Pressure Loading Potential)
- Roadway Type (i.e. Surface Loading)
6.4.2 Factors Influencing Deterioration

The purpose of this section is to quantify the relationship between deterioration and the factors that showed a strong correlation to deterioration. These factors are summarized below:

- Material Vintage
- Diameter
- Soil Shrink Swell Potential
- Corrosion Potential

Each subsection:

- Describes the theory regarding why a relationship may exist
- Describes how and why infrastructure was grouped into cohorts
- Summarizes the system by cohort
- Quantifies the relationship between the factor and deterioration rate

Material Vintage. Material vintage seeks to establish a relationship between deterioration rate and a combination of pipe material and installation era which may indicate significant changes in manufacturing, installation, and corrosion protection quality. The City’s system is predominantly made up of metallic pipe installed in 1955 or later.

In order to develop statistically relevant data sets, infrastructure and breaks were grouped into cohorts based on observed changes in infrastructure performance in the system, industry guidelines regarding the timing of significant advances in manufacturing and installation practices, and the development of statistically relevant cohorts.

A preliminary analysis of pipe deterioration by material and installation date showed homogeneous deterioration rates with the exception of a time period in the 1990s. Metallic pipe installed prior to 1991 and between 1994 and 1998, during City’s rapid population increase, (called “Poor Vintage”) deteriorated at a similar rate and much more rapidly than the rest of the system (called “Good Vintage”). At the time of this Study, it is unclear what may have caused this change. No major advancement in the manufacturing of ductile iron pipe occurred in this timeframe. The change in performance could be related to the City’s-specific modifications to design and corrosion protection standards.

Figure 6-9 quantifies the deterioration rate of each material vintage cohort. Each point shown in the graph contains at least 10 miles of data. The graph shows that even at the same age Poor Vintage pipe breaks more often and deteriorates faster than Good Vintage pipe. For example, at age 21-25, the break rate of Poor Vintage pipe is roughly 6 times worse than Good Vintage pipe. Figure 6-10 shows the material vintage classifications for the system geographically.

A summary of infrastructure by material vintage cohort is included in Table 6-7.
### Table 6-7. System Mileage by Material Vintage

<table>
<thead>
<tr>
<th>Material Vintage</th>
<th>Abandoned Miles</th>
<th>Active Miles</th>
<th>Total Miles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good Vintage</td>
<td>0.9</td>
<td>203.8</td>
<td>204.7</td>
</tr>
<tr>
<td>Poor Vintage</td>
<td>19.4</td>
<td>376.8</td>
<td>396.1</td>
</tr>
<tr>
<td>Total</td>
<td>20.3</td>
<td>580.6</td>
<td>600.8</td>
</tr>
</tbody>
</table>

![Deterioration Rates by Material Vintage](image)

**Figure 6-9. Deterioration Rates by Material Vintage**
Map Feature Key

- Olathe Water Service Area
- Water Facilities
- Pressure Zone Boundary

Material Vintage

1 - Poor Vintage Metallic
2 - Unknown
3 - Asbestos Cement
4 - Good Vintage Metallic
5 - PVC, HDPE, Steel

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Figure 6-10 Material Vintage
Deterioration Factor
Diameter. Industry experience suggests that small diameter pipe deteriorates at a more rapid rate than large diameter pipes. In theory, this is related to thinner pipe walls and the generally smaller section modulus associated with smaller diameter pipe. Additionally, large diameter pipes commonly have more stringent application of design, installation, testing, and construction inspection leading to longer service lives. Pipe diameters in the City’s system range from 1 to 48 inches; however, the majority is between 6 and 18 inches. In order to develop statistically relevant data sets, infrastructure and breaks were grouped into cohorts based on observed changes in infrastructure performance and development of statistically relevant cohorts. Table 6-8 summarizes the asset groups assessed and summarizes the miles of pipe in each group.

Table 6-8. System Mileage and Breaks by Diameter

<table>
<thead>
<tr>
<th>Diameter Classification</th>
<th>Abandoned Miles</th>
<th>Active Miles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 6 inch</td>
<td>6.8</td>
<td>14.5</td>
</tr>
<tr>
<td>6 inch</td>
<td>3.3</td>
<td>243.7</td>
</tr>
<tr>
<td>8 to 16 inch</td>
<td>9.6</td>
<td>287.2</td>
</tr>
<tr>
<td>Greater than 18 inch</td>
<td>0.6</td>
<td>35.1</td>
</tr>
<tr>
<td>Total</td>
<td>20.3</td>
<td>580.6</td>
</tr>
</tbody>
</table>

Figure 6-11 quantifies the deterioration rate of each diameter cohort. Each point shown in the graph contains at least 10 miles of data. Even at the same age, smaller diameter pipe break more often and deteriorate faster than larger diameter pipe. Figure 6-12 shows the diameter groupings.
Shrink Swell Potential. Industry experience suggests that the reliability of pipes exposed to larger fluctuations in soil shrinkage and swelling will deteriorate faster. In theory, this is related to material fatigue and stresses imposed on the pipe during soil shrinkage and swelling. The severity of this loading is dependent upon the relative variability of moisture content in the system and a soil property called linear extensibility. Linear extensibility refers to the change in length of an unconfined clod as moisture content is decreased from a moist to a dry state. It is an expression of the volume change between the water content of the clod at 1/3- or 1/10-bar tension (33kPa or 10kPa tension) and oven dryness. The volume change is reported as percent change for the whole soil. The amount and type of clay minerals in the soil influence volume change. A higher linear extensibility value generally leads to increases in cyclic loadings and a shorter useful life of pipelines.

Readily available linear extensibility data was obtained from the Web Soil Survey (SSURGO database) on November 10, 2015. The data was prepared by the USDA’s Natural Resources Conservation Service (NRCS) Soil Survey Staff. The linear extensibility percentage used for this Study is based on a depth-weighted average of all available layers. Using GIS analysis, each pipe in the system was associated to the nearest linear extensibility value. This assigned a weighted score to each pipe based on the nearest value.

Less than 1-percent of the City’s system was categorized as having a low linear extensibility. Therefore, pipes exposed to soils with low and medium linear extensibility were categorized as “lower” and pipes exposed to soils with high linear extensibility were categorized as “higher”. Table 6-9 summarizes the asset groups assessed and summarizes the mile of pipe and breaks in each group. Note that the shrink-swell potential is a measurement of how the volume of the soil will fluctuate when exposed to varying moisture contents. It does not specify for a particular area the frequency or severity of moisture variations.

<table>
<thead>
<tr>
<th>Shrink Swell Potential</th>
<th>Abandoned Miles</th>
<th>Active Miles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Higher</td>
<td>15</td>
<td>351</td>
</tr>
<tr>
<td>Lower</td>
<td>5</td>
<td>230</td>
</tr>
<tr>
<td>Total</td>
<td>20</td>
<td>581</td>
</tr>
</tbody>
</table>

Figure 6-13 quantifies the deterioration rate of each Shrink Swell Potential cohort. Each point shown in the graph contains at least 10 miles of data. The data shows that the performance of pipes is relatively similar over the first 20 years of life. However, after age 20, the cyclical wear begins to take a toll on the pipe and deterioration in the pipe exposed to higher shrink swell typically breaks at approximately twice the rate. Figure 6-14 shows the shrink swell classifications geographically across the system.
Figure 6-13. Deterioration Rate by Shrink Swell Potential
Figure 6-14  Shrink Swell Potential
Deterioration Factor

Olathe, Kansas
Water Master Plan Update

Shrink Swell
1 - Higher
2 - Not Used
3 - Not Used
4 - Lower
5 - Not Used

Map Feature Key
- Olathe Water Service Area
- Water Facilities
- Pressure Zone Boundary

Renner Rd
Standpipe & Pump Station

Black Bob
Standpipe & Pump Station

Olathe Water Treatment Plant #2

City of Olathe
City of Lenexa
City of Overland Park
Soil Corrosion Potential. Corrosion rates vary significantly due to soil type and can significantly affect a pipe’s lifespan. External pitting from corrosive soils is a known problem on metallic pipe materials including grey cast iron, ductile iron, and steel. The rate of external pitting attack on ferrous materials is governed primarily by the corrosivity of the environment, presence of water, and the presence or lack of corrosion protection. Therefore, soil type can be considered an influence on main breaks.

Several factors in the soil can affect its corrosivity potential including pH, resistivity, moisture content, oxidation-reduction (redox) potential, and organic and sulfide content. Soils of low pH (<4) serve well as an electrolyte and soils of high pH (>8.5) are often high in dissolved salts, both of which can be corrosive to metallic pipe. Soils of lower resistivity (>2000 ohms), are likely to cause more rapid pitting attack to ferrous materials at rates that increase as resistivity decreases. Prevailing moisture content due to water tables is extremely important to soil corrosion.

The redox potential of a soil is significant, because the most common sulfate-reducing bacteria can live only in anaerobic conditions. Sulfide determination is important since sulfate-reducing bacteria (SRB) can be a primary cause for the acceleration of the cathodic reaction leading to corrosion.

Corrosion as graphitization is a major factor influencing iron pipe failure. Graphitization of grey cast irons can be expected when soil conditions favor anaerobic bacterial growth with the appropriate conditions of pH, dissolved salts, and organic content. The result is a matrix consisting of a mass of residual graphite flakes interspersed with oxides of iron, which are the graphite-containing corrosion products. This material matrix leads to corrosion-induced loss of wall thickness which can eventually lead to pipe failure. Additional external corrosion can be caused by galvanic corrosion from dissimilar metals (commonly with copper services) and stray electrical currents.

Soil information was obtained through the Web Soil Survey service from the NRCS. Soils are classified in GIS by the NRCS as having either a high, moderate, or low potential for steel corrosion. Almost 90-percent of the pipes in the City’s system have high corrosion potential. Therefore, this portion of the system was categorized as having higher corrosion potential and the rest of the system was categorized as having lower corrosion potential. Table 6-10 summarizes the asset groups assessed and summarizes the mile of pipe and breaks in each group.

<table>
<thead>
<tr>
<th>Corrosion Potential</th>
<th>Abandoned Miles</th>
<th>Active Miles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Higher</td>
<td>15</td>
<td>351</td>
</tr>
<tr>
<td>Lower</td>
<td>5</td>
<td>230</td>
</tr>
<tr>
<td>Total</td>
<td>20</td>
<td>581</td>
</tr>
</tbody>
</table>

Figure 6-15 quantifies the deterioration rate of each Corrosion Potential cohort. Each point shown contains at least 3 miles of data (a smaller threshold was required compared to other cohorts because “lower” represents less than 11-percent of the system). The data shows that the performance of pipe is relatively similar over the first 15 years of life.
However, after age 15, the pipe exposed to higher corrosion potential typically break at approximately twice the rate.

Figure 6-15. Deterioration Rate by Corrosion Potential

HDR also performed insitu soil corrosivity testing at various locations across the system in April, 2016. The Report from this testing is contained in Appendix H. The testing found that between 0 to 15 foot soil depth profile, there is an 83 percent probability that a pipeline will experience severely corrosive or corrosive soil; the remaining 17 percent is moderately corrosive.

Figure 6-16 shows the soil corrosion potential deterioration factor geographically across the system.
Olathe, Kansas
Water Master Plan Update
Figure 6-16 Soil Corrosion Potential Deterioration Factor

Map Feature Key
- Olathe Water Service Area
- Water Facilities
- Pressure Zone Boundary

Soil Corrosion
1 - Higher
2 - Not Used
3 - Not Used
4 - Lower
5 - Not Used
6.5 Useful Life Quantification

The useful life of a water main represents the median number of years the water main is expected to remain in service from installation to replacement. Useful life estimates are meant for planning purposes only. The life of a particular pipe is highly dependent on the quality of construction, the quality of the manufacturing, the pipe’s unique environment, and other pipe specific characteristics, all of which are eventually reflected in its break history.

In the United States, life expectancies of water mains typically range anywhere from 50 to over 300 years. This broad range is due to a variability of installed conditions, system operations, pipe manufacturing, and materials. As a result, pipe age by itself is a poor predictor of pipe condition. In addition, the “failure” of a pipe is not a definitive event. A water pipe can be made to last indefinitely, as long as a utility is willing to repair it. Different utilities choose to manage systems differently, as a result, have different life spans for pipelines.

For the purposes of this Study, a Weibull model was used to quantify useful life. The Weibull Distribution is a commonly used methodology to assess asset life expectancy in the water industry. The results of this model will be compared to industry and regional benchmarks for replacement rates at various system performance levels to validate the model results.

6.5.1 Assumptions

The purpose of this Section is to describe the assumptions used to build various renewal scenarios for water mains. Useful life models are dependent upon the definition of the asset (i.e. where does one pipe end and the next begin) and the definition of failure (i.e., at what threshold does the owner replace the asset). A description of each assumption is included below.

**Definition of an Asset.** Utilities tend to divide water mains in GIS into small lengths of pipe at diameter changes, material changes, install date changes, valves, tees/crosses, bends, and other attributes. This is ideal for some analyses (e.g. hydraulic modeling, cohort analysis, attribute data management, etc.). However, it is not an ideal method for determining useful life or for asset-specific decision making because it is not cost effective to renew infrastructure in such small units. If desktop analyses were performed on such short segments, it can lead to poor quality analysis and project selections that are not cost effective or realistic.

Currently, the City’s pipe network is divided into short assets with an average length of less than 100-feet. However, replacement projects are typically longer. Therefore, it is necessary to aggregate these short pipes into more meaningful groups which better align with how decisions will need to be made.

For this Study, GIS pipes were aggregated based on unique Project Names. If the project name was null or generic (e.g. “Waterline Rehabilitation”), the Project Year and Project Number were also concatenated to the Project Name. Based on this grouping method, the average pipe length increased from less than 100 feet to 2,419 feet, which better aligns with typical pipeline replacement projects. Industry experience suggests that material and construction quality (which are often similar in a single project) is a
primary indicator of the useful life of a pipe. In general, the City’s projects tend to be one to several contiguous groups of pipe. Therefore, using this method to identify “assets” for modeling was deemed to be the most cost effective approach.

It should be noted that pipe groupings are not meant to constrain the extent upon which asset specific decisions must be made. Rather, the intent is to group short pipes in GIS in a way that more directly aligns with the extents of which asset based decisions will be made.

**Definition of Failure.** For water mains, the Weibull model is dependent upon the definition of failure. A “failure” is meant to describe when an asset should be replaced rather than repaired. This is a planning-level definition of failure that is meant to describe the average point of failure for the entire system. The actual definition of failure for a particular asset may vary due to other risk factors (e.g. consequence of failure) and construction opportunities (e.g. renewing a roadway).

For the purpose of this analysis, approximately five years of break data was available. Based on the following definition of failure, which is typical in the industry:

- Must have at least 3 breaks; and
- Average break rate must be greater than 1 break per mile per year.

For example, all pipes less than 0.6 miles long will require at least three breaks over the five years of available break data to qualify as failed. However, for pipes between 0.6 miles and 0.8 miles, four breaks over the five years of available break data will be needed to qualify as failed.

### 6.5.2 Cohorts Modeled

Section 6.4.2 verified that while the City’s pipes are generally deteriorating as they age, there are cohorts of pipes that are deteriorating much faster or much slower than average. For the purposes of the Weibull Model, the two Material Vintage cohorts were modeled. This cohort was selected because of the clear difference in break rate between the Good Vintage and the Poor Vintage.

**Weibull Distribution Limitations.** When using a Weibull model to forecast long-term capital improvement program levels, it is important to consider that this model has certain limitations, including:

- Based on the count of assets, 3.5-percent of the system qualifies as failed; having such a small proportion of system as failed makes future failure rate projections less accurate.
- Break data was only available dating back to 2011, while the average install date of the system modeled is 1982. Therefore, some break data is not included in the model resulting in some potential error within the model. As more data points are collected and modeled over time, the accuracy of the model will increase.
- Weibull modeling and projection of failures is not useful for identifying specific assets for future remediation. The results are aggregated by cohort.
- The average age of the system modeled was approximately 28 years while the model is projecting the shortest average life expectancy of 89 years. As larger
portions of the system approach the average useful life, the accuracy of the model will increase.

- Unknown future changes (e.g. technology advancements, regulatory changes, changes in risk tolerance, changes in affordability, performance changes, etc.) may also alter the basis of the life expectancy estimates.

Weibull Distribution Input. For each cohort modeled, Table 6-11 summarizes the Weibull model input data including the count of assets that failed, did not fail, and the failure rate. The average failure rate is 3.5-percent.

Table 6-11. Weibull Distribution Input

<table>
<thead>
<tr>
<th>Cohort</th>
<th>Count Failed</th>
<th>Count Not Failed</th>
<th>Failure Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good Material Vintage</td>
<td>2</td>
<td>497</td>
<td>0.4%</td>
</tr>
<tr>
<td>Poor Material Vintage</td>
<td>42</td>
<td>722</td>
<td>5.5%</td>
</tr>
<tr>
<td>Total</td>
<td>44</td>
<td>1,219</td>
<td>3.5%</td>
</tr>
</tbody>
</table>

Model Methodology. A review of the assumptions, output needs, and data was performed. Based on this review, a Weibull probability distribution function was selected. This type of model is often used to predict time to failure due to its versatility. The shape of the curve is modeled to fit the data.

The Weibull probability density function (PDF) is shown in Equation 5-1 below. The Weibull PDF distribution is a parametric function. Its shape is determined by up to three parameters:

- \( \beta \) (beta), which determines the shape of the distribution (e.g., bell-curved);
- \( \eta \) (eta), which determines scale (linked to the units of time used in the model and identifies the characteristic life expectancy at what value 63.2-percent of the units will fail); and
- \( \gamma \) (gamma), which is a location value, representing the unit of time before failures can occur, and is equal to zero in a two-parameter distribution.

Equation 5-1: 
\[
f(t|\beta, \eta, \gamma) = \frac{\beta}{\eta} \left(\frac{t - \gamma}{\eta}\right)^{\beta-1} e^{-\left(\frac{t - \gamma}{\eta}\right)^{\beta}}, \beta > 0, \eta > 0, -\infty < \gamma \text{ and } t < \infty.
\]

The Weibull cumulative density function (CDF) is a more useful expression. The value of the CDF at time \( t \) is equal to the area under the PDF up to time \( t \). The Weibull CDF defines the Weibull unreliability function, or probability of failure. This function represents the probability of an item failing by time \( t \). This equation is shown in Equation 5-2.

Equation 5-2: 
\[
F(t|\beta, \eta, \gamma) = 1 - e^{-\left(\frac{t - \gamma}{\eta}\right)^{\beta}}, \beta > 0, \eta > 0, -\infty < \gamma \text{ and } t < \infty.
\]

For pipes that meet the failure criteria, the date of the failure and the installation date were used to determine the age of the asset at the time of failure. For assets that have not failed, the installation date was used to calculate the age of the asset at the time when break data was last available for this Study (i.e., April 30, 2016). Assets that had
not yet met the failure criteria were treated as censored observations with the assumption that they still can fail at any point in the future.

Model Output. The software package Weibull++ 7 (Reliasoft) was used to fit the Weibull distribution to the data. Based on the rate of pipes surviving over time, the software fits a distribution curve to model the underlying distribution that can best explain the observed failures. The results of the “Poor Material Vintage” model are shown in Figure 6-17.

The “Good Material Vintage” cohort only has two failed pipes. This was not enough data to develop a reasonably accurate deterioration model. The performance of the City’s “Good Material Vintage” cohort is similar to recently installed metallic pipe in Lincoln, Nebraska. Lincoln has many more years of break data and could develop a data driven Weibull curve. Therefore, this analysis uses Lincoln Nebraska curve to estimate the useful life of the City’s “Good Material Vintage” cohort; the results of the “Good Material Vintage” model are shown in Figure 6-18.

Each figure shows the probability of failure on the y-axis and the age of the pipe on the x-axis. All graphs show the unadjusted input (shown as points based on the input file), the best-fit curve, and the curve parameters (in the bottom left corner). When the data is best-fit by a three-parameter Weibull distribution, the third parameter is introduced. The shape parameters and unadjusted curve show the modeled best-fit curve in relation to the axes. The average useful life for each scenario is the median age at failure. In other words, 50-percent of pipes will fail after this age and 50-percent will fail before this age.

---

Figure 6-17. Olathe Poor Material Vintage Weibull Results

\[ \beta = 3.4345, \eta = 87.1224 \]
Figure 6-18. Good Material Vintage Weibull Results (from Lincoln, NE)
6.5.3 Summary of Results

A summary of the life expectancies and the miles of active pipe for each cohort are shown in Table 6-12.

<table>
<thead>
<tr>
<th>Cohort</th>
<th>Active Miles</th>
<th>Average Useful Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good Material Vintage</td>
<td>203.8</td>
<td>140</td>
</tr>
<tr>
<td>Poor Material Vintage</td>
<td>376.8</td>
<td>89</td>
</tr>
<tr>
<td>Total</td>
<td>580.6</td>
<td>-</td>
</tr>
</tbody>
</table>

In general, the survival model results appear plausible and reasonably consistent with the results of the break rate analysis and industry experience. While the average useful life (median life) should not be used as a precise estimate, the model results should be reasonably accurate for long-term renewal planning. As more break data are collected and modeled in the future, the accuracy and confidence level in long-term projections will increase.

6.6 System Renewal Alternatives

No model can perfectly predict the future renewal needs of any utility, but perfection is not needed. While decisions made in the near term have long-term consequences, course corrections are allowed and encouraged. As additional data is gathered, it is appropriate to re-examine the life expectancies of mains, particularly as changes occur in risk tolerance, regulations, renewal technologies, costs, and funding availability.

This Section identifies a range of renewal forecasts based on the analysis presented in the previous sections. It is anticipated that these forecasts, in conjunction with engineering and operational judgment, will enable the City to make informed renewal decisions with confidence that appropriate levels of service will be maintained. As more data is collected, the accuracy of these long-term renewal projections will increase.

Life expectancy estimates from the Weibull Model will therefore be used, in conjunction with the historic installation dates, to estimate future backlogs of failed pipe. The effects of several renewal strategies will be explored.

Figure 6-19 shows the miles of currently active pipe by installation date and by the cohorts analyzed.
6.6.1 50 Year Failure Forecast

The two Weibull Cohort Probability Distributions shown in Figures 6-17 and 6-18 for material vintage predict the percent of the cohort that will fail at each age. For each year in the 50-year renewal forecast, the age profile was determined. The system mileage by age and asset class was multiplied by the corresponding percent failed as projected by the appropriate Weibull Probability Distribution to determine the miles of pipe predicted to fail. For example, Figure 6-20 shows the age profile in 2040. There are 10 miles of currently active “Poor Material Vintage” pipe installed in 1955. This pipe will be 85 years old in 2040. Based on the Poor Material Vintage Weibull Cohort cumulative probability of failure at 85 years of age less the cumulative probability of failure at 84 years of age, 1.5-percent of the Poor Material Vintage cohort will fail at age 85 years. Therefore, the model predicts that 0.15 miles of this pipe (i.e. 85 years old) will fail in 2040. This calculation was applied to all active pipes in the system over the next 50-year period to determine the miles of pipe that will fail in each year.
The existing backlog of failed pipe was determined by applying the definition of failure to the currently active system to determine which assets have failed. At the time of this Study, 29.8 miles of active pipe was categorized as failed. The projected miles of failed pipe was determined by adding the predicted failures in future years to the existing backlog. This is summarized in Figure 6-21.
6.6.2 Interpreting the Impact of Failed Pipe Growth Trends

The backlog of failed pipe will grow or shrink over the next 50 years based on how much pipe the City plans to replace. In the industry, it is accepted that most breaks are concentrated in a relatively small portion of the system (i.e. failed pipes). This is supported by data from a variety of utilities.

For example, Figure 6-22 shows the likelihood that a pipe will break again and the duration until the next break as a function of historic break count for an Example Utility with approximately 28,000 historic documented breaks. The blue points (associated to the primary y-axis) in Figure 6-22 indicate the proportion of the pipe that broke again (associated to the primary y-axis). For example:

- 38-percent of pipes that have experienced one break have broken a second time;
- 56-percent of pipes that have experienced two breaks have broken a third time;
- 69-percent of pipes that have experienced three breaks have broken a fourth time; and
- 95-percent of pipes that have experienced 19 breaks have already experienced a 20th break.

The data trend is best described by a logarithmic equation (blue line) that has a strong coefficient of determination ($R^2 = 0.9295$). This data indicates that as a pipe experiences more breaks, it is more likely to experience another break.
The red points (associated to the secondary y-axis) in Figure 6-22 summarize the average duration between subsequent breaks. For example, for pipes that have at least:

- **Two breaks**, the average duration between the first and second break is **4.6 years**;
- **Three breaks**, the average duration between the second and third break is **3.0 years**;
- **Four breaks**, the average duration between the third and fourth break is **2.2 years**; and
- **Twenty breaks**, the average duration between the 19th and 20th break is **0.38 years**

The data trend is best described by the red line which has a strong coefficient of determination ($R^2 = 0.9784$). This data indicates that as a pipe experiences more breaks, the duration until the next break becomes shorter. Both trends support the theory that historic break count is a good indicator of future performance of a pipe.

![Figure 6-22. Likelihood of Multiple Breaks and Duration between Breaks (Example Utility)](image)

If a utility controls the increase of failed pipes, running 5 year average break rates should be level (which account for year to year fluctuations due to weather and other factors). If the backlog decreases, break rates should decrease as well.

**6.6.3 Results of 50-Year Renewal Projection**

The backlog of failed pipe will grow or shrink over the next 50 years based on how much failed pipe the City replaces. Five preliminary investment scenarios (pipe replacement) are summarized below. Each scenario includes an investment profile (i.e. how many miles of pipe will be replaced each year over the next 50 years) and the impact of that
work in addressing failed pipe backlog growth. Scenarios are generally ordered from lowest investment level to highest investment level.

Scenario 1, as shown in Figure 6-23, sustains the current replacement rate of 1.5 miles per year, as provided by City staff during the September 2016 workshops held by HDR. This replacement rate equates to a financial investment of $575,000 per year (assuming approximately $75 per foot of water main replaced assuming material only). This scenario will likely cause breaks to increase over time and the annual replacement rate will decrease from the current 0.26-percent per year to 0.21-percent in 2035 (assuming approximately 7 miles of pipe are added to the system each year).

Figure 6-23. Forecast Scenario 1: Sustain Current Investment

Scenarios 2 through 4 will result in a fairly steady number of annual breaks. The investments for these scenarios are as follows (all dollar values are based on an average of $75/ft assuming materials only):

- Scenario 2 (Figure 6-24): Annual replacements increase from the current 1.5 mi/yr to 5.6 mi/yr in 2058 (which levels out to 2065). The annual replacement increases from the current 0.26-percent to 0.53-percent in 2035.
  - 2016 to 2020 CIP investment: $3.8 million
  - 2021 to 2025 CIP investment: $4.9 million
Scenario 3 (Figure 6-25): Annual replacements increase by 1 mile every 5 years beginning in 2018 to a maximum of 5.5 mi/yr in 2032. The annual replacement increases from the current 0.26-percent to 0.77-percent in 2035.
  - 2016 to 2020 CIP investment: $4.2 million
  - 2021 to 2025 CIP investment: $6.1 million

Scenario 4 (Figure 6-26): Annual replacements increase by 5-percent per year to a maximum of 6 miles in 2045. The annual replacement increases from the current 0.26-percent to 0.53-percent in 2035.
  - 2016 to 2020 CIP investment: $3.3 million
  - 2021 to 2025 CIP investment: $4.2 million
Figure 6-25. Forecast Scenario 3: Increase Replacement Every 5 Years by 1 Mile

Figure 6-26. Forecast Scenario 4: Increase Replacement 5-percent per Year to 6 Miles
Scenario 5 increases the main replacements by 250,000 feet each year (see Figure 6-27). The annual replacement increases from the current 0.26-percent to 0.74-percent in 2035. It will result in annual break reductions, although staff will not likely see those reductions for about 10 years when the investment level reaches a point to impact the backlog. Under this scenario the CIP investments will be:

- 2016 to 2020 CIP investment: $4.0 million
- 2021 to 2025 CIP investment: $6.5 million

It is anticipated that additional funding scenarios, as defined by City staff, will be run to find the appropriate balance between level of service and the cost of service. Investment scenarios incorporate investment size, timing, and whether the City desires to “ramp up” (e.g. increase replacement by 5-percent per year every year) or “step up” (e.g. when new resources in-house or contracted come online, output jumps then stays steady until the next resource change).

![Figure 6-27. Forecast Scenario 5: Increase Replacement by a Quarter Mile per Year](image)

6.7 Project Prioritization

Once the optimal annual investment is determined, projects can begin to be prioritized. After analyzing the City’s system data, HDR developed a relative risk scoring methodology. The intent of the relative risk score is to define a consistent, transparent, and defensible approach for prioritizing water main replacement projects. The relative risk score is not intended to replace the need for planning staff to evaluate the extents and/or priorities of particular renewal projects; rather, it is meant to focus these resources
by triaging potential replacement projects by relative risk. The relative risk model should be updated regularly to account for new data such as break history and crew observations. As the Program continues to mature, it is anticipated that the relative risk methodology will adapt to changing drivers, experiences, and readily available information.

### 6.7.1 Project Prioritization Factors

This methodology estimates a relative risk score based on assessment of the consequence of failure (CoF) and the likelihood of failure (LoF). The CoF is primarily a desktop analysis that focuses on the impact the failure will have on the service provided by the system or the risk for financial expenditures the City will incur due to the failure. The following CoF criteria are considered in the risk assessment:

- Level of service
- Damage
- Fire flow required
- Impact to system operation

Likelihood of failure is a data-driven assessment that focuses on the relative probability of failure of the asset. The following LoF criteria are considered in the risk assessment:

- Break count
- Annual break rate (breaks per 100 miles)
- Date of last break
- Pipe diameter
- Corrosion potential
- Shrink swell potential
- Pipe age
- Material vintage (combines the age of the pipe with material)
- Crew observation of condition

All criteria are given a ranking on a scale of one (1) to five (5), with one (1) being the most critical consequence. Additionally, each criterion is given a weighting indicating the relative importance of the criterion. The following describes the basis for the selection of the above criteria and the process of determining the relative risk score.

#### Level of Service

The level of service attribute prioritizes pipes according to the types of facilities served or that are likely to be impacted by the outage; this criterion is similar to the City’s rating structure for service criticality. This factor is assumed to require the greatest weight (30-percent) because it captures the consequence of failure on the community as a whole. For example, critical facilities such as medical care facilities, government operations, schools, and critical industries will receive a rating of one (1) because an outage
affecting the ability of these types of services to continue operations will have a negative
effect on the community. Land uses that affect a low number of people such as park
space receive a higher rating (e.g., closer to five [5]).

At this time this attribute is generally based on land use. It will be advantageous, as the
data becomes available, to quantify the number of service taps on a pipe segment,
thereby, the number of people potentially impacted by a break.

Ratings were assigned for each pipe based on the City’s future land use layer. Future
land use was selected for its simplicity compared to current zoning; it is representative of
the land use of existing developed areas and also represents commercial, industrial, and
government facilities in a way that is more straightforward.

**Damages**

The damages attribute prioritizes pipes according to the magnitude of damage and
resulting financial expenditure that can result from a failure. This factor is assumed to
require the greatest weight (30-percent), similar to level of service, because it captures
the consequence of a failure on the financial burden to the City, and subsequently the
rate payers.

Most pipelines are installed within public Right of Ways (ROWs) where a water main
break has an impact on the surrounding infrastructure. For example, a water main break
under a road can cause the buckling of pavement or sinkholes, in addition to requiring
the street to be dug up and replaced in order to repair the water main. However, paved
areas in ROWs or easements may be addressed differently. The damages attribute is
rated according to the level of importance of the infrastructure affected. For example, a
pipe crossing a State Highway will likely carry the most potential for damages upon
failure, the most expensive repairs, and impacts a larger number of people versus a
residential road.

Ratings were assigned for each pipe based on the STCLASSDSC category of the
Johnson County roads layer. This feature class represents the centerline of a road;
therefore a “buffer” from the centerline of the road was used to identify where a water
main is under a road. The STCLASSDSC category classifies roads within the City of
Olathe as follows:

- Local residential roads (a 15 foot buffer from the road centerline was used);
- Collector roads (a 20 foot buffer from the road centerline was used);
- Thoroughfare roads (a 40 foot buffer from the road centerline was used); and
- Highways (a 75 foot buffer from the road centerline was used).

The railroad layer was also used to identify water mains that cross or parallel a railroad
right-of-way; a 50 foot buffer from the railroad centerline was used.

**Fire Flow Required**

This criterion assesses the impact to the system’s ability to deliver required fire flows
based on the level of fire flow required in vicinity of the pipe, in lieu of the actual fire flow
available based on hydraulic modeling. The reason for this change is that pipes that
contribute to insufficient fire flows have been prioritized as capital projects. It will also
enable staff to assess on an ongoing basis. It is difficult to quantify the impact of a main
break on the amount of fire flow available in a well-looped system without significant effort (main breaks will have to be simulated throughout the system in a hydraulic model). Therefore, this criterion considers that critical areas needing high amounts of fire flow (such as commercial and industrial areas) rated lower on the ranking scale (e.g., closer to one [1]) versus residential areas, which don’t require as much fire flow.

This factor is weighted 20-percent of the consequence of failure category. The failure of a pipe does not have a significant direct impact to fire flow availability because the system is well-looped; the impact to the system will be more in the form of reduced fire flow available, which is difficult to quantify.

Fire flow required is based on the City’s future land use layer and is the same as what was established for hydraulic modeling as presented in Section 5. One-and-two family residential areas have a fire flow requirement of 1,000 gpm, whereas high-density residential, industrial, and commercial areas have a fire flow requirement of 3,500 gpm.

**System Operations**

This criterion assesses the impact to the City’s ability to adequately operate the system and provide service to customers during an outage. For example, main breaks close to the water treatment plant or other storage/pumping stations within the system will have more impact to the system as a whole than a main break in a residential area. This criterion also assumes that larger diameter pipes are likely closer to critical infrastructure within the water distribution system.

This factor is weighted 20-percent of the consequence of failure category. While it seems this criterion should be weighted higher, the City has adequate redundancy within the system to allow continued operation during a main break. For example, the City has three large diameter transmission mains that leave WTP2; if a break occurred on one service will not likely be significantly compromised.

**Break Count**

As shown in Figure 6-22, there is a strong relationship between break count and the duration to the next break within most systems. A pipe that has broken in the past is more likely to break again in the future; multiple breaks on a single line increase that likelihood even more. A weighting of 30-percent was assigned to this factor.

**Annual Break Rate**

This method places additional emphasis on recent breaks and on shorter pipe where a more cost-effective may be realized. A weighting of 30-percent was assigned to this factor.

The annual breaks per 100 miles (i.e., break rate) are calculated for a Construction Project between the first break and April 30, 2016 (i.e. the last date where break data was used for this analysis). For example, if a 528 foot Construction Project XYZ which is 0.1 miles long, first broke in 12/31/2010 and broke again in 2011 and 2012, the break rate would be calculated as:

\[
\text{Break Rate} = \frac{(3 \text{ breaks}) \times 100}{(3.0 \text{ years}) / (0.1 \text{ miles})} = 1,000 \text{ annual breaks per 100 miles}
\]
Date of Last Break

The date of the last break places emphasis on segments that have had a recent break. For example, all other factors being equal, a segment that broke last year will be rated lower than a pipe that has lasted several years since its last break. The date of the last break is assigned a weight of 15-percent.

Pipe Characteristics

Section 6.4 of this Water Master Plan Update determined that five pipe characteristics can be correlated with accelerated deterioration rates: material vintage (the material and era of installation), pipe age, corrosion potential, shrink swell potential, and diameter. Each characteristic has a similar impact on the deterioration rate. Therefore, each factor was weighted at 5-percent for a total value of 25-percent of the LoF score.

Crew Observation

A pipe that has been observed to be in deteriorated condition (and also likely has a low desktop ranking for the other factors) should be prioritized ahead of the pipes that have not been observed in the field and may or may not have the level of deterioration indicated by the desktop ranking. There is not currently a GIS layer containing crew observations. It is recommended that the City develop this layer over time and update the ratings for this attribute as observations are made. If this factor is implemented in the future by City staff, it is recommended that the weighting of break rate, annual break rate, and date of last break be weighted less to accommodate the weighting for crew observations.

6.7.2 Risk Assessment Process

The risk assessment begins with rating each pipe in the system for each of the criteria discussed previously. From there, the LoF Index and the CoF Index are determined. Each are then weighted to determine the Relative Risk Score. The overall process of defining the Relative Risk Score is shown in Figure 6-28 and detailed below.
Consequence of Failure (CoF)

Each pipe should be rated on a scale of 1 to 5 for each of the CoF criteria shown in Table 6-13. The CoF Index is determined by the following equation:

\[
CoF \text{ Index} = \sum (\text{Criteria Rating} \times \text{Weight}) \times 20
\]

An example calculation is as follows:

**Level of service rating = 5**

**Damage rating = 4**

- Fire Flow Required Rating = 4
- Impact to System Operations = 3

\[
CoF \text{ Index} = (5 \times 0.3 + 4 \times 0.3 + 4 \times 0.2 + 3 \times 0.2) \times 20 = 82
\]

On a scale of 1 to 100, with 100 being a pipe with the lowest consequence of failure possible, this pipe has a CoF Index of 82.
Table 6-13. Consequence of Failure (CoF) Criteria Scoring

<table>
<thead>
<tr>
<th>Score</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Level of Service (30%)</strong></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Park, open space</td>
</tr>
<tr>
<td>4</td>
<td>Low-density residential</td>
</tr>
<tr>
<td>3</td>
<td>Medium density residential, business parks</td>
</tr>
<tr>
<td>2</td>
<td>High density residential, commercial, industrial</td>
</tr>
<tr>
<td>1</td>
<td>Critical industries, government facilities, schools, and medical facilities</td>
</tr>
</tbody>
</table>

| **Damages (30%)** | |
| 5 | Outside of roads |
| 4 | Local (residential) roads – within 15’ of centerline |
| 3 | Collector roads – within 20’ of centerline |
| 2 | Thoroughfares – within 40’ of centerline |
| 1 | Highways (interstates, state highways) – within 75’ of centerline |
| | Railroads – within 50’ of centerline |

| **Fire Flow Required (20%)** | |
| 4 | 1,000 gpm (one and two family residential) |
| 2 | 3,500 gpm (high-density residential, commercial, and general industrial) |
| 1 | Special industrial requirements > 3,500 gpm |

| **Impact to System Operations (20%)** | |
| 5 | Hydrant services |
| 4 | Minor water distribution lines (4-in) |
| 3 | Minor water distribution lines (6-8-in) |
| 2 | Major water distribution lines (10-12-in) |
| 1 | Major transmission mains (16-in and larger) |

**Likelihood of Failure (LoF)**

Each pipe is rated on a scale of 1 to 5 for each of the LoF criteria shown in Table 6-14. The LoF Index is determined by the following equation:

\[
\text{LoF Index} = \sum (\text{Criteria Rating} \times \text{Weight}) \times 20
\]

An example calculation is as follows:

- Break Count rating = 2
- Annual Break Rate rating = 1
- Date of Last Break rating = 2
• Pipe Diameter rating = 3
• Material Vintage rating = 4
• Crew Observation rating = 3

\[ \text{LoF Index} = (2x0.2 + 1x0.2 + 2x0.1 + 3x0.1 + 4x0.15 + 3x0.25) \times 20 = 49 \]

On a scale of 1 to 100, with 100 being a pipe with the lowest likelihood of failure possible, this pipe has a LoF Index of 49.

### Table 6-14. Likelihood of Failure Criteria Scoring

<table>
<thead>
<tr>
<th>Score</th>
<th>Description</th>
<th>Score</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>No recorded breaks</td>
<td>5</td>
<td>&gt;18-in</td>
</tr>
<tr>
<td>4</td>
<td>&lt; 3 breaks</td>
<td>4</td>
<td>Not used</td>
</tr>
<tr>
<td>3</td>
<td>3 - 5 breaks</td>
<td>3</td>
<td>8 – 18-in</td>
</tr>
<tr>
<td>2</td>
<td>6 - 10 breaks</td>
<td>2</td>
<td>6-in</td>
</tr>
<tr>
<td>1</td>
<td>&gt; 10 breaks</td>
<td>1</td>
<td>&lt; 6-in</td>
</tr>
<tr>
<td></td>
<td><strong>Break Count (30%)</strong></td>
<td></td>
<td><strong>Pipe Diameter (5%)</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Score</th>
<th>Description</th>
<th>Score</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0</td>
<td>5</td>
<td>PVC, HDPE, Steel</td>
</tr>
<tr>
<td>4</td>
<td>1-40</td>
<td>4</td>
<td>Good Vintage Metallic</td>
</tr>
<tr>
<td>3</td>
<td>41-80</td>
<td>3</td>
<td>Asbestos Cement</td>
</tr>
<tr>
<td>2</td>
<td>81-120</td>
<td>2</td>
<td>–Unknown</td>
</tr>
<tr>
<td>1</td>
<td>120+</td>
<td>1</td>
<td>Poor Vintage Metallic</td>
</tr>
<tr>
<td></td>
<td><strong>Annual Break Rate (30%)</strong></td>
<td></td>
<td><strong>Material Vintage (5%)</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Score</th>
<th>Description</th>
<th>Score</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>10+ years ago or never</td>
<td>5</td>
<td>Less than 20 years old</td>
</tr>
<tr>
<td>4</td>
<td>Within last 6 - 10 years</td>
<td>4</td>
<td>20 – 39 years old</td>
</tr>
<tr>
<td>3</td>
<td>Within last 3 - 5 years</td>
<td>3</td>
<td>Unknown</td>
</tr>
<tr>
<td>2</td>
<td>Within last 2 years</td>
<td>2</td>
<td>40 – 59 years old</td>
</tr>
<tr>
<td>1</td>
<td>Within last year</td>
<td>1</td>
<td>60 + years old</td>
</tr>
<tr>
<td></td>
<td><strong>Date of Last Break (15%)</strong></td>
<td></td>
<td><strong>Age (5%)</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Score</th>
<th>Description</th>
<th>Score</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Lower Potential</td>
<td>4</td>
<td>Lower</td>
</tr>
<tr>
<td>1</td>
<td>Higher Potential</td>
<td>1</td>
<td>Higher</td>
</tr>
</tbody>
</table>

**Corrosion Potential (5%)**

<table>
<thead>
<tr>
<th>Score</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Lower</td>
</tr>
<tr>
<td>1</td>
<td>Higher</td>
</tr>
</tbody>
</table>

**Shrink Swell Potential (5%)**

Note: Crew observation should be considered as a rating factor in the future as the data is collected. It is recommended that it be weighted 20-percent and the weightings for the break rate and break count reduced to 20-percent each. The goal is for data based on real experiences such as break rate, date of last break, break count, and crew observations be weighted approximately 75-percent.
Relative Risk Score

Once the CoF Index and the LoF Index are known, the Relative Risk Score can be determined according to the following equation.

\[
Relative \ Risk \ Score = CoF \ Index \times 0.4 + LoF \ Index \times 0.6
\]

An example calculation is as follows (continuing from the previous examples):

- CoF Index = 82
- LoF Index = 49

\[
Relative \ Risk \ Score = 82 \times 0.4 + 49 \times 0.6 = 62.2
\]

The relative risk score of this pipe is 62.2. This means that other pipes with lower ratings (closer to 0) will be prioritized for replacement ahead of this segment while pipes with higher ratings (closer to 100) will be prioritized for replacement after this segment. In this example, the LoF Index being lower than the CoF index indicates that the pipe is fairly critical but has a moderate condition.

This process is repeated for every pipe in the system. The results are sorted from low to high. Priority projects can then be selected from the top of the list.

Figure 6-29 shows the relative risk scores for each main within the system.
Figure 6-29 Prioritized Water Main Replacement Projects
### 6.8 Strategies for Water Main Renewal

Table 6-15 lists the common water main rehabilitation technologies. Each of these methods is appropriate for the rehabilitation of water mains, depending on the structural condition of the existing pipe, and other considerations. The selection of which system to use generally depends on cost, owner preferences, and other factors. All materials in contact with water should be tested and certified in accordance with ANSI/NSF 61 requirements.

Table 6-15. Common Water Main Rehabilitation Methods

<table>
<thead>
<tr>
<th>Description</th>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement mortar lining, spray-applied, in situ (ANSI/AWWA Standard C602)</td>
<td>• Low cost</td>
<td>• “Non-structural”—not recommended if pipe is structurally deficient</td>
</tr>
<tr>
<td></td>
<td>• Time-tested protection against internal corrosion</td>
<td>• Not recommended where water is soft</td>
</tr>
<tr>
<td></td>
<td>• Service reconnection not required</td>
<td></td>
</tr>
<tr>
<td>Polymer lining, 1 mm thick (epoxy, polyurethane, or polyurea), spray-applied, in-situ (ANSI/AWWA Standard C620)</td>
<td>• Low cost</td>
<td>• “Non-structural”—not recommended if pipe is structurally deficient</td>
</tr>
<tr>
<td></td>
<td>• Time-tested protection against internal corrosion</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Service reconnection not required</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Rapid set-up of some linings may allow same-day return to service (avoiding bypass system costs)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• “Non-structural”—not recommended if pipe is structurally deficient</td>
<td></td>
</tr>
<tr>
<td>Polymer lining, 3 to 8 mm thick (epoxy, polyurethane, or polyurea), spray-applied, in-situ</td>
<td>Moderate cost</td>
<td>Not likely to survive fracturing of the pipe</td>
</tr>
<tr>
<td></td>
<td>“Semi-structural”—proven ability to span holes and gaps.</td>
<td>Ability to serve as fully structural system has not been confirmed</td>
</tr>
<tr>
<td></td>
<td>• Service reconnection not required</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Rapid set-up of some linings may allow same-day return to service (avoiding bypass system costs)</td>
<td></td>
</tr>
<tr>
<td>Cured-in-place pipe lining, reinforced with fiberglass, polyester or carbon fibers</td>
<td>• Fully or semi-structural</td>
<td>More costly than spray-applied linings</td>
</tr>
<tr>
<td></td>
<td>• Appears capable of surviving pipe fracture</td>
<td>Service reconnections are required, but many can be performed by in-pipe robot</td>
</tr>
<tr>
<td></td>
<td>• Robotic service restoration is possible in many cases</td>
<td>Long-term performance of some products not proven</td>
</tr>
<tr>
<td>Tight-fit HDPE slip lining, using roll-down, swage, or deformed methods</td>
<td>• Semi- or fully structural</td>
<td>More costly than spray-applied linings</td>
</tr>
<tr>
<td></td>
<td>• Capable of surviving pipe fracture</td>
<td>Service reconnections are required</td>
</tr>
<tr>
<td></td>
<td>• Design criteria and properties are well established</td>
<td>Limited wall thicknesses available</td>
</tr>
<tr>
<td>Pipe bursting replacement</td>
<td>• Fully structural</td>
<td>More costly than most other methods, although competitive market exists (not proprietary)</td>
</tr>
<tr>
<td></td>
<td>• Some upsizing possible</td>
<td>Service reconnections are required</td>
</tr>
<tr>
<td></td>
<td>• Design criteria and properties are well established</td>
<td>Long-running cracks have occurred with fused PVC, but HDPE is very crack resistant</td>
</tr>
<tr>
<td></td>
<td>• Compared to tight-fit lining, pipe materials should be more easily procured (less critical sizing</td>
<td></td>
</tr>
<tr>
<td></td>
<td>requirements and different materials can be used</td>
<td></td>
</tr>
<tr>
<td>Cathodic Protection Retrofit</td>
<td>• Can economically extend the lives of water mains</td>
<td>Where mains are electrically discontinuous, protection is limited</td>
</tr>
<tr>
<td></td>
<td>• Low-dig methods are available, using vacuum excavation and “keyhole” tools</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Can be used in conjunction with in-pipe NDE to target corroded pipe</td>
<td></td>
</tr>
</tbody>
</table>

8 Testing will soon be conducted at the Trenchless Technology Center of Louisiana Tech University.

9 Per testing performed at the Trenchless Technology Center of Louisiana Tech University.
The rehabilitation techniques presented in Table 6-15 are proven for their effectiveness in water main rehabilitation. Many other techniques are promoted, but not all are effective, efficient, or durable. Method selection depends on many site-specific factors including the structural integrity of the host pipe, the locations and numbers of valves, laterals, connections, future system plans, and the owner’s preferences.

Typically, a pipeline rehabilitation project will concurrently include upgrades or replacements of valves and other appurtenances such as hydrants, meters, and substandard service laterals, particularly those with lead pipe.

Several utilities have developed cost-effective programs using these low-dig methods10:

- **Pipe bursting.** WaterOne, the utility that serves several Kansas City suburbs, decided to try pipe bursting for routine water main replacement, using their own construction crews. The utility hoped that pipe bursting would produce cost saving of about 15 percent, by reducing the amount of repaving that is required. In reality, the cost savings exceeded 25 percent because work proceeded more quickly—more footage was accomplished each day. A similar story, but with more remarkable cost savings has been reported by Western Slope Utilities, the utility that serves Breckenridge, Colorado. Western Slope reports 50 percent cost savings.

- **Cement-mortar lining.** Several utilities in North America, Australia, and Europe have routinely employed cement-mortar lining (CML) to improve water quality and hydraulic performance, while extending the lives of their water mains. LADWP, Sydney Water, and many other large cities have in-fact completed the lining of all unlined cast-iron pipe in their systems, and have experienced reductions in break rates as a result. CML is typically performed at less than half the cost of replacement11.

- **Cathodic protection retrofits.** Each year, the City of Calgary scans a small portion of its system, using the remote-field electromagnetic method and uses the results to determine which mains are best suited for cathodic protection retrofits. Several criteria are used to select mains for scanning, including the corrosivity of the soil, the history of leaks and breaks, and whether a scanning tool can be readily deployed. Through this program, the number of breaks has been cut in half, paying twice over for the cost of the inspection and retrofits. In Des Moines, Iowa, a pilot program demonstrated that a 20-year life extension was achievable at a cost of less than 10 percent of open-trench replacement.

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10 Except as otherwise noted, these case studies are from WaterRF Report 4367, “Answers to Challenging Distribution Infrastructure Questions”, Ellison, et al., 2013.

6.9 Summary and Program Recommendations

6.9.1 Data Collection Improvements

This section describes recommendations associated with break response data collection. Gathering and storing high quality break data is critical for cost effective water main decision making. This may include sizing future budgets, prioritizing direct condition assessment projects, making direct renewal decisions (replace, non-structural lining, structural lining, continue to operate, etc.). At a minimum, each break should contain the location of the break (in GIS), the asset that broke, the date of the break, the cause of the break, and a visual assessment of the pipe exposed.

Update Observation Data Collection. The intent of observation data is to cost-effectively capture data that could support condition assessment and renewal decision-making. Such data would be collected any time an active main is exposed (e.g. main repair, service connection, etc.) to support more cost effective data collection. Data currently collected includes failure cause (e.g. corrosion, contractor, and ground shift), failure type (e.g. split, old repair), asset type (e.g. water main, water valve), comments, and GPS location. This is important data and should continue to be collected. The City should also consider collecting a visual assessment of the exterior condition, interior condition, soil type and/or resistivity, duration of outage, number of properties impacted, repair type, and whether the pipe is polywrapped.

Associate Each Break to the Pipe that Broke. In addition to the location of each break (which the City documents in GIS), it is also important to document the pipe that broke. When a pipe that has broken is replaced, some research and analysis will require access to those breaks (e.g. historic performance of the system). In other research and analysis (e.g. replacement project planning), it is important to be able to efficiently identify and remove breaks that are associated with pipes that have since been replaced. Therefore, it is recommended that the City also document the unique pipe identifier in the break database. Figure 6-30 shows historic break records through April 30, 2016. Blue bars show the count of breaks that were associated to a pipe (using the SDEID) by month while red bars show the count of breaks that were not associated to a pipe. This figure shows that this task has been performed in the past as recently as January of 2015. The City should consider implementing a scheduled procedure to perform this association on a monthly or yearly basis.
6.9.2 Evaluate and Update Design Standards to Reduce Long term Cost of Ownership

The life-cycle performance of DIP is variable, because it depends on the corrosivity of the environment and how well the pipe is protected from corrosion. In some cases, pipes will last hundreds of years. In other cases, the pipe needs replacement after 20 years due to rapid corrosion. At a minimum, DIP should be installed with polyethylene wrapping, for protection against external corrosion, and should be electrically isolated from copper services.

The City’s system is relatively new with an average age of 26 years. However, the system has relatively poor performance with approximately 22 annual breaks per 100 miles. This phenomenon is likely related to the installing metallic pipe with inadequate corrosion protection in an environment with elevated soil corrosivity levels.

By contrast, WaterRF Report 2879, “Long-Term Performance Prediction for PVC Pipes” (Burn, et al., 2005) predicts approximately 10 annual breaks per 100 miles when a pipe is 100 years old. The PVC performance can be improved and long term cost of ownership can be reduced even more by “overdesigning” the pipe (i.e. using Class 200 when Class 150 is required).

Improving design standard will dramatically increase the life of infrastructure and reduce the long term cost of ownership. The City should consider changes to the existing design standards to improve corrosion protection of the metallic pipe or to move to a material that will perform better in this environment (e.g., PVC or HDPE).

Although not currently used by the City, PVC is the most commonly used alternative to DIP in the industry. Installation practices are critical in PVC pipe installation. PVC can be
susceptible to high pressures and cyclic loading. This issue can cost effectively be mitigated by using a large safety factor (i.e. use class 200 where class 100 would suffice). If designed and installed properly, PVC should last much longer and be more cost effective than DIP in the City’s aggressively corrosive soil environment. PVC should not crack during tapping if the tapping is performed correctly. PVC will crack during tapping if the wrong bits are used, the tap occurs on the outside edge of a pipe bend, or the PVC material is bad. When PVC pipe does break, it tends to be more catastrophic.

City staff has indicated that the City is moving towards the use of HDPE. HDPE is another alternative that seldom fails catastrophically and has as long or longer life than PVC. However, HDPE installation requires a different method of construction which can be cost prohibitive in some markets.

6.9.3 Risk Assessment and Project Prioritization

The relative risk model should be updated regularly to account for new data such as breaks. As the Program continues to mature, it is anticipated that the relative risk methodology will adapt to changing drivers, experiences, and readily available information. The current analysis extends through April 30, 2016. It is recommended to annually update the project prioritization spreadsheet with the breaks for the previous year. This will allow the City to reprioritize water main replacements for upcoming years using the most current data available. It is also recommended that City staff regularly review the weighting of the criteria listed in Tables 6-13 and 6-14.

6.9.4 System Renewal Investment

In Section 6.6.3 above, scenarios for the Water Main Replacement Program were identified with an investment profile (i.e. how many miles of pipe will be replaced each year over the next 50 years) and the impact of that work in addressing failed pipe backlog growth. Based on the results of the 50-year renewal projection it is recommended the City proceed with Scenario 3.

Scenario 3: Annual replacements increase by 1 mile every 5 years beginning in 2018 to a maximum of 5.5 miles per year in 2032. The annual replacement increases from the current 0.26-percent to 0.77-percent in 2035 which is above the existing replacement percentage of WaterOne and below the replacement percentage for the City of Lawrence. Refer to Table 6-4 and Figure 6-25.

- 2017 to 2022 CIP investment: $4.2 million
- 2023 to 2027 CIP investment: $6.1 million

6.9.5 Future Inspection Programs and Planning Strategies

Industry experience suggests that pipe age is not a reliable indicator of pipe condition and likelihood for future breakage. Figure 6-31 below shows two metallic pipes that illustrate this concept. On the left is a coupon from a 130 year old cast iron water main showing minimal deterioration. On the right is a 40 year old ductile iron main that was severely corroded and well past its useful life.
Figure 6-31. Age is a poor predictor of deterioration (example)

Corrosion and the overall condition of metallic water mains varies greatly over a short distance. A water main sharing similar properties (material, installation year, diameter, original wall thickness, etc.) may be in good condition at one location and in poor condition just a few feet away. This theory has been validated both by pipe excavation and assessment and by direct condition assessment.

For example, Figure 6-32 shows a 1,300 foot water main with pit depth\(^{12}\) on a red to blue scale where red dots are pits with less than 20-percent remaining wall thickness and light blue dots are pits with 50-80-percent remaining wall thickness. Water mains without dots have no recorded pits. On this 1,300 foot pipe with similar characteristics, there are five distinct water main conditions. Three sections are in poor condition (many deep pits) and two sections are in good condition (no significant pits).

Figure 6-32. In a 1,300 foot pipe, significant deterioration variation can be seen over a short distance.

Figure 6-33 shows the same pipe but now overlays recorded breaks (red stars). As you can see, breaks are concentrated in the three water main sections in the worst condition as measured by density of deep pits.

\(^{12}\) Shown to be the best indicator of past and future breaks based on WRF Project 4471
Currently, most utilities make water main renewal decisions based on desktop analyses which infer condition and future breaks using data such as historic breaks, pipe/soil characteristics, and estimated stresses (pressure, shrink swell, frost heave, etc.). A lack of direct condition knowledge leads to imperfect decision making which includes:

- Discarding good pipe;
- Unnecessary breaks (not replacing the bad pipe soon enough);
- Selecting renewal technologies that are not the most cost effective because the condition is not known; and
- Improper renewal program sizing.

Currently, the Water Research Foundation and HDR are evaluating the cost effectiveness of direct condition assessment for small diameter metallic water main decision making. This includes an:

- Assessment of approximately 70 miles of direct condition assessment data;
- Excavating several miles of pipe to verify condition assessment accuracy;
- Correlating older inspection data the breaks that have occurred since the inspections; and
- Evaluation of the costs and benefits of implementing a proactive condition assessment program.

**Recommendation**

Based on the above evaluation, it has been determined that current direct condition assessment technology for metallic pipes is cost effective. Therefore, it is recommended that the City develop, pilot, and implement a proactive direct condition assessment program to ensure ratepayers realize the greatest return on their investment.
7 Capital Improvements Plan

The purpose of the Water Master Plan (WMP) Update was to evaluate the City’s existing system and determined the feasibility and requirements for future development of the City’s water supply and distribution system. This Section summarizes the recommended Capital Improvements Plan Projects based on the evaluation which includes the raw water supply and transmission, the water distribution system, and the water main replacement program.

7.1 Raw Water Supply CIP

Section 4 of this WMP Update included the evaluation of the City’s existing raw water supply and transmission and the impacts to that existing system infrastructure based on the projected future demands. This evaluation led to the determination of the raw water supply deficit through 2055. The recommended Capital Projects to meet this raw water supply deficit are presented in Table 7-1 below.

Table 7-1. Raw Water Supply Capital Improvements Plan

<table>
<thead>
<tr>
<th>Capital Project</th>
<th>Project Duration</th>
<th>Cost ¹</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initiates</td>
<td>Completes</td>
</tr>
<tr>
<td>Maintenance of Existing Wells</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cleaning VWs ²</td>
<td>2017</td>
<td>-</td>
</tr>
<tr>
<td>Cleaning HCWs</td>
<td>2017</td>
<td>-</td>
</tr>
<tr>
<td>Replacement of VWs</td>
<td>2018</td>
<td>2020</td>
</tr>
<tr>
<td>River Crossing Transmission Pipeline ⁴</td>
<td>2028</td>
<td>2030</td>
</tr>
<tr>
<td>Construction of HCW 5</td>
<td>2028</td>
<td>2030</td>
</tr>
<tr>
<td>Construction of HCW 6</td>
<td>2055+</td>
<td>2042</td>
</tr>
<tr>
<td>Construction of HCW 7</td>
<td>2055+</td>
<td>2055</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Cost estimate does not include City of Olathe contingencies.

¹ Total costs presented in 2017 dollars.
² Total cost for cleaning all VWs ($100,000 per year for two years).
³ Total cost for cleaning all HCWs ($220,000 per year for four years).
⁴ Construction planned to begin in 2019 on HCW 5 and transmission pipeline. Project is included in the City’s 2016 to 2020 CIP.
⁵ Assumes Alternative 2 36-in carrier pipe with 48-in tunnel.

Additional Capital Projects that are already included the City’s current 2016 to 2020 CIP that have been addressed in the Water Treatment Plant 2 Rehabilitation Report by Black & Veatch and referenced in this Water Master Plan Update include:

- WTP2: Electrical/Backup Power – Chemical Feed and Electrical Modifications: recommends generators be installed at HCW 3 and 4.
- Remote Facilities Improvements: recommends installation of lightning protection at the HCW and VW field scheduled for 2020 and beyond.
It is recommended the City complete the following activities in addition to the Capital Projects detailed above to ensure the City can provide additional supply to meet the current raw water supply deficit:

- Perform detailed geotechnical and geophysical evaluation of the soil strata as part of the HCW 5 Transmission Pipeline Design including a review completed by large-diameter HDD and tunneling contractors.
- Confirm lateral centerline elevations and minimum pumping level elevations currently set in SCADA to ensure the City is utilizing the HCWs most efficiently.
- Install redundant power feed loop for the VW field.
- Continue perfection and certification of the City’s existing wells as detailed in Section 4.1.3.

7.2 Water Distribution System CIP

A hydraulic model of the City’s water distribution system was developed to assess the performance of the existing system and the future system as demands increase through ultimate build-out (2055). The recommended CIP Projects to address pressure, fire flow, and water age issues throughout the distribution system are presented in Table 7-2 below.

Table 7-2. Water Distribution System Capital Improvements Plan

<table>
<thead>
<tr>
<th>Capital Project</th>
<th>Cost ¹</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>5-year CIP (2017 to 2022)</strong></td>
<td></td>
</tr>
<tr>
<td>New Century Pressure Sustaining Valve Replacement</td>
<td>$10,000</td>
</tr>
<tr>
<td>Blackbob 2 Bypass Improvements</td>
<td>$168,000</td>
</tr>
<tr>
<td>Lone Elm Main and Booster Station</td>
<td>$15,266,000</td>
</tr>
<tr>
<td>Phase One Hedge Lane Transmission Main – Extension to Curtis Street</td>
<td>$11,460,000</td>
</tr>
<tr>
<td><strong>Future Operational Enhancements (2035)</strong></td>
<td></td>
</tr>
<tr>
<td>Conversion to Single Pressure Zone</td>
<td></td>
</tr>
<tr>
<td>Phase Two Hedge Lane Transmission Main – Extension to Blackbob</td>
<td>$10,540,000</td>
</tr>
<tr>
<td>Blackbob Piping Improvements</td>
<td>$315,000</td>
</tr>
<tr>
<td>Elevated Storage ²</td>
<td></td>
</tr>
<tr>
<td>Option 1 – Single Pressure Zone (1-MG storage tank required)</td>
<td>$2,750,000</td>
</tr>
<tr>
<td>Option 2 – Two Pressure Zones (2, 1-MG storage tanks required)</td>
<td>$4,400,000</td>
</tr>
<tr>
<td><strong>Total Capital Cost</strong></td>
<td><strong>$44,909,000</strong></td>
</tr>
</tbody>
</table>

¹ Presented in 2017 dollars.
² Cost developed by City Staff and includes City contingencies. All other cost estimates presented in the table do not include City of Olathe contingencies.
³ Old Hwy 56 Transmission Main may be constructed in lieu of Lone Elm Project depending on which southwest service area develops first.
⁴ Location of the tank(s) to be determined at the time project is required.
It is recommended the City complete the following activities in addition of the Capital Projects detailed above:

- Regularly monitor AMI data for large wholesale customers to determine any potential adverse impacts that may be occurring in the system.
- Perform an investigation of dead end mains to determine if flushing, upsizing, or looping is required.

### 7.3 Water Main Replacement Program CIP

In Section 6.6.3 above, scenarios for the Water Main Replacement Program were identified with an investment profile (i.e. how many miles of pipe will be replaced each year over the next 50 years) and the impact of that work in addressing failed pipe backlog growth. Based on the results of the 50-year renewal projection it is recommended the City proceed with Scenario 3.

Scenario 3 (Figure 7-1): Annual replacements increase by 1 mile every 5 years beginning in 2018 to a maximum of 5.5 miles per year in 2032. The annual replacement increases from the current 0.26-percent to 0.77-percent in 2035.

- 2017 to 2022 CIP investment: $4.2 million
- 2023 to 2027 CIP investment: $6.1 million

![Figure 7-1. Forecast Scenario 3: Increase Replacement Every 5 Years by 1 Mile](image)
7.4 Water Master Plan Update CIP

The final recommended Water Master Plan Update CIP Project list is presented for each CIP period in Table 7-3 below.

Table 7-3. Water Master Plan Update Capital Improvements Plan

<table>
<thead>
<tr>
<th>Capital Project</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>5-year CIP (2017 to 2022)</strong></td>
<td></td>
</tr>
<tr>
<td>Maintenance of Existing VWs</td>
<td>$200,000</td>
</tr>
<tr>
<td>Maintenance of Existing HCWs</td>
<td>$880,000</td>
</tr>
<tr>
<td>Replacement of 9 VWs</td>
<td>$9,650,000</td>
</tr>
<tr>
<td>New Century Pressure Sustaining Valve Replacement</td>
<td>$10,000</td>
</tr>
<tr>
<td>Blackbob 2 Bypass Improvements</td>
<td>$168,000</td>
</tr>
<tr>
<td>Lone Elm Main and Booster Station</td>
<td>$15,266,000</td>
</tr>
<tr>
<td>Phase One Hedge Lane Transmission Main – Extension to Curtis Street</td>
<td>$11,460,000</td>
</tr>
<tr>
<td>Annual Water Main Replacement Program</td>
<td>$4,200,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td>$41,834,000</td>
</tr>
<tr>
<td><strong>10-year CIP (2023 to 2027)</strong></td>
<td></td>
</tr>
<tr>
<td>River Crossing Transmission Pipeline</td>
<td>$21,339,000</td>
</tr>
<tr>
<td>Construction of HCW 5</td>
<td>$9,784,000</td>
</tr>
<tr>
<td>Annual Water Main Replacement Program</td>
<td>$6,100,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td>$37,223,000</td>
</tr>
<tr>
<td><strong>20-year CIP (2027 to 2037)</strong></td>
<td></td>
</tr>
<tr>
<td>Conversion of Distribution System to Single Pressure Zone</td>
<td></td>
</tr>
<tr>
<td>Phase Two Hedge Lane Transmission Main – Extension to Blackbob</td>
<td>$10,540,000</td>
</tr>
<tr>
<td>Blackbob Piping Improvements</td>
<td>$315,000</td>
</tr>
<tr>
<td>Elevated Storage Single Pressure Zone (1-MG storage tank required)</td>
<td>$2,750,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td>$13,605,000</td>
</tr>
<tr>
<td><strong>30-year CIP (2037 to 2047)</strong></td>
<td></td>
</tr>
<tr>
<td>None</td>
<td>na</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td>na</td>
</tr>
<tr>
<td><strong>40-year CIP (2047 to 2057)</strong></td>
<td></td>
</tr>
<tr>
<td>Construction of HCW 6</td>
<td>$7,482,000</td>
</tr>
<tr>
<td>Construction of HCW 7</td>
<td>$8,652,000</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td>$16,134,000</td>
</tr>
<tr>
<td><strong>Total CIP Project Cost</strong></td>
<td>$108,796,000</td>
</tr>
</tbody>
</table>
Appendix A
Definitions
Definitions

A-distance to recharge – a measure of distance to a source of recharge where the drawdown from the well is almost zero
Alluvium/alluvial aquifer – the water-bearing sand and gravel materials adjacent to and beneath the river
Aquifer transmissivity (gpd/ft) – indicates the capacity of the aquifer to transfer water through its entire thickness
Drawdown – the difference between the static water level and the pumping water level
Hydraulic conductivity (gpd/ft²) – describes the ease with which a fluid can move through the pore spaces of the alluvium
Pumping water level – the depth or elevation of the water table when a well is pumping, measured inside the well screen or caisson
Recharge – the ability for the water table to increase due to precipitation or the nearby presence of a body of water
Saturated thickness – the thickness of the water in sandy or gravelly, water bearing layers of the aquifer
Static water levels – the natural level of the water table without influences of well pumping
Well caisson – the center concrete structure of a horizontal collector well
Well lateral – the horizontal well screen of a horizontal collector well that extends out from the caisson like spokes of a wheel.
Well screen – perforated pipe of the well structure that holds back the aquifer materials but allows water to reach the pump.
Appendix B
Water Rights Information Reports
Vertical Wellfield
WATER RIGHT INFORMATION REPORT FOR: 10042 00

RIGHT TYPE: Appropriation
SOURCE: Groundwater USE: MUN
CURRENT STATUS: Certificate Issued
PRIORITY DATE: 18-MAY-64
CURRENT COMPLETE BY DATE: 31-DEC-85
COMPLETION ACKNOWLEDGED DATE:
CURRENT PERFECT BY DATE: 31-DEC-90
YEAR PERFECTED:
CERTIFICATE ISSUED DATE: 05-NOV-99

APPLICANT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:
18-MAY-64 Pending Initial Review
06-JAN-65 Approved Pending Completion N+P BY 31-DEC-68 PERF BY 31-DEC-68
21-OCT-74 Change Application For Place Of Use
03-JUN-75 Change Approved
03-NOV-78 F & O (12 Wells)
06-NOV-78 Extended Time To Complete N+P BY 31-DEC-85 PERF BY 31-DEC-85
28-APR-86 F & O - P/D
23-MAY-86 Completed - Extended Time To Perfect PERF BY 31-DEC-90
16-SEP-86 Assurance District Eligibility Notification
17-JUL-90 Change Application For Point Of Diversion
17-JUL-90 Change Approved
17-JUL-90 Install Meter Prior To Actual Use Of Water
17-JUL-90 N&P By12/31/91 Chg Approval
10-JAN-92 N&P 12/91 (Chg)
10-JAN-92 Meter Installed
27-MAR-92 Change Application For Point Of Diversion
27-MAR-92 Change Approved
27-MAR-92 N&P By12/31/92 Chg Approval
28-MAY-97 Change Application For Place Of Use
23-JUN-97 Change Approved
18-AUG-99 Inspected Pending Perfection
18-AUG-99 Proposed Certificate
16-SEP-99 Suspended For Draft Certificate
28-OCT-99 Certificate Ready For Chief Engineer Signature
05-NOV-99 Certificate Issued
05-NOV-99 Record Yr 1990: See Jkt For Gpm'S & Id'S
28-NOV-99 Conservation Plan By Consolidated Rwd #6 Johnson Co
28-NOV-99 Microfilmed Under Jo 4
CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:

SPECIAL CONDITIONS:

QUANTITIES:

RATES BY POINT OF DIVERSION:

LIMITATIONS:

STORAGE QUANTITIES:

STORAGE RATES:

AUTHORIZED POINTS(S) OF DIVERSION

QUALIFIERS: NE SW SW

DIST. FROM SE CORNER: 760 ft North 4385 ft West

NUMBER OF WELLS: 1

COMMENT: WELL #7
OLD LONGITUDE: 94.924024  OLD LATITUDE: 38.985343
NEW LONGITUDE: 94.924088  NEW LATITUDE: 38.987439
GPS LONGITUDE: 94.923760  GPS LATITUDE: 38.987630
GPS FEET NORTH: 830  GPS FEET WEST: 4292
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:
BASEIN: KANSAS RIVER
STREAM:
SPECIAL_USE_AREA(S):
AQUIFER(S):
> MAIN STEM ALLUVIUM
TEST INFORMATION:
> 17-JUL-87 1004 gpm Field Inspection Test
METER ACTION TRAIL:
METER INFORMATION:
Date Installed : 13-NOV-15  Currently Installed? Y
> Manufacturer: FOXBORO  Model: 9108ASICABHJGN
> Type: Electromagnetic  Serial No.: 661401D464
> Meter Unit: Gallons  Meter Size: 5 Inch  Multiplier: 000
> Portable Pump Installation? N  Multiple PDs? N  Straightening Vanes? N
> Measuring Chamber? Y  Meter Comment:
METER INFORMATION:
Date Installed : 13-NOV-15  Currently Installed? N
> Manufacturer: WATER SPECIALTIES  Model: ML04-08
> Type: Propellor  Serial No.: 892390-08
> Meter Unit: Gallons  Meter Size: 5 Inch  Multiplier: 000
> Portable Pump Installation? N  Multiple PDs? Y  Straightening Vanes? Y
> Measuring Chamber? Y  Meter Comment:
OVERLAPS:
-----------
Section 24, T 12, R 22E, ID 5 (Internal PDIV_ID = 534)
QUALIFIERS: NW SW SW
DIST. FROM SE CORNER: 1150 ft North 4670 ft West
NUMBER OF WELLS: 1
COMMENT: WELL #9
OLD LONGITUDE: 94.926329  OLD LATITUDE: 38.987154
NEW LONGITUDE: 94.925091  NEW LATITUDE: 38.988509
GPS LONGITUDE: 94.924790  GPS LATITUDE: 38.988690
GPS FEET NORTH: 1216  GPS FEET WEST: 4585
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:
BASEIN: KANSAS RIVER
STREAM:
SPECIAL_USE_AREA(S):
AQUIFER(S):
> MAIN STEM ALLUVIUM
TEST INFORMATION:
> 17-JUL-88 830 gpm Field Inspection Test
METER ACTION TRAIL:
METER INFORMATION:
Date Installed : 01-JAN-01  Currently Installed? N
> Manufacturer: WATER SPECIALTIES  Model: ML04-06
> Type: Propellor  Serial No.: 200149-6
> Meter Unit: Gallons  Meter Size: 3 Inch  Multiplier: 000
> Portable Pump Installation?  Multiple PDs?  Straightening Vanes? Y
> Measuring Chamber? Y  Meter Comment:

METER INFORMATION:
Date Installed : 10-NOV-15  Currently Installed? Y
> Manufacturer: FOXBORO  Model: 9106ASICABHJGN
> Type: Electromagnetic  Serial No.: 660701D464
> Meter Unit: Gallons  Meter Size: 3 Inch  Multiplier: 000
> Portable Pump Installation? N  Multiple PDs? N  Straightening Vanes? N
> Measuring Chamber? Y  Meter Comment:

OVERLAPS:
-----------------
Section 24, T 12, R 22E, ID 6 (Internal PDIV_ID = 30132)
QUALIFIERS: SW SW SW
DIST. FROM SE CORNER: 21 ft North 4880 ft West
NUMBER OF WELLS: 1
COMMENT: WELL #11
OLD LONGITUDE: 94.924031  OLD LATITUDE: 38.987148
NEW LONGITUDE: 94.925829  NEW LATITUDE: 38.985410
GPS LONGITUDE: 94.925540  GPS LATITUDE: 38.985360
GPS FEET NORTH: 5276  GPS FEET WEST: 4786
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE

GMD:

BASIN: KANSAS RIVER
STREAM:
SPECIAL_USE_AREA(S):
> MAIN STEM ALLUVIUM

TEST INFORMATION:
> 17-JUL-87  797 gpm  Field Inspection Test

METER ACTION TRAIL:

METER INFORMATION:
Date Installed : 01-JAN-00  Currently Installed? N
> Manufacturer: WATER SPECIALTIES  Model: ML04-6
> Type: Propellor  Serial No.: 900150-06
> Meter Unit: Gallons  Meter Size: 3 Inch  Multiplier: 000
> Portable Pump Installation?  Multiple PDs?  Straightening Vanes? Y
> Measuring Chamber? Y  Meter Comment:

METER INFORMATION:
Date Installed : 10-NOV-15  Currently Installed? Y
> Manufacturer: FOXBORO  Model: 9106ASICABHJGN
> Type: Electromagnetic  Serial No.: 723501D494
> Meter Unit: Gallons  Meter Size: 3 Inch  Multiplier: 000
> Portable Pump Installation? N  Multiple PDs? N  Straightening Vanes? N
> Measuring Chamber? Y  Meter Comment:

OVERLAPS:
-----------------
Section 24, T 12, R 22E, ID 13 (Internal PDIV_ID = 25821)
QUALIFIERS: SW SW SW
DIST. FROM SE CORNER: 309 ft North 5215 ft West
NUMBER OF WELLS: 1
COMMENT: WELL #3
OLD LONGITUDE: 94.927255 OLD LATITUDE: 38.985270
NEW LONGITUDE: 94.927008 NEW LATITUDE: 38.986200
GPS LONGITUDE: 94.926690 GPS LATITUDE: 38.986260
GPS FEET NORTH: 331 GPS FEET WEST: 5125
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:
BASIN: KANSAS RIVER
STREAM:
SPECIAL_USE_AREA(S):
AQUIFER(S):
> MAIN STEM ALLUVIUM
TEST INFORMATION:
> 18-AUG-99 431 gpm Field Inspection Test
METER ACTION TRAIL:
METER INFORMATION:
Date Installed : 01-JAN-03 Currently Installed? N
> Manufacturer: WATER SPECIALTIES Model: ML04-06-V
> Type: Propellor Serial No.: 20032260-6
> Meter Unit: Gallons Meter Size: 3 Inch Multiplier: 000
> Portable Pump Installation? Multiple PDs? Straightening Vanes? Y
> Measuring Chamber? Y Meter Comment:
METER INFORMATION:
Date Installed : 10-NOV-15 Currently Installed? Y
> Manufacturer: FOXBORO Model: 9106ASICABHJGN
> Type: Electromagnetic Serial No.: 898401D055
> Meter Unit: Gallons Meter Size: 3 Inch Multiplier: 000
> Portable Pump Installation? N Multiple PDs? N Straightening Vanes? N
> Measuring Chamber? Y Meter Comment:
OVERLAPS:
----------------
Section 24, T 12, R 22E, ID 14 (Internal PDIV_ID = 32794)
QUALIFIERS: SW SW SW
DIST. FROM SE CORNER: 417 ft North 4705 ft West
NUMBER OF WELLS: 1
COMMENT: 38' N & 75' E OF ORIGINAL POINT OF DIVERSION - WELL #4
OLD LONGITUDE: 94.925482 OLD LATITUDE: 38.985566
NEW LONGITUDE: 94.925214 NEW LATITUDE: 38.986502
GPS LONGITUDE: 94.924730 GPS LATITUDE: 38.986500
GPS FEET NORTH: 418 GPS FEET WEST: 4568
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:
BASIN: KANSAS RIVER
STREAM:
SPECIAL_USE_AREA(S):
AQUIFER(S):
> MAIN STEM ALLUVIUM
TEST INFORMATION:
> 18-AUG-99       722 gpm  Field Inspection Test

METER ACTION TRAIL:

METER INFORMATION:
Date Installed : 01-JAN-03   Currently Installed? N
> Manufacturer: WATER SPECIALTIES       Model: ML04-06
> Type: Propellor                    Serial No.: 20031173-6
> Meter Unit: Gallons   Meter Size: 3 Inch   Multiplier: 000
> Portable Pump Installation?    Multiple PDs?    Straightening Vanes? Y
> Measuring Chamber? Y  Meter Comment:

METER INFORMATION:
Date Installed : 10-NOV-15   Currently Installed? Y
> Manufacturer: FOXBORO                   Model: 9106ASICABHJGN
> Type: Electromagnetic              Serial No.: 723401D494
> Meter Unit: Gallons   Meter Size: 3 Inch   Multiplier: 000
> Portable Pump Installation? N   Multiple PDs? N   Straightening Vanes? N
> Measuring Chamber? Y  Meter Comment:

OVERLAPS:
----------------
Section 25, T 12, R 22E,  ID    3 (Internal PDIV_ID = 17326)
QUALIFIERS:  NW NW NW
DIST. FROM SE CORNER:    5020 ft North     5180 ft West
NUMBER OF WELLS:   1
COMMENT: WELL #5
OLD LONGITUDE:  94.926317   OLD LATITUDE:  38.983544
NEW LONGITUDE:  94.926927   NEW LATITUDE:  38.984654
GPS LONGITUDE:  94.926630   GPS LATITUDE:  38.984610
GPS FEET NORTH:    5003   GPS FEET WEST:    5096
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD :
BASIN: KANSAS RIVER
STREAM:
SPECIAL_USE_AREA(S):
AQUIFER(S):
> MAIN STEM ALLUVIUM
TEST INFORMATION:
> 17-JUL-87      1041 gpm  Field Inspection Test

METER ACTION TRAIL:

METER INFORMATION:
Date Installed : 01-JAN-03   Currently Installed? N
> Manufacturer: WATER SPECIALTIES       Model: ML04-08
> Type: Propellor                    Serial No.: 20033890-8
> Meter Unit: Gallons   Meter Size: 5 Inch   Multiplier: 000
> Portable Pump Installation?    Multiple PDs?    Straightening Vanes? Y
> Measuring Chamber? Y  Meter Comment:

METER INFORMATION:
Date Installed : 13-NOV-15   Currently Installed? Y
> Manufacturer: FOXBORO                   Model: 9108ASICABHJGN
> Type: Electromagnetic              Serial No.: 661301D464
> Meter Unit: Gallons   Meter Size: 5 Inch   Multiplier: 000
> Portable Pump Installation? N   Multiple PDs? N   Straightening Vanes? N
OVERLAPS:

Section 25, T 12, R 22E, ID 4 (Internal PDIV_ID = 127)

QUALIFIERS: NE NW NW

DIST. FROM SE CORNER: 5140 ft North 4415 ft West

NUMBER OF WELLS: 1

COMMENT: WELL #6

OLD LONGITUDE: 94.924018 OLD LATITUDE: 38.983538
NEW LONGITUDE: 94.924236 NEW LATITUDE: 38.984984
GPS LONGITUDE: 94.923910 GPS LATITUDE: 38.984920
GPS FEET NORTH: 5116 GPS FEET WEST: 4322

COUNTY: JOHNSON

FIELD OFFICE: TOPEKA FIELD OFFICE

GMD :

BASIN: KANSAS RIVER

STREAM:

SPECIAL_USE_AREA(S):

AQUIFER(S):

> MAIN STEM ALLUVIUM

TEST INFORMATION:

> 17-JUL-87 1032 gpm Field Inspection Test

METER ACTION TRAIL:

METER INFORMATION:

Date Installed : 01-JAN-02 Currently Installed? N

> Manufacturer: WATER SPECIALTIES Model: ML04-08
> Type: Propellor Serial No.: 200210-08
> Meter Unit: Gallons Meter Size: 5 Inch Multiplier: 000
> Portable Pump Installation? Multiple PDs? Straightening Vanes? Y

> Measuring Chamber? Y Meter Comment:

METER INFORMATION:

Date Installed : 13-NOV-15 Currently Installed? Y

> Manufacturer: FOXBORO Model: 9108ASICABHJGN
> Type: Electromagnetic Serial No.: 36001D344
> Meter Unit: Gallons Meter Size: 5 Inch Multiplier: 000
> Portable Pump Installation? N Multiple PDs? N Straightening Vanes? N

> Measuring Chamber? Y Meter Comment:

OVERLAPS:

Section 25, T 12, R 22E, ID 5 (Internal PDIV_ID = 33731)

QUALIFIERS: NC E2 NW NW

DIST. FROM SE CORNER: 4610 ft North 4325 ft West

NUMBER OF WELLS: 1

COMMENT: WELL #10

OLD LONGITUDE: 94.924018 OLD LATITUDE: 38.983538
NEW LONGITUDE: 94.923920 NEW LATITUDE: 38.983528
GPS LONGITUDE: 94.923910 GPS LATITUDE: 38.983440
GPS FEET NORTH: 4577 GPS FEET WEST: 4248

COUNTY: JOHNSON

FIELD OFFICE: TOPEKA FIELD OFFICE

GMD :

BASIN: KANSAS RIVER
STREAM:

SPECIAL_USE_AREA(S): 
AQUIFER(S):
> MAIN STEM ALLUVIUM

TEST INFORMATION:
> 17-JUL-87  767 gpm  Field Inspection Test

METER ACTION TRAIL:

METER INFORMATION:

Date Installed : 01-JAN-98  Currently Installed? N
> Manufacturer: WATER SPECIALTIES  Model: ML04-06
> Type: Propellor  Serial No.: 942045-6
> Meter Unit: Gallons  Meter Size: 3 Inch  Multiplier: 000
> Portable Pump Installation?  Multiple PDs?  Straightening Vanes? Y
> Measuring Chamber? Y  Meter Comment:

METER INFORMATION:

Date Installed : 10-NOV-15  Currently Installed? Y
> Manufacturer: FOXBORO  Model: 9106ASICABHJGN
> Type: Electromagnetic  Serial No.: 661301D464
> Meter Unit: Gallons  Meter Size: 3 Inch  Multiplier: 000
> Portable Pump Installation?  Multiple PDs?  Straightening Vanes? N
> Measuring Chamber? Y  Meter Comment:

OVERLAPS:

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AUTHORIZED PLACE(S) OF USE

Section 10, T 14, R 23E, ID 1 (Internal PUSE_ID = 5051)

OWNER:  CITY OF OLATHE

Address: JOHN GILROY
**  PO BOX 768
**  OLATHE KS 66051

COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY

OVERLAPS:
> A  44613 00 MUN
> A  44614 00 MUN
> A  42542 00 MUN
> A  42541 00 MUN
> A  45994 00 MUN
> A  45993 00 MUN
> A  45648 00 MUN
> A  45649 00 MUN
> A  47202 00 MUN
> A  47203 00 MUN

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Section 10, T 14, R 23E, ID 2 (Internal PUSE_ID = 19416)

OWNER:  CITY OF OLATHE

Address: JOHN GILROY
**  PO BOX 768
**  OLATHE KS 66051

COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED

OVERLAPS:
> A  44613 00 MUN
> A  44614 00 MUN
Section 10, T 14, R 23E, ID 3 (Internal PUSE_ID = 50863)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051

COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07

OVERLAPS:
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> A 44614 00 MUN
> A 42541 00 MUN
> A 42542 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

Section 10, T 14, R 23E, ID 4 (Internal PUSE_ID = 50864)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051

COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1

OVERLAPS:
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> A 44614 00 MUN
> A 42541 00 MUN
> A 42542 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

Section 10, T 14, R 23E, ID 5 (Internal PUSE_ID = 50865)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051

COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E

OVERLAPS:
Section 10, T 14, R 23E, ID 6 (Internal PUSE_ID = 50866)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051

COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE

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Horizontal Collector Well 1
WATER RIGHT INFORMATION REPORT FOR: 42541 00

RIGHT TYPE: Appropriation
SOURCE: Groundwater USE: MUN
CURRENT STATUS: Certificate Issued

PRIORITY DATE: 21-NOV-96
CURRENT COMPLETE BY DATE: 31-DEC-99
COMPLETION ACKNOWLEDGED DATE: 18-OCT-99
CURRENT PERFECT BY DATE: 31-DEC-17

YEAR PERFECTED:
CERTIFICATE ISSUED DATE: 16-NOV-15

APPLICANT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:
21-NOV-96 Pending Initial Review
20-DEC-96 New Application Initial Review Began
23-APR-97 New App Suspended For Comments From Adj Right Holders
16-JUN-97 Pending Chief Engineer Action
23-JUN-97 Approved Pending Completion N+P BY 31-DEC-98 PERF BY 31-DEC-17
23-JUN-97 Assurance District Eligibility
29-JAN-99 Extended Time To Complete N+P BY 31-DEC-99 PERF BY 31-DEC-17
18-OCT-99 Completed Pending Inspection
28-NOV-99 Conservation Plan By Consolidated Rwd #6 Johnson Co
28-NOV-99 Microfilmed Under Jo 4
15-JUL-03 Water Conservation Plan Microfilmed Under Jo4
28-AUG-08 Inspected Pending Perfection
28-AUG-08 Compliance Check/In Comp
05-MAY-09 Compliance Check/In Comp
30-NOV-09 Conservation Plan Scanned Under Jo 4
23-MAY-11 Conservation Plan By Johnson Rwd #7
23-MAY-11 Conservation Plan Scanned Under 10042
10-SEP-15 Proposed Certificate
16-NOV-15 Certificate Issued
16-NOV-15 Record Yr 2000: 10500 Gpm Max & Norm
28-JAN-16 Compliance Check/In Comp

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:
21-NOV-96 Plan Required With Approval Compliance date: 31-DEC-98
28-NOV-99 Approved Conservation Plan
15-JUL-03 Volunteer Plan
23-MAY-11  Approved Conservation Plan
30-NOV-09  Modified Conservation Plan Volunteered
30-NOV-09  Modified Conservation Plan Approved
23-MAY-11  Plan Required By Kansas Water Office
28-NOV-99  Volunteer Plan
15-JUL-03  Approved Conservation Plan

SPECIAL CONDITIONS:

QUANTITIES:
  >AUTHORIZED 219.900 mgd  ADDITIONAL 219.900 mgd

RATES:
  >AUTHORIZED 10500.000 gpm  ADDITIONAL 10500.000 gpm

LIMITATIONS: None

STORAGE QUANTITIES: No active storage quantities associated with MUN use under this water right
STORAGE RATES: No active storage rates associated with MUN use under this water right

AUTHORIZED POINTS(S) OF DIVERSION
Section 27, T 12, R 22E, ID 2 (Internal PDIV_ID = 60024)
QUALIFIERS: SE NE NE
DIST. FROM SE CORNER: 4090 ft North 67 ft West
NUMBER OF WELLS:
COMMENT: WATER STRUCTURES PERMIT #LJO-0126 - RANNEY WELL
OLD LONGITUDE: OLD LATITUDE:
NEW LONGITUDE: 94.946022  NEW LATITUDE: 38.982330
GPS LONGITUDE: 94.946020  GPS LATITUDE: 38.982330
GPS FEET NORTH: 4090  GPS FEET WEST: 67
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:
BASIN: KANSAS RIVER
STREAM: KANSAS RIVER
SPECIAL_USE_AREA(S):
AQUIFER(S):
  > MAIN STEM ALLUVIUM
TEST INFORMATION:
  > 28-AUG-08 23 cfs Field Inspection Test
METER ACTION TRAIL:
METER INFORMATION:
Date Installed: 24-MAY-99  Currently Installed? N
  > Manufacturer: FISHER + PORTER  Model: 10DX3111
  > Type: Not Available  Serial No.: 98W025474
  > Meter Unit: Gallons  Meter Size: N/A  Multiplier: N/A
  > Portable Pump Installation?  Multiple PDs?  Straightening Vanes?
  > Measuring Chamber?  Meter Comment:
METER INFORMATION:
Date Installed: 01-JAN-08  Currently Installed? Y
  > Manufacturer: FOXBORO  Model: 9124A-SIBA-N556
  > Type: Not Available  Serial No.:
> Meter Unit: Gallons Meter Size: N/A Multiplier: N/A
> Portable Pump Installation? Multiple PDs? Straightening Vanes?
> Measuring Chamber? Y Meter Comment: ACTUAL SIZE 24 IN

OVERLAPS:
> A 42542 00 MUN

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AUTHORIZED PLACE(S) OF USE

Section 10, T 14, R 23E, ID 1 (Internal PUSE_ID = 5051)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY
OVERLAPS:
> A 10042 00 MUN
> A 44613 00 MUN
> A 44614 00 MUN
> A 42542 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

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Section 10, T 14, R 23E, ID 2 (Internal PUSE_ID = 19416)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED
OVERLAPS:
> A 10042 00 MUN
> A 44613 00 MUN
> A 44614 00 MUN
> A 42542 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

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Section 10, T 14, R 23E, ID 3 (Internal PUSE_ID = 50863)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07
OVERLAPS:
> A 44613 00 MUN
Section 10, T 14, R 23E, ID 4 (Internal PUSE_ID = 50864)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1
OVERLAPS:
> A 44613 00 MUN
> A 44614 00 MUN
> A 10042 00 MUN
> A 42542 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

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Section 10, T 14, R 23E, ID 5 (Internal PUSE_ID = 50865)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E
OVERLAPS:
> A 44613 00 MUN
> A 44614 00 MUN
> A 10042 00 MUN
> A 42542 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
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> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

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Section 10, T 14, R 23E, ID 6 (Internal PUSE_ID = 50866)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE
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WATER RIGHT INFORMATION REPORT FOR: 42542 00

RIGHT TYPE: Appropriation
SOURCE: Surface Water USE: MUN
CURRENT STATUS: Certificate Issued

PRIORITY DATE: 21-NOV-96
CURRENT COMPLETE BY DATE: 31-DEC-99
COMPLETION ACKNOWLEDGED DATE: 18-OCT-99
CURRENT PERFECT BY DATE: 31-DEC-17

YEAR PERFECTED:
CERTIFICATE ISSUED DATE: 16-NOV-15

APPLICANT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:
21-NOV-96 Pending Initial Review
20-DEC-96 New Application Initial Review Began
23-APR-97 New App Suspended For Comments From Adj Right Holders
16-JUN-97 Pending Chief Engineer Action
23-JUN-97 Approved Pending Completion N+P BY 31-DEC-98 PERF BY 31-DEC-17
23-JUN-97 Assurance District Eligibility
29-JAN-99 Extended Time To Complete N+P BY 31-DEC-99 PERF BY 31-DEC-17
18-OCT-99 Completed Pending Inspection
28-NOV-99 Conservation Plan By Consolidated Rwd #6 Johnson Co
28-NOV-99 Microfilmed Under Jo 4
15-JUL-03 Water Conservation Plan Microfilmed Under Jo4
15-DEC-05 Corr O: Document Error
28-AUG-08 Inspected Pending Perfection
28-AUG-08 Compliance Check/In Comp
05-MAY-09 Compliance Check/In Comp
30-NOV-09 Conservation Plan Scanned Under Jo 4
23-MAY-11 Conservation Plan By Johnson Rwd #7
23-MAY-11 Conservation Plan Scanned Under 10042
10-SEP-15 Proposed Certificate
16-NOV-15 Certificate Issued
16-NOV-15 Record Yr 2000: 10500 Gpm Max & Norm
28-JAN-16 Compliance Check/In Comp

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:
21-NOV-96 Plan Required With Approval Compliance date: 31-DEC-98
28-NOV-99 Approved Conservation Plan
**SPECIAL CONDITIONS:**

**QUANTITIES:**
- AUTHORIZED: 2224.000 mgy
- ADDITIONAL: 2224.000 mgy

**RATES:**
- AUTHORIZED: 10500.000 gpm
- ADDITIONAL: .000 gpm

**LIMITATIONS:**
- Combined Rate: 10500 GPM COM/W #42541

**STORAGE QUANTITIES:** No active storage quantities associated with MUN use under this water right

**STORAGE RATES:** No active storage rates associated with MUN use under this water right

**AUTHORIZED POINT(S) OF DIVERSION**
- Section 27, T 12, R 22E, ID 2 (Internal PDIV_ID = 60024)
- QUALIFIERS: SE NE NE
- DIST. FROM SE CORNER: 4090 ft North, 67 ft West
- COMMENT: WATER STRUCTURE PERMIT #LJO-0126 - RANNEY WELL
- OLD LONGITUDE: 94.946022
- OLD LATITUDE: 38.982330
- NEW LONGITUDE: 94.946020
- NEW LATITUDE: 38.982330
- GPS FEET NORTH: 4090
- GPS FEET WEST: 67
- COUNTY: JOHNSON
- FIELD OFFICE: TOPEKA FIELD OFFICE
- BASIN: KANSAS RIVER
- STREAM: KANSAS RIVER
- SPECIAL_USE_AREA(S): MAIN STEM ALLUVIUM
- AQUIFER(S): MAIN STEM ALLUVIUM

**TEST INFORMATION:**
- 28-AUG-08: 23 cfs Field Inspection Test

**METER ACTION TRAIL:**

**METER INFORMATION:**
- Date Installed: 24-MAY-99
- Currently Installed? N
- Manufacturer: FISHER + PORTER
- Model: 10DX3111
- Type: Not Available
- Serial No.: 98W025474
- Meter Unit: Gallons
- Meter Size: N/A
- Multiplier: N/A
- Portable Pump Installation? Yes
- Multiple PDs? No
- Straightening Vanes? No
- Measuring Chamber? Yes
- Meter Comment:

**METER INFORMATION:**
- Date Installed: 01-JAN-08
- Currently Installed? Y
- Manufacturer: FOXBORO
- Model: 9124A-SIBA-N556
> Type: Not Available  Serial No.:
> Meter Unit: Gallons  Meter Size: N/A  Multiplier: N/A
> Portable Pump Installation?  Multiple PDs?  Straightening Vanes?
> Measuring Chamber?  Y  Meter Comment: ACTUAL SIZE 24 IN

OVERLAPS:
> A 42541 00 MUN

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AUTHORIZED PLACE(S) OF USE
Section 10, T 14, R 23E, ID 1 (Internal PUSE_ID = 5051)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY
OVERLAPS:
> A 10042 00 MUN
> A 44613 00 MUN
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> A 42541 00 MUN
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Section 10, T 14, R 23E, ID 2 (Internal PUSE_ID = 19416)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED
OVERLAPS:
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Section 10, T 14, R 23E, ID 3 (Internal PUSE_ID = 50863)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07
OVERLAPS:
Section 10, T 14, R 23E, ID 4 (Internal PUSE_ID = 50864)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1
OVERLAPS:
> A 44613 00 MUN
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Section 10, T 14, R 23E, ID 5 (Internal PUSE_ID = 50865)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E
OVERLAPS:
> A 44613 00 MUN
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Section 10, T 14, R 23E, ID 6 (Internal PUSE_ID = 50866)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE

OVERLAPS:
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Horizontal Collector Well 2
WATER RIGHT INFORMATION REPORT FOR: 44613 00

RIGHT TYPE: Appropriation
SOURCE: Groundwater USE: MUN
CURRENT STATUS: Inspected Pending Perfection
PRIORITY DATE: 09-APR-01
CURRENT COMPLETE BY DATE: 31-DEC-02
COMPLETION ACKNOWLEDGED DATE: 16-MAY-03
CURRENT PERFECT BY DATE: 31-DEC-21

YEAR PERFECTED:
CERTIFICATE ISSUED DATE:

APPLICANT(S):
   > CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
   > PERSON ID (Old Address Code): 13184
   > CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):
   > CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
   > PERSON ID (Old Address Code): 13184
   > CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:
09-APR-01 Pending Initial Review
27-APR-01 New Application Initial Review Began
27-APR-01 New App Suspended For Comments From Adj Right Holders
27-APR-01 New App Suspended Pending Action On (Another?) Right
29-OCT-01 Pending Chief Engineer Action
14-NOV-01 Approved Pending Completion N+P BY 31-DEC-02 PERF BY 31-DEC-21
14-NOV-01 Assurance District Eligibility
16-MAY-03 Completed Pending Inspection
15-JUL-03 Water Conservation Plan Microfilmed Under Jo4
27-AUG-08 Inspected Pending Perfection
27-AUG-08 Compliance Check/Not In Comp
15-DEC-08 Compliance Check/In Comp
30-NOV-09 Conservation Plan Scanned Under Jo4
23-MAY-11 Conservation Plan By Johnson Rwd #7
23-MAY-11 Conservation Plan Scanned Under 10042
28-JAN-16 Compliance Check/In Comp

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:
15-JUL-03 Volunteer Plan
15-JUL-03 Approved Conservation Plan
23-MAY-11 Approved Conservation Plan
30-NOV-09 Modified Conservation Plan Approved
23-MAY-11 Plan Required By Kansas Water Office
30-NOV-09 Modified Conservation Plan Volunteered

SPECIAL CONDITIONS:
QUANTITIES:
  > AUTHORIZED 171.072 mgy ADDITIONAL 171.072 mgy

RATES:
  > AUTHORIZED 7000.000 gpm ADDITIONAL 7000.000 gpm

LIMITATIONS: None

STORAGE QUANTITIES: No active storage quantities associated with MUN use under this water right
STORAGE RATES: No active storage rates associated with MUN use under this water right

AUTHORIZED POINT(S) OF DIVERSION
Section 25, T 12, R 22E, ID 6 (Internal PDIV_ID = 65938)
QUALIFIERS: NE NW NW
DIST. FROM SE CORNER: 5140 ft North 4345 ft West
NUMBER OF WELLS: 1
COMMENT: STRUCTURES #LJ0-0191
OLD LONGITUDE: OLD LATITUDE:
NEW LONGITUDE: 94.923990 NEW LATITUDE: 38.984984
GPS LONGITUDE: 94.923660 GPS LATITUDE: 38.984920
GPS FEET NORTH: 5116 GPS FEET WEST: 4251
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:

BASIN: KANSAS RIVER
STREAM: KANSAS RIVER
SPECIAL_USE AREA(S):
AQUIFER(S):
  > MAIN STEM ALLUVIUM

TEST INFORMATION:
  > 27-AUG-08 8332 gpm Field Inspection Test

METER ACTION TRAIL:

METER INFORMATION:
Date Installed: 15-AUG-02 Currently Installed? N
  > Manufacturer: FOXBORO Model: 9118ASIBAESJGM
  > Type: Electromagnetic Serial No.: FE-1
  > Meter Unit: N/A Meter Size: N/A Multiplier: N/A
  > Portable Pump Installation? Multiple PDe? Straightening Vanes?
  > Measuring Chamber? Meter Comment: ACTUAL SIZE 18"

METER INFORMATION:
Date Installed: Currently Installed? Y
  > Manufacturer: FOXBORO Model: MT-25
  > Type: Electromagnetic Serial No.: 02211230
  > Meter Unit: Gallons Meter Size: N/A Multiplier: 000
  > Portable Pump Installation? Multiple PDe? Straightening Vanes?
  > Measuring Chamber? Y Meter Comment: METER ACTUAL SIZE 18"

OVERLAPS:
  > A 44614 00 MUN

AUTHORIZED PLACE(S) OF USE
Section 10, T 14, R 23E, ID 1 (Internal PUSE_ID = 5051)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY
OVERLAPS:
> A 10042 00 MUN
> A 44614 00 MUN
> A 42542 00 MUN
> A 42541 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

Section 10, T 14, R 23E, ID 2 (Internal PUSE_ID = 19416)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED
OVERLAPS:
> A 10042 00 MUN
> A 44614 00 MUN
> A 42542 00 MUN
> A 42541 00 MUN
> A 45994 00 MUN
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> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

Section 10, T 14, R 23E, ID 3 (Internal PUSE_ID = 50863)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07
OVERLAPS:
> A 44614 00 MUN
> A 10042 00 MUN
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> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN
Section 10, T 14, R 23E, ID  4 (Internal PUSE_ID = 50864)
OWNER:  CITY OF OLATHE
Address:  JOHN GILROY
**  PO BOX 768
**  OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1
OVERLAPS:
> A  47203 00 MUN
----------------
Section 10, T 14, R 23E, ID  5 (Internal PUSE_ID = 50865)
OWNER:  CITY OF OLATHE
Address:  JOHN GILROY
**  PO BOX 768
**  OLATHE KS 66051
COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E
OVERLAPS:
> A  47203 00 MUN
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Section 10, T 14, R 23E, ID  6 (Internal PUSE_ID = 50866)
OWNER:  CITY OF OLATHE
Address:  JOHN GILROY
**  PO BOX 768
**  OLATHE KS 66051
COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE
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WATER RIGHT INFORMATION REPORT FOR : 44614 00

RIGHT TYPE: Appropriation
SOURCE: Surface Water        USE: MUN
CURRENT STATUS: Inspected Pending Perfection
PRIORITY DATE: 09-APR-01
CURRENT COMPLETE BY DATE: 31-DEC-02
COMPLETION ACKNOWLEDGED DATE: 16-MAY-03
CURRENT PERFECT BY DATE: 31-DEC-21

APPLICANT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:
09-APR-01 Pending Initial Review
27-APR-01 New Application Initial Review Began
27-APR-01 New App Suspended For Comments From Adj Right Holders
27-APR-01 New App Suspended Pending Action On (Another?) Right
29-OCT-01 Pending Chief Engineer Action
14-NOV-01 Approved Pending Completion N+P BY 31-DEC-02 PERF BY 31-DEC-21
14-NOV-01 Assurance District Eligibility
28-AUG-02 Corr O: Document Error
16-MAY-03 Completed Pending Inspection
15-JUL-03 Water Conservation Plan Microfilmed Under Jo4
27-AUG-08 Inspected Pending Perfection
27-AUG-08 Compliance Check/Not In Comp
15-DEC-08 Compliance Check/In Comp
30-NOV-09 Conservation Plan Scanned Under Jo 4
23-MAY-11 Conservation Plan By Johnson Rwd #7
23-MAY-11 Conservation Plan Scanned Under 10042
28-JAN-16 Compliance Check/In Comp

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:
15-JUL-03 Volunteer Plan
15-JUL-03 Approved Conservation Plan
23-MAY-11 Approved Conservation Plan
30-NOV-09 Modified Conservation Plan Approved
23-MAY-11 Plan Required By Kansas Water Office
30-NOV-09 Modified Conservation Plan Volunteered
SPECIAL CONDITIONS:

QUANTITIES:
> AUTHORIZED 1539.646 mgY ADDITIONAL 1539.646 mgY

RATES:
> AUTHORIZED 7000.000 gpm ADDITIONAL .000 gpm

LIMITATIONS:
> Combined Quantity 6393.569 MGY/YR COM/W #JO-04;9590;10042;32825;42541;42542;44613
> Combined Rate 7000GPM COM/W #44613

STORAGE QUANTITIES: No active storage quantities associated with MUN use under this water right
STORAGE RATES: No active storage rates associated with MUN use under this water right

AUTHORIZED POINT(S) OF DIVERSION
Section 25, T 12, R 22E, ID 6 (Internal PDIV_ID = 65938)
QUALIFIERS: NE NW NW
DIST. FROM SE CORNER: 5140 ft North 4345 ft West
COMMENT: STRUCTURES #LJ0-0191
OLD LONGITUDE: OLD LATITUDE:
NEW LONGITUDE: 94.923990 NEW LATITUDE: 38.984984
GPS LONGITUDE: 94.923660 GPS LATITUDE: 38.984920
GPS FEET NORTH: 5116 GPS FEET WEST: 4251
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD :
BASIN: KANSAS RIVER
STREAM: KANSAS RIVER
SPECIAL_USE_AREA(S):
AQUIFER(S):
> MAIN STEM ALLUVIUM

TEST INFORMATION:
> 27-AUG-08 8332 gpm Field Inspection Test

METER ACTION TRAIL:
METER INFORMATION:
Date Installed: 15-AUG-02 Currently Installed? N
> Manufacturer: FOXBORO Model: 9118ASIBAESJGM
> Type: Electromagnetic Serial No.: PE-1
> Meter Unit: N/A Meter Size: N/A Multiplier: N/A
> Portable Pump Installation? Multiple PDs? Straightening Vanes?
> Measuring Chamber? Meter Comment: ACTUAL SIZE 18"

METER INFORMATION:
Date Installed: Currently Installed? Y
> Manufacturer: FOXBORO Model: MT-25
> Type: Electromagnetic Serial No.: 02211230
> Meter Unit: Gallons Meter Size: N/A Multiplier: 000
> Portable Pump Installation? Multiple PDs? Straightening Vanes?
> Measuring Chamber? Y Meter Comment: METER ACTUAL SIZE 18"
OVERLAPS:
> A 44613 00 MUN

----------------
AUTHORIZED PLACE(S) OF USE

Section 10, T 14, R 23E, ID   1 (Internal PUSE_ID = 5051)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
**   PO BOX 768
**   OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY
OVERLAPS:
> A  10042  00 MUN
> A  44613  00 MUN
> A  42542  00 MUN
> A  42541  00 MUN
> A  45994  00 MUN
> A  45993  00 MUN
> A  45648  00 MUN
> A  45649  00 MUN
> A  47202  00 MUN
> A  47203  00 MUN

-------------

Section 10, T 14, R 23E, ID   2 (Internal PUSE_ID = 19416)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
**   PO BOX 768
**   OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED
OVERLAPS:
> A  10042  00 MUN
> A  44613  00 MUN
> A  42542  00 MUN
> A  42541  00 MUN
> A  45994  00 MUN
> A  45993  00 MUN
> A  45648  00 MUN
> A  45649  00 MUN
> A  47202  00 MUN
> A  47203  00 MUN

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Section 10, T 14, R 23E, ID   3 (Internal PUSE_ID = 50863)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
**   PO BOX 768
**   OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07
OVERLAPS:
> A  44613  00 MUN
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> A  45648  00 MUN
Section 10, T 14, R 23E, ID 4 (Internal PUSE_ID = 50864)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1
OVERLAPS:
> A 44613 00 MUN
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Section 10, T 14, R 23E, ID 5 (Internal PUSE_ID = 50865)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E
OVERLAPS:
> A 44613 00 MUN
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Section 10, T 14, R 23E, ID 6 (Internal PUSE_ID = 50866)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE
OVERLAPS:
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Horizontal Collector Well 3
WATER RIGHT INFORMATION REPORT FOR: 45648 00

RIGHT TYPE: Appropriation

SOURCE: Surface Water USE: MUN

CURRENT STATUS: Inspected Pending Perfection

PRIORITY DATE: 08-SEP-03

CURRENT COMPLETE BY DATE: 31-DEC-05

COMPLETION ACKNOWLEDGED DATE: 19-OCT-05

CURRENT PERFECT BY DATE: 31-DEC-24

YEAR PERFECTED:

CERTIFICATE ISSUED DATE:

APPLICANT(S):

> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):

> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:

08-SEP-03 Pending Initial Review
12-SEP-03 New Application Initial Review Began
12-SEP-03 New Application Returned For Additional Data
30-SEP-03 New Application Special Projects
14-OCT-03 New App Suspended For Comments From Adj Right Holders
14-NOV-03 Pending Chief Engineer Action
17-NOV-03 Approved Pending Completion N+P BY 31-DEC-05 PERF BY 31-DEC-24
17-NOV-03 Assurance District Eligibility
19-OCT-05 Completed Pending Inspection
27-AUG-08 Inspected Pending Perfection
27-AUG-08 Compliance Check/Not In Comp
15-DEC-08 Compliance Check/In Comp
30-NOV-09 Conservation Plan Scanned Under Jo 4
23-MAY-11 Conservation Plan By Johnson Rwd #7
23-MAY-11 Conservation Plan Scanned Under 10042
28-JAN-16 Compliance Check/In Comp

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:

30-NOV-09 Modified Conservation Plan Volunteered
23-MAY-11 Approved Conservation Plan
23-MAY-11 Plan Required By Kansas Water Office
30-NOV-09 Modified Conservation Plan Approved

SPECIAL CONDITIONS:

QUANTITIES:
>AUTHORIZED 1539.646 mgy ADDITIONAL 253.033 mgy

RATES:
>AUTHORIZED 7000.000 gpm ADDITIONAL 7000.000 gpm

LIMITATIONS:
> Combined Quantity SEE JKT FOR LIMITATION

STORAGE QUANTITIES: No active storage quantities associated with MUN use under this water right
STORAGE RATES: No active storage rates associated with MUN use under this water right

AUTHORIZED POINT(S) OF DIVERSION
Section 26, T 12, R 22E, ID 11 (Internal PDIV_ID = 69418)
QUALIFIERS: NW NW NW
DIST. FROM SE CORNER: 4950 ft North 4666 ft West
COMMENT: STRUCTURES PERMIT #LJO-212
OLD LONGITUDE: 94.943506 OLD LATITUDE: 38.984569
NEW LONGITUDE: 94.943510 GPS LATITUDE: 38.984500
GPS FEET NORTH: 4924 GPS FEET WEST: 4667
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:
BASIN: KANSAS RIVER
STREAM: KANSAS RIVER
SPECIAL_USE_AREA(S):
AQUIFER(S):
> MAIN STEM ALLUVIUM
TEST INFORMATION:
> 27-AUG-08 7986 gpm Field Inspection Test

METER ACTION TRAIL:
METER INFORMATION:
Date Installed: Currently Installed? Y
> Manufacturer: FOXBORO Model: IMT-25
> Type: Not Available Serial No.: 0221123
> Meter Unit: N/A Meter Size: N/A Multiplier: N/A
> Portable Pump Installation? Multiple PDs? Straightening Vanes?
> Measuring Chamber? Meter Comment:
OVERLAPS:
> A 45649 00 MUN
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AUTHORIZED PLACE(S) OF USE
Section 10, T 14, R 23E, ID 1 (Internal PUSE_ID = 5051)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY
OVERLAPS:
> A 10042 00 MUN
> A 44613 00 MUN
Section 10, T 14, R 23E, ID 2 (Internal PUSE_ID = 19416)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED
OVERLAPS:
> A 10042 00 MUN
> A 44613 00 MUN
> A 44614 00 MUN
> A 42542 00 MUN
> A 42541 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

Section 10, T 14, R 23E, ID 3 (Internal PUSE_ID = 50863)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07
OVERLAPS:
> A 44613 00 MUN
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> A 45993 00 MUN
> A 45649 00 MUN
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> A 47203 00 MUN

Section 10, T 14, R 23E, ID 4 (Internal PUSE_ID = 50864)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1
OVERLAPS:
> A 44613 00 MUN
> A 44614 00 MUN
> A 10042 00 MUN
> A 42541 00 MUN
> A 42542 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

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Section 10, T 14, R 23E, ID 5 (Internal PUSE_ID = 50865)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051

COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E

OVERLAPS:
> A 44613 00 MUN
> A 44614 00 MUN
> A 10042 00 MUN
> A 42541 00 MUN
> A 42542 00 MUN
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> A 45993 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

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Section 10, T 14, R 23E, ID 6 (Internal PUSE_ID = 50866)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051

COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE

OVERLAPS:
> A 44613 00 MUN
> A 44614 00 MUN
> A 10042 00 MUN
> A 42541 00 MUN
> A 42542 00 MUN
> A 45994 00 MUN
> A 45993 00 MUN
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WATER RIGHT INFORMATION REPORT FOR : 45649 00

RIGHT TYPE: Appropriation
SOURCE: Groundwater USE: MUN
CURRENT STATUS: Inspected Pending Perfection
PRIORITY DATE: 08-SEP-03
CURRENT COMPLETE BY DATE: 31-DEC-04
COMPLETION ACKNOWLEDGED DATE: 19-OCT-05
CURRENT PERFECT BY DATE: 31-DEC-24

YEAR PERFECTED:
CERTIFICATE ISSUED DATE:

APPLICANT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:
08-SEP-03 Pending Initial Review
12-SEP-03 New Application Initial Review Began
12-SEP-03 New Application Returned For Additional Data
30-SEP-03 New Application Special Projects
14-OCT-03 New App Suspended For Comments From Adj Right Holders
14-NOV-03 Pending Chief Engineer Action
17-NOV-03 Approved Pending Completion N+P BY 31-DEC-04 PERF BY 31-DEC-24
17-NOV-03 Assurance District Eligibility
08-DEC-03 Corr O: Document Error
19-OCT-05 Completed Pending Inspection
15-DEC-05 Corr O: Document Error
27-AUG-08 Inspected Pending Perfection
27-AUG-08 Compliance Check/Not In Comp
15-DEC-08 Compliance Check/In Comp
30-NOV-09 Conservation Plan Scanned Under Jo 4
23-MAY-11 Conservation Plan By Johnson Rwd #7
23-MAY-11 Conservation Plan Scanned Under 10042
28-JAN-16 Compliance Check/In Comp

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:
30-NOV-09 Modified Conservation Plan Volunteered
23-MAY-11 Approved Conservation Plan
23-MAY-11 Plan Required By Kansas Water Office
30-NOV-09 Modified Conservation Plan Approved

SPECIAL CONDITIONS:
QUANTITIES:
> AUTHORIZED 171.072 mgy ADDITIONAL .000 mgy

RATES:
> AUTHORIZED 7000.000 gpm ADDITIONAL .000 gpm

LIMITATIONS:
> Combined Quantity SEE JKT FOR LIMITATION
> Combined Rate 7000GPM COM/W #45648

STORAGE QUANTITIES: No active storage quantities associated with MUN use under this water right
STORAGE RATES: No active storage rates associated with MUN use under this water right

AUTHORIZED POINT(S) OF DIVERSION
Section 26, T 12, R 22E, ID 11 (Internal PDIV_ID = 69418)
QUALIFIERS: NW NW NW
DIST. FROM SE CORNER: 4950 ft North 4666 ft West
NUMBER OF WELLS: 1
COMMENT: STRUCTURES PERMIT #LJO-0212
OLD LONGITUDE: 94.943506
NEW LONGITUDE: 94.943510
GPS LONGITUDE: 94.943510
GPS FEET NORTH: 4924
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:
BASIN: KANSAS RIVER
STREAM: KANSAS RIVER
SPECIAL USE AREA(S):
> MAIN STEM ALLUVIUM
TEST INFORMATION:
> 27-AUG-08 7986 gpm Field Inspection Test
METER ACTION TRAIL:
METER INFORMATION:
Date Installed: Currently Installed? Y
> Manufacturer: FOXBORO Model: IMT-25
> Type: Not Available Serial No.: 0221123
> Meter Unit: N/A Meter Size: N/A Multiplier: N/A
> Portable Pump Installation? Multiple PDs? Straightening Vanes?
> Measuring Chamber? Meter Comment:
OVERLAPS:
> A 45648 00 MUN

AUTHORIZED PLACE(S) OF USE
Section 10, T 14, R 23E, ID 1 (Internal FUSE_ID = 5051)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY

OVERLAPS:
> A 10042 00 MUN
> A 44613 00 MUN
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> A 45648 00 MUN
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> A 47203 00 MUN

Section 10, T 14, R 23E, ID 2 (Internal PUSE_ID = 19416)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED
OVERLAPS:
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Section 10, T 14, R 23E, ID 3 (Internal PUSE_ID = 50863)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07
OVERLAPS:
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> A 45993 00 MUN
> A 45648 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

Section 10, T 14, R 23E, ID 4 (Internal PUSE_ID = 50864)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
**       PO BOX 768
**       OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1
OVERLAPS:
> A       44613 00 MUN
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> A       10042 00 MUN
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Section 10, T 14, R 23E, ID 5 (Internal PUSE_ID = 50865)
OWNER:   CITY OF OLATHE
Address: JOHN GILROY
**       PO BOX 768
**       OLATHE KS 66051
COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E
OVERLAPS:
> A       44613 00 MUN
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Section 10, T 14, R 23E, ID 6 (Internal PUSE_ID = 50866)
OWNER:   CITY OF OLATHE
Address: JOHN GILROY
**       PO BOX 768
**       OLATHE KS 66051
COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE
OVERLAPS:
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Horizontal Collector Well 4
WATER RIGHT INFORMATION REPORT FOR : 45993 00

RIGHT TYPE: Appropriation
SOURCE: Groundwater USE: MUN
CURRENT STATUS: Inspected Pending Perfection
PRIORITY DATE: 15-JUL-04
CURRENT COMPLETE BY DATE: 31-DEC-05
COMPLETION ACKNOWLEDGED DATE: 11-APR-06
CURRENT PERFECT BY DATE: 31-DEC-24

YEAR PERFECTED:
CERTIFICATE ISSUED DATE:

APPLICANT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:
15-JUL-04 Pending Initial Review
26-JUL-04 New Application Initial Review Began
26-JUL-04 New App Suspended For Comments From Applicant
15-OCT-04 Pending Chief Engineer Action
20-OCT-04 Approved Pending Completion N+P BY 31-DEC-05 PERF BY 31-DEC-24
20-OCT-04 Assurance District Eligibility
11-APR-06 Completed Pending Inspection
28-AUG-08 Inspected Pending Perfection
28-AUG-08 Compliance Check/Not In Comp
06-MAY-09 Compliance Check/In Comp
30-NOV-09 Conservation Plan Scanned Under Jo 4
23-MAY-11 Conservation Plan By Johnson Rwd #7
23-MAY-11 Conservation Plan Scanned Under 10042
28-JAN-16 Compliance Check/In Comp

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:
30-NOV-09 Modified Conservation Plan Volunteered
23-MAY-11 Approved Conservation Plan
23-MAY-11 Plan Required By Kansas Water Office
30-NOV-09 Modified Conservation Plan Approved

SPECIAL CONDITIONS:

QUANTITIES:
> AUTHORIZED 130.340 mgy ADDITIONAL 130.340 mgy
RATES:
> AUTHORIZED  4500.000 gpm  ADDITIONAL  .000 gpm

LIMITATIONS:
> Combined Quantity  7146.058MGY COM/W - SEE JKT FOR FILES

STORAGE QUANTITIES: No active storage quantities associated with MUN use under this water right
STORAGE RATES:  No active storage rates associated with MUN use under this water right

AUTHORIZED POINT(S) OF DIVERSION
Section  27, T 12, R 22E,  ID   5  (Internal PDIV_ID = 70633)
QUALIFIERS:  NW SE NE
DIST. FROM SE CORNER:  3704 ft North  849 ft West
NUMBER OF WELLS:  1
COMMENT: STRUCTURE PERMIT # LJO-0227
OLD LONGITUDE:  OLD LATITUDE:
NEW LONGITUDE:  94.948772  NEW LATITUDE:  38.981270
GPS LONGITUDE:  94.948050  GPS LATITUDE:  38.981440
GPS FEET NORTH:  3766  GPS FEET WEST:  644
COUNTY:  JOHNSON
FIELD OFFICE:  TOPEKA FIELD OFFICE
GMD :
BASIN:  KANSAS RIVER
STREAM:  KANSAS RIVER
SPECIAL_USE_AREA(S):
AQUIFER(S):
TEST INFORMATION:
> 28-AUG-08  7638 gpm  Field Inspection Test

METER ACTION TRAIL:
METER INFORMATION:
Date Installed :  Currently Installed? Y
>  Manufacturer: FOXBORO  Model: IMT25-SDADB11M
>  Type: Not Available  Serial No.: 05150848
>  Meter Unit: N/A  Meter Size: N/A  Multiplier: N/A
>  Portable Pump Installation?  Multiple PDs?  Straightening Vanes?
>  Measuring Chamber?  Meter Comment: ACTUAL SIZE 18 IN

OVERLAPS:
> A   45994 00 MUN

AUTHORIZED PLACE(S) OF USE
Section 10, T 14, R 23E,  ID   1  (Internal PUSE_ID = 5051)
OWNER:  CITY OF OLATHE
Address:  JOHN GILROY
**  PO BOX 768
**  OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY

OVERLAPS:
> A   10042 00 MUN
> A   44613 00 MUN
> A   44614 00 MUN
> A   42542 00 MUN
> A       42541 00 MUN
> A       45994 00 MUN
> A       45648 00 MUN
> A       45649 00 MUN
> A       47202 00 MUN
> A       47203 00 MUN

----------------
Section  10, T 14, R 23E, ID  2 (Internal PUSE_ID = 19416)
OWNER:   CITY OF OLATHE
Address: JOHN GILROY
**       PO BOX 768
**       OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED
OVERLAPS:
> A       10042 00 MUN
> A       44613 00 MUN
> A       44614 00 MUN
> A       42542 00 MUN
> A       42541 00 MUN
> A       45994 00 MUN
> A       45648 00 MUN
> A       45649 00 MUN
> A       47202 00 MUN
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----------------
Section  10, T 14, R 23E, ID  3 (Internal PUSE_ID = 50863)
OWNER:   CITY OF OLATHE
Address: JOHN GILROY
**       PO BOX 768
**       OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07
OVERLAPS:
> A       44613 00 MUN
> A       44614 00 MUN
> A       10042 00 MUN
> A       42542 00 MUN
> A       42541 00 MUN
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> A       45648 00 MUN
> A       45649 00 MUN
> A       47202 00 MUN
> A       47203 00 MUN

----------------
Section  10, T 14, R 23E, ID  4 (Internal PUSE_ID = 50864)
OWNER:   CITY OF OLATHE
Address: JOHN GILROY
**       PO BOX 768
**       OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1
OVERLAPS:
> A       44613 00 MUN
> A       44614 00 MUN
> A       10042 00 MUN
> A       42541 00 MUN
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> A       45648 00 MUN
> A       45649 00 MUN
> A       47202 00 MUN
> A       47203 00 MUN

----------------
Section 10, T 14, R 23E, ID 5 (Internal PUSE_ID = 50865)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E
OVERLAPS:

----------------
Section 10, T 14, R 23E, ID 6 (Internal PUSE_ID = 50866)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE
OVERLAPS:

----------------
Section 27, T 12, R 22E, ID 3 (Internal PUSE_ID = 50019)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY
OVERLAPS:
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WATER RIGHT INFORMATION REPORT FOR: 45994 00

RIGHT TYPE: Appropriation
SOURCE: Surface Water USE: MUN
CURRENT STATUS: Inspected Pending Perfection
PRIORITY DATE: 15-JUL-04
CURRENT COMPLETE BY DATE: 31-DEC-05
COMPLETION ACKNOWLEDGED DATE: 11-APR-06
CURRENT PERFECT BY DATE: 31-DEC-24

CERTIFICATE ISSUED DATE:

APPLICANT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:
15-JUL-04 Pending Initial Review
26-JUL-04 New Application Initial Review Began
26-JUL-04 New App Suspended For Comments From Applicant
15-OCT-04 Pending Chief Engineer Action
20-OCT-04 Approved Pending Completion N+P BY 31-DEC-05 PERF BY 31-DEC-24
20-OCT-04 Assurance District Eligibility
15-DEC-05 Corr O: Document Error
11-APR-06 Completed Pending Inspection
28-AUG-08 Inspected Pending Perfection
28-AUG-08 Compliance Check/Not In Comp
06-MAY-09 Compliance Check/In Comp
30-NOV-09 Conservation Plan Scanned Under Jo 4
23-MAY-11 Conservation Plan By Johnson Rwd #7
23-MAY-11 Conservation Plan Scanned Under 10042
28-JAN-16 Compliance Check/In Comp

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:
30-NOV-09 Modified Conservation Plan Volunteered
23-MAY-11 Approved Conservation Plan
23-MAY-11 Plan Required By Kansas Water Office
30-NOV-09 Modified Conservation Plan Approved

SPECIAL CONDITIONS:

QUANTITIES:
>AUTHORIZED 2215.787 mgy ADDITIONAL 622.150 mgy
REPORT DATE: Thursday, June 30 2016

RATES:
> AUTHORIZED  4500.000 gpm  ADDITIONAL  .000 gpm

LIMITATIONS:
> Combined Quantity  7146.058 MGY COM/W - SEE JKT FOR FILES

STORAGE QUANTITIES: No active storage quantities associated with MUN use under this water right
STORAGE RATES: No active storage rates associated with MUN use under this water right

AUTHORIZED POINT(S) OF DIVERSION
Section 27, T 12, R 22E, ID 5 (Internal PDIV_ID = 70633)
QUALIFIERS: NW SE NE
DIST. FROM SE CORNER: 3704 ft North 849 ft West
COMMENT: STRUCTURE PERMIT # LJO-0227
OLD LATITUDE: 38.981270
OLD LONGITUDE: 94.948772
NEW LATITUDE: 38.981440
NEW LONGITUDE: 94.948050
GPS FEET NORTH: 3766
GPS FEET WEST: 644
COUNTY: JOHNSON
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:
BASIN: KANSAS RIVER
STREAM: KANSAS RIVER
SPECIAL_USE_AREA(S):
AQUIFER(S):
TEST INFORMATION:
> 28-AUG-08  7638 gpm Field Inspection Test
METER ACTION TRAIL:
METER INFORMATION:
Date Installed: Currently Installed? Y
> Manufacturer: FOXBORO  Model: IMT25-SDADB11M
> Type: Not Available  Serial No.: 05150848
> Meter Unit: N/A  Meter Size: N/A  Multiplier: N/A
> Portable Pump Installation?  Multiple PDs?  Straightening Vanes?
> Measuring Chamber?  Meter Comment: ACTUAL SIZE 18 IN
OVERLAPS:
> A  45993 00 MUN

AUTHORIZED PLACE(S) OF USE
Section 10, T 14, R 23E, ID 1 (Internal PUSE_ID = 5051)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
**  PO BOX 768
**  OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY
OVERLAPS:
> A  10042 00 MUN
> A  44613 00 MUN
> A  44614 00 MUN
> A  42542 00 MUN
Section 10, T 14, R 23E, ID 2 (Internal PUSE_ID = 19416)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED
OVERLAPS:
> A 10042 00 MUN
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> A 44614 00 MUN
> A 42542 00 MUN
> A 42541 00 MUN
> A 45993 00 MUN
> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

Section 10, T 14, R 23E, ID 3 (Internal PUSE_ID = 50863)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07
OVERLAPS:
> A 44613 00 MUN
> A 44614 00 MUN
> A 10042 00 MUN
> A 42541 00 MUN
> A 42542 00 MUN
> A 45993 00 MUN
> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN
> A 47203 00 MUN

Section 10, T 14, R 23E, ID 4 (Internal PUSE_ID = 50864)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1
OVERLAPS:
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> A 44614 00 MUN
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Horizontal Collector Well 5
WATER RIGHT INFORMATION REPORT FOR: 47202 00

RIGHT TYPE: Appropriation

SOURCE: Groundwater        USE: MUN

CURRENT STATUS: Extended Time To Complete

PRIORITY DATE: 18-NOV-08

CURRENT COMPLETE BY DATE: 31-DEC-17

COMPLETION ACKNOWLEDGED DATE:

CURRENT PERFECT BY DATE: 31-DEC-30

YEAR PERFECTED:

CERTIFICATE ISSUED DATE:

APPLICANT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):
> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:
18-NOV-08  Pending Initial Review
01-DEC-08  New Application Initial Review Began
02-DEC-08  New App Suspended For Comments From Applicant
02-DEC-08  New App Suspended For Comments From Adj Right Holders
10-NOV-09  New App Suspended For Comments From Field Office
10-NOV-09  New App Suspended For Other Dwr Permitting
25-FEB-10  Pending Chief Engineer Action
01-MAR-10  Approved Pending Completion  N+P BY 31-DEC-11  PERF BY 31-DEC-30
23-MAY-11  Conservation Plan By Johnson Rwd #7
23-MAY-11  Conservation Plan Scanned Under 10042
19-DEC-11  Extended Time To Complete  N+P BY 31-DEC-12  PERF BY 31-DEC-30
03-JUL-14  Extended Time To Complete  N+P BY 31-DEC-17  PERF BY 31-DEC-30

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:
23-MAY-11  Approved Conservation Plan
23-MAY-11  Plan Required By Kansas Water Office

SPECIAL CONDITIONS:

QUANTITIES:
>AUTHORIZED 97.755 mgy ADDITIONAL 97.755 mgy

RATES:
>AUTHORIZED 9500.000 gpm ADDITIONAL 9500.000 gpm

LIMITATIONS: None
STORAGE QUANTITIES: No active storage quantities associated with MUN use under this water right
STORAGE RATES: No active storage rates associated with MUN use under this water right

AUTHORIZED POINT(S) OF DIVERSION

Section 27, T 12, R 22E, ID 6 (Internal PDIV_ID = 76126)
QUALIFIERS: SW NE NW
DIST. FROM SE CORNER: 4458 ft North 3907 ft West
NUMBER OF WELLS: 1
COMMENT: STRUCTURE PERMIT #LLV-0114; COLLECTOR WELL #5
OLD LONGITUDE:  94.959528
NEW LONGITUDE:  94.959528
OLD LATITUDE:    38.983340
NEW LATITUDE:    38.983340
GPS LONGITUDE:   94.959528
GPS LATITUDE:    38.983340
GPS FEET NORTH:   4458
GPS FEET WEST:    3907
COUNTY: LEAVENWORTH
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD:
BASIN: KANSAS RIVER
STREAM: KANSAS RIVER
SPECIAL USE AREA(S):
AQUIFER(S):
> MAIN STEM ALLUVIUM
TEST INFORMATION:
METER ACTION TRAIL:
OVERLAPS:
> A  47203 00 MUN

AUTHORIZED PLACE(S) OF USE

Section 10, T 14, R 23E, ID 1 (Internal PUSE_ID = 5051)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY
OVERLAPS:
> A  10042 00 MUN
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> A  45648 00 MUN
> A  45649 00 MUN
> A  47203 00 MUN

Section 10, T 14, R 23E, ID 2 (Internal PUSE_ID = 19416)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED

OVERLAPS:

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Section 10, T 14, R 23E, ID 3 (Internal PUSE_ID = 50863)

OWNER:  CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051

COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07

OVERLAPS:

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----------------

Section 10, T 14, R 23E, ID 4 (Internal PUSE_ID = 50864)

OWNER:  CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051

COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1

OVERLAPS:

> A  44613 00 MUN
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> A  45648 00 MUN
> A  45649 00 MUN
> A  47203 00 MUN

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Section 10, T 14, R 23E, ID 5 (Internal PUSE_ID = 50865)

OWNER:  CITY OF OLATHE
Address: JOHN GILROY
**       PO BOX 768
**       OLATHE KS 66051

COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E

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Section 10, T 14, R 23E, ID 6 (Internal PUSE_ID = 50866)

OWNER: CITY OF OLATHE
Address: JOHN GILROY
**       PO BOX 768
**       OLATHE KS 66051

COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE

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WATER RIGHT INFORMATION REPORT FOR: 47203 00

RIGHT TYPE: Appropriation

SOURCE: Surface Water USE: MUN

CURRENT STATUS: Extended Time To Complete

PRIORITY DATE: 18-NOV-08

CURRENT COMPLETE BY DATE: 31-DEC-17

COMPLETION ACKNOWLEDGED DATE: 

CURRENT PERFECT BY DATE: 31-DEC-30

YEAR PERFECTED:

CERTIFICATE ISSUED DATE:

APPLICANT(S):

> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

WATER USE CORRESPONDENT(S):

> CITY OF OLATHE JOHN GILROY PO BOX 768 OLATHE KS 66051
> PERSON ID (Old Address Code): 13184
> CORRESPONDENT SEQUENCE NUMBER: 1

ACTION TRAIL:

18-NOV-08 Pending Initial Review

01-DEC-08 New Application Initial Review Began

02-DEC-08 New App Suspended For Comments From Applicant

02-DEC-08 New App Suspended For Comments From Adj Right Holders

10-NOV-09 New App Suspended For Comments From Field Office

10-NOV-09 New App Suspended For Other Dwr Permitting

25-FEB-10 Pending Chief Engineer Action

01-MAR-10 Approved Pending Completion N+P BY 31-DEC-11 PERF BY 31-DEC-30

23-MAY-11 Conservation Plan By Johnson Rwd #7

23-MAY-11 Conservation Plan Scanned Under 10042

19-DEC-11 Extended Time To Complete N+P BY 31-DEC-12 PERF BY 31-DEC-30

03-JUL-14 Extended Time To Complete N+P BY 31-DEC-17 PERF BY 31-DEC-30

CONSERVATION CONTRACT ACTION TRAIL:

CONSERVATION PLAN ACTION TRAIL:

23-MAY-11 Approved Conservation Plan

23-MAY-11 Plan Required By Kansas Water Office

SPECIAL CONDITIONS:

QUANTITIES:

AUTHORIZED  2280.960 mgd ADDITIONAL  2280.960 mgd

RATES:

AUTHORIZED  9500.000 gpm ADDITIONAL .000 gpm

LIMITATIONS:
> Combined Rate 9500GPM COM/W #47202

STORAGE QUANTITIES: No active storage quantities associated with MUN use under this water right

STORAGE RATES: No active storage rates associated with MUN use under this water right

AUTHORIZED POINT(S) OF DIVERSION
Section 27, T 12, R 22E, ID 6 (Internal PDIV_ID = 76126)
QUALIFIERS: SW NE NW
DIST. FROM SE CORNER: 4458 ft North 3907 ft West
COMMENT: STRUCTURE PERMIT #LLV-0114; COLLECTOR WELL #5
OLD LONGITUDE: OLD LATITUDE:
NEW LONGITUDE: 94.959528 NEW LATITUDE: 38.983340
GPS LONGITUDE: GPS LATITUDE:
GPS FEET NORTH: GPS FEET WEST:
COUNTY: LEAVENWORTH
FIELD OFFICE: TOPEKA FIELD OFFICE
GMD :
BASIN: KANSAS RIVER
STREAM: KANSAS RIVER
SPECIAL_USE_AREA(S):
AQUIFER(S):
> MAIN STEM ALLUVIUM
TEST INFORMATION:
METER ACTION TRAIL:
OVERLAPS:
> A 47202 00 MUN

AUTHORIZED PLACE(S) OF USE
Section 10, T 14, R 23E, ID 1 (Internal PUSE_ID = 5051)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: CITY OF OLATHE & IMMEDIATE VICINITY
OVERLAPS:
> A 10042 00 MUN
> A 44613 00 MUN
> A 44614 00 MUN
> A 42542 00 MUN
> A 42541 00 MUN
> A 45994 00 MUN
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> A 45648 00 MUN
> A 45649 00 MUN
> A 47202 00 MUN

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Section 10, T 14, R 23E, ID 2 (Internal PUSE_ID = 19416)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN BOUNDARIES OF JOHNSON RWD 06 CONSOLIDATED

OVERLAPS:
> A 10042 00 MUN
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> A 45993 00 MUN
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> A 47202 00 MUN

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Section 10, T 14, R 23E, ID 3 (Internal PUSE_ID = 50863)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON RWD 07

OVERLAPS:
> A 44613 00 MUN
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> A 10042 00 MUN
> A 42541 00 MUN
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> A 45994 00 MUN
> A 45993 00 MUN
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> A 45649 00 MUN
> A 47202 00 MUN

----------------
Section 10, T 14, R 23E, ID 4 (Internal PUSE_ID = 50864)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
** PO BOX 768
** OLATHE KS 66051
COMMENT: WITHIN THE BOUNDARIES OF JOHNSON COUNTY WATER DISTRICT #1

OVERLAPS:
> A 44613 00 MUN
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> A 10042 00 MUN
> A 42541 00 MUN
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> A 45994 00 MUN
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> A 45649 00 MUN
> A 47202 00 MUN

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Section 10, T 14, R 23E, ID 5 (Internal PUSE_ID = 50865)
OWNER: CITY OF OLATHE
Address: JOHN GILROY
COMMENT: NEW CENTURY AIR CENTER LOCATED IN 18-14-23E

OVERLAPS:
> A 44613 00 MUN
> A 44614 00 MUN
> A 10042 00 MUN
> A 42541 00 MUN
> A 42542 00 MUN
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> A 45649 00 MUN
> A 47202 00 MUN

Section 10, T 14, R 23E, ID 6 (Internal PUSE_ID = 50866)

OWNER: CITY OF OLATHE
Address: JOHN GILROY

COMMENT: IMM VIC OF MAIN FROM TREATMENT PLANT #2 SW 30-12-23E TO CITY OF OLATHE

OVERLAPS:
> A 44613 00 MUN
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> A 10042 00 MUN
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Appendix C
Well Construction Information
Well Construction Information

Municipal water supply wells are typically constructed to maximize production capacity given aquifer characteristics and applicable well construction methods and materials. Wells should be designed to be operated below a certain well screen inflow velocity with the pumping water level minimally a certain margin above the top of the well screen at the design flow rate. There should be additional margin above the well screen to allow the well to degrade in condition between well cleanings.

Current well construction standards from ANSI, AWWA, and KDHE allow for a maximum vertical well screen inflow velocity of 6 feet per minute (ft/min) assuming 50 percent of the screen will be blocked by aquifer or gravel pack material. It is recommended that the pumping water level stay above the top of the well screen to prevent accelerated clogging of the well. Vertical wells in alluvial formations are typically screened over the lower 33 to 50 percent of the saturated thickness of the aquifer to maximize the available drawdown above the screen and to minimize well inefficiency due to shorter screens.

The active wells range in age from 24 to 40 years old. In the City’s oldest wells (constructed in 1976), for the wells constructed in 1976, DWR-authorized maximum pumping rates higher than the recommended pumping rate for the wells based on screen design. At the time these vertical wells were originally constructed, limiting screen inflow velocity to less than 6 ft/min was not a widely accepted design practice, and in some cases was not practical with the well screen materials available. For all new wells, KDHE Public Water Supply System design standards generally require the 6 ft/min screen inflow velocity limitation. Table C-1 presents the known well construction details for the City’s vertical wells, both active and abandoned.

Current HCW well design recommendations allow for HCW screen inflow velocity to be less than 2 ft/min assuming 50 percent of the screen will be blocked by aquifer or gravel pack material. Design recommendations limit flow based on a maximum screen approach velocity (based on hydraulic conductivity) and a maximum axial flow velocity within the lateral below 5 ft/sec (preferably below 3.3 ft/sec). It is typically recommended that the pumping water level be maintained a minimum of ten feet above the centerline of the well laterals; however, because the City’s HCWs are shallow, therefore this recommendation can be reduced to five feet. Table C-2 presents known well construction details for the City’s HCWs.
<table>
<thead>
<tr>
<th>Well Number</th>
<th>Date Drilled</th>
<th>Date Plugged</th>
<th>Well Status</th>
<th>Depth to Bedrock (ft bgs)</th>
<th>Well Diameter (inches)</th>
<th>Bore Diameter (inches)</th>
<th>Length of Well Screen (feet)</th>
<th>Well Screen Type</th>
<th>Well Screen Slot Rate</th>
<th>Maximum Pumping Rate (gpm)</th>
<th>KSDWR Maximum Authorized Rate (gpm)</th>
<th>Approx. Elevation Ground Surface (ft msl)</th>
<th>Elevation Bottom of Well (ft msl)</th>
<th>Elevation Top of Screen (ft msl)</th>
<th>Original Saturated Thickness (feet)</th>
<th>Original Specific Capacity (gpm/ft)</th>
<th>Approximate Recent Non-Pumping Water Level Elevation (ft msl)</th>
</tr>
</thead>
<tbody>
<tr>
<td>VW 1</td>
<td>Apr-64</td>
<td>Aug-06</td>
<td>Plugged</td>
<td>66</td>
<td>16</td>
<td>NA</td>
<td>20</td>
<td>NA</td>
<td>NA</td>
<td>-</td>
<td>730</td>
<td>781</td>
<td>712.9</td>
<td>732.9</td>
<td>46.1</td>
<td>117.0</td>
<td>-</td>
</tr>
<tr>
<td>VW 2</td>
<td>Apr-64</td>
<td>Aug-06</td>
<td>Plugged</td>
<td>62</td>
<td>16</td>
<td>NA</td>
<td>20</td>
<td>NA</td>
<td>- 1.14</td>
<td>730</td>
<td>781</td>
<td>717.0</td>
<td>737.0</td>
<td>42.0</td>
<td>75.0</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>VW 3</td>
<td>Apr-64</td>
<td>Sep-91</td>
<td>Plugged</td>
<td>60.5</td>
<td>30</td>
<td>60</td>
<td>15</td>
<td>wirewrap</td>
<td>0.100</td>
<td>1013</td>
<td>730</td>
<td>783</td>
<td>722.5</td>
<td>737.5</td>
<td>32.1</td>
<td>107.2</td>
<td>740 to 750</td>
</tr>
<tr>
<td>VW 3R</td>
<td>Sep-91</td>
<td>-</td>
<td>Active</td>
<td>60.5</td>
<td>30</td>
<td>60</td>
<td>15</td>
<td>wirewrap</td>
<td>0.140</td>
<td>1013</td>
<td>730</td>
<td>783</td>
<td>722.5</td>
<td>737.5</td>
<td>32.1</td>
<td>107.2</td>
<td>740 to 750</td>
</tr>
<tr>
<td>VW 4</td>
<td>Apr-64</td>
<td>Jun-92</td>
<td>Plugged</td>
<td>55.5</td>
<td>16</td>
<td>NA</td>
<td>20</td>
<td>NA</td>
<td>- 0.10</td>
<td>775</td>
<td>780</td>
<td>721.7</td>
<td>741.7</td>
<td>37.3</td>
<td>59.2</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>VW 4R</td>
<td>Jun-92</td>
<td>-</td>
<td>Active</td>
<td>60</td>
<td>30</td>
<td>60</td>
<td>15</td>
<td>wirewrap</td>
<td>0.100</td>
<td>1013</td>
<td>730</td>
<td>780</td>
<td>721.0</td>
<td>736.0</td>
<td>26.0</td>
<td>136.7</td>
<td>744 to 748</td>
</tr>
<tr>
<td>VW 5</td>
<td>Aug-76</td>
<td>-</td>
<td>Active</td>
<td>64</td>
<td>26</td>
<td>50</td>
<td>20</td>
<td>shutter</td>
<td>7 gage</td>
<td>267</td>
<td>1045</td>
<td>781</td>
<td>719.0</td>
<td>739.0</td>
<td>36.3</td>
<td>147.0</td>
<td>743 to 750</td>
</tr>
<tr>
<td>VW 6</td>
<td>Aug-76</td>
<td>-</td>
<td>Active</td>
<td>65</td>
<td>26</td>
<td>50</td>
<td>25</td>
<td>shutter</td>
<td>7 gage</td>
<td>334</td>
<td>1035</td>
<td>780</td>
<td>714.0</td>
<td>739.0</td>
<td>40.2</td>
<td>177.0</td>
<td>742 to 746</td>
</tr>
<tr>
<td>VW 7</td>
<td>Oct-78</td>
<td>-</td>
<td>Active</td>
<td>54</td>
<td>30</td>
<td>60</td>
<td>23</td>
<td>wirewrap</td>
<td>0.100</td>
<td>1552</td>
<td>1005</td>
<td>782</td>
<td>727.1</td>
<td>750.1</td>
<td>26.3</td>
<td>92.3</td>
<td>747 to 751</td>
</tr>
<tr>
<td>VW 8</td>
<td>Oct-78</td>
<td>Aug-96</td>
<td>Plugged</td>
<td>52.5</td>
<td>30</td>
<td>60</td>
<td>20</td>
<td>wirewrap</td>
<td>0.100</td>
<td>1015</td>
<td>781</td>
<td>729.0</td>
<td>749.0</td>
<td>25.5</td>
<td>103.7</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>VW 9</td>
<td>Mar-81</td>
<td>-</td>
<td>Active</td>
<td>51</td>
<td>30</td>
<td>60</td>
<td>20</td>
<td>wirewrap</td>
<td>0.080</td>
<td>1169</td>
<td>830</td>
<td>781</td>
<td>726.5</td>
<td>748.5</td>
<td>24.8</td>
<td>92.4</td>
<td>747 to 751</td>
</tr>
<tr>
<td>VW 10</td>
<td>Feb-81</td>
<td>-</td>
<td>Active</td>
<td>63</td>
<td>30</td>
<td>60</td>
<td>25</td>
<td>wirewrap</td>
<td>0.080</td>
<td>1461</td>
<td>770</td>
<td>781</td>
<td>717.8</td>
<td>742.8</td>
<td>35.0</td>
<td>78.3</td>
<td>743 to 747</td>
</tr>
<tr>
<td>VW 11</td>
<td>Feb-81</td>
<td>-</td>
<td>Active</td>
<td>62</td>
<td>30</td>
<td>60</td>
<td>25</td>
<td>wirewrap</td>
<td>0.080</td>
<td>1461</td>
<td>800</td>
<td>780</td>
<td>717.8</td>
<td>742.8</td>
<td>34.0</td>
<td>116.5</td>
<td>741 to 751</td>
</tr>
</tbody>
</table>

1 Calculated for a screen entrance velocity of 6 ft/min. Assumes 50-percent blockage of the well screen. Assumes wirewrap screen in VW 3R and VW 4R.

2 The total combined maximum instantaneous rate for the vertical wells is 5,215 gpm.

3 Original VWs 1 through 4 were estimated from a static water level of 759 ft msl due to limited original records.

4 Based on high pumping periods and low flow river data collected by the City between 2011 and 2015.

NA = Not Available
| Well Number | Date Constructed | Well Status | Depth to Bedrock (ft) | Caisson Diameter (feet) | Number of Well Laterals | Number of Spare Ports | Well Screen Diameter (inches) | Well Screen Slot Size (feet) | Well Screen Type | Well Screen Slotted Velocity (gpm) | Maximum Pumping Rate Based on Screen Entrance Velocity (gpm) | Maximum Pumping Rate Based on Approach Velocity (gpm) | Maximum Pumping Rate Based on Axial Velocity (gpm) | Constructor's Original Design Rate (gpm) | KS DWR Maximum Authorized Rate (gpm) | Approx. Elevation Ground Surface (ft msl) | Elevation Centerline of Laterals (ft msl) | Elevation of Canyon Floor (ft msl) | Measuring Datum (ft msl) | Original Saturated Thickness (feet) |
|-------------|------------------|-------------|----------------------|------------------------|-------------------------|-----------------------|---------------------------|-------------------------------|----------------|-----------------------------------|---------------------------------------------|-----------------------------------------------|-----------------------------------------------|-------------------------------------|---------------------------------|---------------------------------|---------------------------------------|---------------------------------|
| HCW 1       | Feb-08 Active    | 69          | 16/12 / 20/00        | 2                      | 5                       | 2                     | 12                        | 365                           | wirewrap | 0.030 to 0.130                     | 9.049 / 0.030 to 0.130                     | 9.612                          | 8.813                            | 6.944                              | 10.500                          | 784                             | 722.6                           | 720.1                           | 797.6                             | 43.7                             |
| HCW 2       | Oct-02 Active    | 66          | 16/12 / 19/00        | 3                      | 2                       | 2                     | 12                        | 356                           | wirewrap | 0.008 to 0.120                     | 6.690 / 0.008 to 0.120                     | 6.413                          | 7.051                            | 3.472                              | 7.000                           | 780                             | 722.4                           | 739.4                           | 793.6                             | 33.4                             |
| HCW 3       | Oct-04 Active    | 60          | 16/12 / 19/00        | 4                      | 2                       | 2                     | 12                        | 765                           | wirewrap | 0.030 to 0.130                     | 7.174 / 0.030 to 0.130                     | 7.636                          | 7.051                            | 3.472                              | 7.000                           | 782                             | 728.4                           | 725.4                           | 794.1                             | 33.6                             |
| HCW 4       | Jul-05 Active    | 64          | 16/12 / 19/00        | 4                      | 1                       | 1                     | 12                        | 760                           | wirewrap | 0.030 to 0.130                     | 7.127 / 0.030 to 0.130                     | 5.735                          | 7.051                            | 3.472                              | 4.500                           | 783                             | 726.1                           | 723.3                           | 796.6                             | 40.5                             |

1 Calculated for a screen entrance velocity of 2 ft/min. Assumes 50 percent blockage of well screen.
2 Calculated for an approach velocity as a function of well site hydraulic conductivity.
3 Calculated for an axial velocity of 5 ft/s.
4 As allowed by water rights.
5 The measuring datum is the elevation from which water level measurements are made.
Appendix D
Well Performance Graphs
Vertical Wells
LEGEND
- SWL
- PWL
- GPM
- Top Of Screen
- Safe Pumping
- DWR Max Rate
- River - 31 Day Avg

Specific Capacity (gpm/ft)
0 25 50 75 100 125 150 175
Pumping Rate (gpm)
720 725 730 735 740 745 750 755 760 765 770
Water Level Elevation (MSL)
1/1/60 1/1/65 1/1/70 1/1/75 1/1/80 1/1/85 1/1/90 1/1/95 1/1/00 1/1/05 1/1/10 1/1/15 1/1/20

VW3: Historical Pumping Performance Data

DWR Max = 730 gpm
Minimum Pumping Level
Top of Screen
Well Redrilled

LEGEND
- Spec Cap
VW4: Historical Pumping Performance Data

LEGEND
- SWL
- PWL
- GPM
- Top Of Screen
- Safe Pumping
- DWR Max Rate
- River - 31 Day Avg

LEGEND
- Spec Cap

DWR Max = 775 gpm

Minimum Pumping Level

Top Of Screen

Well Redrilled
VW5: Historical Pumping Performance Data

- **Water Level Elevation (MSL)**
  - SWL
  - PWL
  - GPM
  - Top Of Screen
  - Safe Pumping
  - DWR Max Rate
  - River - 31 Day Avg

- **Pumping Rate (gpm)**
  - DWR Max = 1045 gpm

- **Specific Capacity (gpm/ft)**

- **Year**
  - 1/1/60 to 1/1/20

**Legend**:
- SWL: Safe Water Level
- PWL: Pump Water Level
- GPM: Great Pumping Level
- Top Of Screen
- Safe Pumping
- DWR Max Rate
- River - 31 Day Avg

- **Spec Cap**

- **DWR Max**
LEGEND

- SWL
- PWL
- GPM
- Top Of Screen
- Safe Pumping
- DWR Max Rate
- River - 31 Day Avg

LEGEND

- Spec Cap
VW10: Historical Pumping Performance Data

**LEGEND**
- SWL
- PWL
- GPM
- Top Of Screen
- Safe Pumping
- DWR Max Rate
- River - 31 Day Avg

- **DWR Max = 770 gpm**
- **Minimum Pumping Level**
- **Top of Screen**

**Minimum Pumping Level**
1/1/60 1/1/65 1/1/70 1/1/75 1/1/80 1/1/85 1/1/90 1/1/95 1/1/00 1/1/05 1/1/10 1/1/15 1/1/20

**Year**

**Specific Capacity (gpm/ft)**

- **0**
- **25**
- **50**
- **75**
- **100**
- **125**

**Pumping Rate (gpm)**

- **0**
- **100**
- **200**
- **300**
- **400**
- **500**
- **600**
- **700**
- **800**
- **900**

**Water Level Elevation (MSL)**

- **720**
- **725**
- **730**
- **735**
- **740**
- **745**
- **750**
- **755**
- **760**
- **765**
- **770**

**VW10: Historical Pumping Performance Data**

**Top of Screen**

- Minimum Pumping Level
- DWR Max = 770 gpm

**Legend**

- **SWL**
- **PWL**
- **GPM**
- **Top Of Screen**
- **Safe Pumping**
- **DWR Max Rate**
- **River - 31 Day Avg**

**Specific Capacity (gpm/ft)**

- **0**
- **25**
- **50**
- **75**
- **100**
- **125**

**Pumping Rate (gpm)**

- **0**
- **100**
- **200**
- **300**
- **400**
- **500**
- **600**
- **700**
- **800**
- **900**

**Water Level Elevation (MSL)**

- **720**
- **725**
- **730**
- **735**
- **740**
- **745**
- **750**
- **755**
- **760**
- **765**
- **770**

**Legend**

- **SWL**
- **PWL**
- **GPM**
- **Top Of Screen**
- **Safe Pumping**
- **DWR Max Rate**
- **River - 31 Day Avg**
VW11: Historical Pumping Performance Data

- Water Level Elevation (MSL)
- Pumping Rate (gpm)
- Specific Capacity (gpm/ft)

Legend:
- SWL
- PWL
- GPM
- Top Of Screen
- Safe Pumping
- DWR Max Rate
- River - 31 Day Avg

DWR Max = 800 gpm

Minimum Pumping Level
Top of Screen
Horizontal Collector Wells
CW1: Historical Pumping Performance Data

**LEGEND**
- River - SWL
- PWL
- GPM
- Top Of Screen
- Safe Pumping
- Design Rate

**Max Mechanical Design Rate (DWR Max = 10,500 gpm)**

**Legend**
- Spec Cap
- Spec Cap_60
- CW1 Temp
- River Temp

**Water Level Elevation (MSL)**
- Year

**Pumping Rate (gpm)**

**Temperature (F)**

**Specific Capacity (gpm/ft)**
CW3: Historical Pumping Performance Data

- **Max Mechanical Design Rate (DWR Max = 7,000 gpm)**
- **Minimum Pumping Level**
- **Well Lateral**
- **Spec Cap**
- **Spec Cap_60**
- **CW1 Temp**
- **CW1 Temp**
- **River Temp**

**Legend:**
- River - SWL
- PWL
- GPM
- Top Of Screen
- Safe Pumping
- Design Rate

**Year:**
- 1/1/96 to 1/1/16

**Water Level Elevation (MSL):**
- 720 to 770

**Temperature (F):**
- 20 to 100

**Pumping Rate (gpm):**
- 0 to 8000

**Specific Capacity (gpm/ft):**
- 0 to 500

**Graph Data:**
- Historical data showing variations in water level elevation, temperature, and pumping rates over time.
CW4: Historical Pumping Performance Data

Max Mechanical Design Rate (DWR Max = 4,500 gpm)

Minimum Pumping Level

Well Laterals

Spec Cap

Spec Cap_60

CW1 Temp

River Temp

River - SWL

PWL

GPM

Top Of Screen

Safe Pumping Design Rate

Max Mechanical Design Rate (DWR Max = 4,500 gpm)
Appendix E
Previous Studies on Additional Wells


Previous Studies on Additional Wells

Vertical Wells

Coker Property

In 2007, in conjunction with the Phase I collector well siting study, MKEC\(^1\) evaluated the Coker property southwest of HCW 4 for vertical well field development. The Study concluded that three wells at 300 gpm each or two wells at 400 gpm each was feasible from this property for a total yield of 800 – 900 gpm (1.15 to 1.30 MGD). The wells would require application for a new groundwater right, which would be backed up by KRWAD #1.

Existing Vertical Well Field, City-Owned Property

The area south of the existing vertical wells within area already owned by the City could be explored for vertical well production. The availability of test hole information in this area is currently limited and a test hole program and study in this area would be required to establish the reliable yield. Existing VWs 1 and 2 are in the deepest part of the aquifer and bedrock gets shallower to the south and east of these wells. The following is a summary of available information for this area based on Kansas Geological Survey (KGS) WWC5 database of drillers well records:

- Two monitoring wells were drilled for the City on City property south of VWs 1 and 2. One monitoring was constructed in 2002. The well construction record shows approximately 16 feet of saturated thickness in the productive material and a depth of approximately 50 feet to bedrock. A second monitoring well was constructed in 2003. The well construction record shows approximately 40 feet of saturated thickness and a depth of approximately 65 feet to bedrock. The productivity of a well located in aquifer similar to that described in the well records is expected to be approximately 400 to 800 gpm, under low river flow conditions.

- Two domestic wells were constructed south of 83rd Street at the Carriage Houses of Johnson County. Both water well construction records show the presence of clay and saturated thicknesses between 5 and 16 feet. Wells in this area are not expected to be good producers.

Based on this information, it is estimated that two or three wells could be installed in this area to yield a total of approximately 2 MGD. New wells in this area would require application for a new groundwater right, which would be backed up by KRWAD #1.

Penny Property

Land owned by David Penny between the City-owned property for HCWs 1, 2 and 3 and the City-owned property for the vertical well field and HCW 2 overlies the deepest part of the aquifer on the south side of the Kansas River. Given its proximity to the existing raw water transmission system, it would be an ideal candidate for development of additional water supply. The area was investigated in 1974 by Layne Western

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Company, Inc.  As a part of this investigation, test holes were completed; however, some test holes showed a considerable amount of fine sand which limits the potential yield. The Report estimates the yield at some test holes to be between 300 and 500 gpm. Therefore based on the Layne Report for a total of 5 wells, approximately 1,500 to 2,000 gpm (2.2 to 2.9 MGD) could be realized.

Since this Report was completed in 1974 and well design standards have changed since that time period, a revised firm yield of 3.5 MGD is estimated (five wells firm plus one stand-by) based on the test hole information from the Layne Report; this yield is based on a static water elevation of 750 feet, which is representative of a conservative Kansas River low flow scenario. An updated test hole program should be conducted for this area if it is to be developed for vertical wells. New wells in this area would require application for a new groundwater right, which would be backed up by KRWAD #1. In order for KRWAD #1 backing, wells would need to be within ½ mile of the Kansas River.

North of the Kansas River

The City of Olathe’s existing property north of the Kansas River (the Burning Tree Golf Course) and the Spring Trust property to the north of the golf course appear to be areas of good well production. As shown in Figure 1, it is the deepest area of the aquifer and therefore has the best potential for thicker amounts of saturated aquifer thickness. The City has not conducted test holes in this area to date and test hole information in this area is limited through KGS’s WWC5 database; however, MKEC did an overview of the area in 2010.

In order to be granted new groundwater rights in this area, which would be backed up by KRWAD #1, the wells would need to be within a quarter of a mile of the river’s edge, which limits the available area for wells to the southern portion of the Spring Trust property and the golf course property. The City has an existing irrigation well at the Burning Tree Golf Course, which as long as it remains active, prevents installation of other non-domestic wells within a 2-mile radius of the well.

Based on information that is available, it is estimated that four wells installed in this area could produce a firm capacity of 3.0 MGD; however, a comprehensive test hole program is needed to confirm this potential yield.

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2 Water Supply Investigation, Johnson County Rural Water District No. 6, De Soto, Kansas. Layne Western Company, Inc. February, 1974.

Olathe, Kansas

Water Master Plan Update

Figure 1. Previous HCW Study Locations and Proposed Additional HCWs

Notes:
1. Shallow bedrock would not support HCWs.
2. Little alluvium available to support HCWs.

*Note: Locations of existing wells and test sites are approximate.*
Horizontal Collector Wells

Previous studies have been conducted to assess the potential for collector well development along the Kansas River in the vicinity of the City’s existing wells. These studies have focused on areas south of the Kansas River west of De Soto (Phase I) and north of the Kansas River west of the City’s existing wells (Phase II). Figure 2 presents areas explored, general findings, and the locations of the currently planned horizontal collector wells.

In support of this discussion, the following documents were reviewed:

- City of Olathe Memorandum, from David E. Cox to Don Seifert, Dave Bries, and Jerald Robnett, November 23, 2004, RE: Applicability to Olathe of the WaterOne Hydrogeological Planning Study, Kansas River Valley Phase 1 and 2 Report and Recommendations for the City of Olathe Water Master Plan.
- Collector Well Siting Study along the Kansas River – Phase I, MKEC Engineering Consultants, Inc. and QSSI, June, 2006.
- Collector Well Siting Study along the Kansas River: Phase II – North Side, MKEC Engineering Consultants, Inc. and QSSI, April, 2007.
- Collector Well Siting Study along the Kansas River – Phase III, Decision Matrix Comparing Phase I and Phase II Investigation Results, MKEC Engineering Consultants, Inc. and QSSI, May, 2007.

The Phase II and Olander Reports studied the area that is planned for HCW 5 and 6. They are planned for what is referred to as the Stevens property, which is the Burning Tree Golf Course that was later purchased by the City for the wells. The Study estimated the primary well at the Stevens property (HCW 5) would be capable of 11.7 MGD during average summer conditions (11,100 cfs) and 10.2 MGD available during summer low flow conditions (876 cfs). A secondary well (HCW 6) was projected to have an aquifer yield of 6.4 MGD during average summer conditions.

Following completion of the Phase I and II siting studies, the City tasked Ranney Collector Wells with a technical peer review of the studies. The main point of disagreement between Ranney’s analysis and the Phase I and II studies was determination of the A-distance, which is the effective distance to the recharge boundary. Based on Ranney’s re-analysis, their projected yields at the Phase I and Phase II sites were 40 to 60-percent lower.

MKEC then responded to the comments from Ranney. Their response disputed Ranney’s comments regarding the A-distance, and provided updated projected yields using an alternative set of assumptions that was used by Burns & McDonnell in the siting and design of the City’s existing horizontal collector wells. Table 1 presents the projected well yields from the previous studies.
Olathe, Kansas
Water Master Plan Update
Figure 2. Previous Vertical Well Study Locations

[Map of Olathe, Kansas with various study locations and well fields indicated.]
Table 1. Previously Projected Horizontal Collector Well Yields

<table>
<thead>
<tr>
<th>Site</th>
<th>South of the Kansas River, West of De Soto</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mistele 1 Site</td>
<td>6.2</td>
</tr>
<tr>
<td>Mistele 2 Site</td>
<td>5.9</td>
</tr>
<tr>
<td>Mistele 3 Site</td>
<td>5.8</td>
</tr>
<tr>
<td>West Site</td>
<td>9.6</td>
</tr>
<tr>
<td>Central Site</td>
<td>3.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Site</th>
<th>North of the Kansas River, West of City’s Existing Well Field</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tinberg 1 Site</td>
<td>13.6</td>
</tr>
<tr>
<td>Tinberg 2 Site</td>
<td>3.6</td>
</tr>
<tr>
<td>Murray Site</td>
<td>2.7</td>
</tr>
<tr>
<td>Stevens 1 Site (HCW 5)</td>
<td>10.2</td>
</tr>
<tr>
<td>Stevens 2 Site (HCW 7)</td>
<td>5.5</td>
</tr>
<tr>
<td>Olander 1 Site (HCW 6)</td>
<td>5.4</td>
</tr>
</tbody>
</table>

1 Yields shown are based on a summer low flow river condition, laterals at 4 to 5 feet above bedrock, minimum water level at 10 feet above laterals, and 20-percent degradation in well yield with time.  
2 Yields shown are based on a summer low flow river condition, laterals at 7 feet above bedrock, minimum water level at 5 feet above laterals, and 20-percent degradation in well yield with time. This design criteria is the same as that used for the design of the City’s existing HCWs.

HDR reviewed the original siting studies for HCWs 1 through 4, along with initial performance testing at the HCWs and long-term performance of the HCWs. Based on these data, effective A-distance to recharge was back calculated using the Hantush and Papadopulous 1962 equation for flow of groundwater to a collector well. Table 2 presents the resulting values.

Table 2. Horizontal Collector Wells – Decline in Specific Capacity From Original

<table>
<thead>
<tr>
<th>Well</th>
<th>Year Built</th>
<th>Original Specific Capacity (gpm/ft)¹</th>
<th>Estimated Attainable Specific Capacity via Cleaning (gpm/ft)²</th>
<th>Estimated Percent Decline with Seasoning</th>
<th>New Condition Effective A-Distance (ft)³</th>
<th>Current Effective A-Distance (ft)³</th>
</tr>
</thead>
<tbody>
<tr>
<td>HCW 1</td>
<td>Feb 1998</td>
<td>753</td>
<td>310</td>
<td>59</td>
<td>120</td>
<td>265</td>
</tr>
<tr>
<td>HCW 2</td>
<td>Oct 2002</td>
<td>196</td>
<td>175</td>
<td>11</td>
<td>340</td>
<td>440</td>
</tr>
<tr>
<td>HCW 3</td>
<td>Oct 2004</td>
<td>355</td>
<td>235</td>
<td>34</td>
<td>255</td>
<td>630</td>
</tr>
<tr>
<td>HCW 4</td>
<td>Jul 2005</td>
<td>352</td>
<td>220</td>
<td>38</td>
<td>140</td>
<td>225</td>
</tr>
</tbody>
</table>

¹ From initial HCW performance testing before seasoning; specific capacity values have been corrected to 60 degrees Fahrenheit.  
² Estimate of specific capacity that can be recovered by cleaning of laterals, corrected to 60 degrees Fahrenheit. The difference between this specific capacity and the column to the left (Note 1) is the seasoning effect.  
³ A-Distance calculated from Hantush and Papadopulous, 1962 based on performance data.
Based on these values, it appears that HCWs initially perform like consultants were predicting from test well pumping tests, but that Ranney uses much larger initial A-distance to account for long-term yield after the collector well goes through its seasoning. Based on this, HDR suggests discounting new HCW predicted rates by 40 percent for this with initial short A-distances, and by 20 percent for those locations not expected to have a great connection to the streambed.

It is also recommended that the close 670 to 1,100-foot spacing between HCWs 1, 3, and 4 not be used at future well sites; or if they are, it should be recognized that this may limit available streambed for infiltration to each well and reduce collector well yield.

Of the potential future HCW sites evaluated on the north side of the river (see Figure 2), HDR focused on properties purchased by the City; the 53 acre golf course, and the 20 acre property immediately east of it. Between the two, they have approximately 5,000 feet of river front.

- The location on the golf course for HCW 5 (Stevens 1) is expected to be the best site.
- The second best location is expected to be where the deepest part of the bedrock channel intersects the river front around the border between the two properties. Construction of a replacement domestic well for the club house, located at the northeast corner of the parking lot confirmed that the 70-foot deep channel crosses the golf course property. Additional test holes would be required to determine this location. This location would be 1,300 to 1,500 feet downstream of HCW 5. This is a potential HCW 6
- The location on the golf course near the Wyandotte Street bridge (Stevens 2), has not had a test hole drilled but may be promising. Stevens 2 is approximately 1,100 feet upstream of HCW 5. This is a potential HCW 7.

HDR estimated yield for each of the three sites using the Hantush and Papadopulous equation, given The City’s design and operational criteria, and using aquifer information proposed by MKEC. Parameters are presented in Table 3.
### Table 3. Calculated Future Horizontal Collector Well Yields

<table>
<thead>
<tr>
<th>Parameter</th>
<th>HCW 5</th>
<th>HCW 6</th>
<th>HCW 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade Elevation</td>
<td>783.9</td>
<td>780</td>
<td>785</td>
</tr>
<tr>
<td>Bedrock Elevation</td>
<td>726</td>
<td>716</td>
<td>728</td>
</tr>
<tr>
<td>Lateral Centerline Elevation</td>
<td>734</td>
<td>724</td>
<td>736</td>
</tr>
<tr>
<td>K (gpd/ft²) @ 60</td>
<td>4,000</td>
<td>4,400</td>
<td>4,000</td>
</tr>
<tr>
<td>Summer Average Saturated Thick (ft)</td>
<td>36</td>
<td>45.5</td>
<td>30</td>
</tr>
<tr>
<td>A-dist</td>
<td>120</td>
<td>220/750¹</td>
<td>150</td>
</tr>
<tr>
<td>Average Lat length</td>
<td>175</td>
<td>175</td>
<td>175</td>
</tr>
<tr>
<td>No. Laterals Required</td>
<td>7 to 8</td>
<td>6 to 7</td>
<td>3 to 4</td>
</tr>
<tr>
<td>Summer Avg at 80% (MGD)²</td>
<td>13.7</td>
<td>12.3</td>
<td>6.5</td>
</tr>
<tr>
<td>Summer Low at 80% (MGD)²</td>
<td>11.6</td>
<td>6.3</td>
<td>5.1</td>
</tr>
<tr>
<td>Winter Avg at 80% (MGD)²</td>
<td>9.7</td>
<td>5.1</td>
<td>4.5</td>
</tr>
</tbody>
</table>

¹ A-distance is 220 feet during average summer flow, and 750 feet during low summer and average winter flow due to the sand bar.

² Yields shown are based on a summer low flow river condition, laterals at 7.5 feet above bedrock, minimum water level at 5 feet above laterals, and 20-percent degradation in well yield with time.
Table 4 presents the projected available well capacity after the capacity has degraded due to river bed clogging external to the well (can take 3 to 4 years as exhibited at HCW 1) and after the well has degraded an additional 20-percent due to internal clogging of the well (which can be restored with regular cleaning). The values presented in Table 4 reflect the long-term, reliable well yields and are used as the basis of comparison with the maximum day demands for planning purposes. After initial startup, the well will certainly be capable of producing much more than what is presented in Table 4; additionally, when river levels are average to high, additional capacity can be realized.

<table>
<thead>
<tr>
<th>Well Site</th>
<th>MKEC Projected Yield (MGD)</th>
<th>Projected Yield Degradation Due to River Bed Clogging</th>
<th>HDR Projected Yield (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stevens 1 Site (HCW 5)</td>
<td>11.6</td>
<td>40%</td>
<td>7.0</td>
</tr>
<tr>
<td>Olander 1 Site (HCW 6)</td>
<td>6.3</td>
<td>20%</td>
<td>5.0</td>
</tr>
<tr>
<td>Stevens 2 Site (HCW 7)</td>
<td>5.1</td>
<td>35%</td>
<td>3.3</td>
</tr>
<tr>
<td>Total</td>
<td>23</td>
<td>-</td>
<td>15.3</td>
</tr>
</tbody>
</table>

1 Yields shown are based on a summer low flow river condition, laterals at 7.5 feet above bedrock, minimum water level at 5 feet above laterals, and 20-percent degradation in well yield with time.

2 Yields shown are based on degradation with seasoning.
Appendix F
Hydraulic Model Information
Hydraulic Model Analysis Summary
Olathe Water Master Plan Update

September 2017
Olathe, KS
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Diurnal Patterns
EPS vs. SCADA Comparison Charts
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Fire Flow Contour Maps
Water Age Contour Maps
1 Introduction

As part of the Olathe Water Master Plan Update, HDR was tasked with creating a hydraulic model for the Olathe raw water system and drinking water distribution system using the InfoWater hydraulic model software by Innovyze. This summary report will discuss the model development, calibration, future scenarios and results obtained using the model.
2 Raw Water System Hydraulic Model

This section summarizes the development and calibration of the raw water system hydraulic model. The model was developed in Innovyze InfoWater Executive Suite 12.2 in 2016. The model is a steady-state application and includes horizontal collector wells, vertical wells, and transmission mains to the head of Water Treatment Plant No. 2.

2.1 Data Sources

Several data sources provided by the City were used to develop the hydraulic model:

- The ArcGIS feature class of water mains and related features were provided by the City of Olathe on December 2, 2015. The data with subtype of “Raw Water” was imported into the model. The data contained the nominal diameter, construction year, and material of the mains. Lines shown to be abandoned were removed from the data set.

- Transmission main record drawings were generally used for verifying line sizes and transmission main interconnections. The following record drawings were used:
  - Well Field Supply Line Project 2. Van Doren, Hazard, Stallings and Schnacke Engineers and Architects. 1964.

- Horizontal collector well record drawings were used to layout pump station piping and to verify transmission main interconnections. The following record drawings were used:
  - Water Treatment Plant No. 2 Expansion, Contract No. 4, Water Production Improvements. Burns & McDonnell. 2000. (Horizontal Collector Well No. 1.)

2.1 Model Network Development

Pipe network and junction information was imported into the model using the GIS information provided by the City. The model network tools were then used to verify the model piping network was complete and connected. After verifying network connectivity elevations were assigned to junction nodes using city-wide elevation contour information. Collector well and vertical well pump curve and piping information was then manually entered into the model using record information provided by City. Reservoir information was entered into the model to simulate well ground water elevations. Reservoirs were also placed at the WTP2 splitter box and Basin #2 to simulate the termination of the raw water system.
2.2 Model Calibration

Model calibration is the process of comparing model results to field observations, and if necessary, adjusting the model data describing the system until the model reasonably matches the measured system performance. This process gives the user confidence that the model was built correctly and the results provided by the model are adequate for decision-making; it also gives insight into the behavior and performance of the system.

SCADA data for the maximum week of 2014 was provided by the City for the purposes of calibration. Since the model is a steady-state model, data for August 23, 2014 at 9:00 AM was chosen for the calibration. This time period represents the week of maximum production of the well field in 2014.

The first step of model calibration was to calibrate the pipe system. Known flow rates from the wells were input to the model and the discharge pressures at the horizontal collector wells were compared to the SCADA data. This process resulted in changes to the C factors.

Once a reasonable match was obtained by adjusting C factors, the actual pump curves were input to the model and the discharge pressures at the horizontal wells were again compared to the SCADA data. The pump curves were then modified so that model pump flows and discharge pressures closely match the SCADA information.

2.1 Model Results

The firm pumping capacity of all existing collector and vertical wells exceed the hydrogeological capacity of the wells except for vertical wells VW 3R and VW 4R. Consequently all scenarios used hydrogeological limits for well pumping maximums except for the two vertical wells. See tables 4-11 through 4-14 in Section 4 of Water Master Plan Update for well pumping and hydrogeological capacity.

All future scenario maximum day demands exceed the existing well hydrogeological capacity, so a future HCW or river intake located on the north side of the Kansas River (west of HCW #4) was assumed for future scenarios. A future river crossing pipeline connecting the proposed future collector well/intake to the existing raw water 48-inch transmission main was assumed to have negligible head loss. Consequently the future collector well pump station was simulated as a negative demand at the far west end of the existing 48-inch raw water transmission main near HCW#4.

All future scenarios evaluated both the entire maximum day demand coming from a future river intake on the north side of the river with no existing wells running, as well as running the existing wells at maximum hydrogeological capacity and using the future horizontal well/intake to bring total supply up to maximum day. Both supply methods for future scenarios had similar results for the existing raw water transmission main system.

With no improvements to the existing system the higher flows increased pressures in system by approximately 10-20psi, depending on which future scenario is evaluated. About half of the pressure increase is from the smaller piping in the plant beyond the raw water flow meter (single 36-inch main). The existing pumps appear to still be in a reasonable location on their pump curves with the pressure increase, however additional or upsized piping inside the plant beyond the raw water meter is recommended at the same time a new horizontal collector well or river intake is constructed to minimize impact from the larger flows. No other improvements to the existing raw water transmission system are expected based on the future demand projections outlined in the Water Master Plan Update.
3 Water Distribution Hydraulic Model

This section summarizes the development, calibration and results of the drinking water distribution system hydraulic model. The model was developed using Innovyze InfoWater Executive Suite 12.2 in 2016. The model has steady-state and extended period simulation (EPS) applications and includes the Olathe water transmission/distribution system, pump stations, storage tanks and WTP2 high service pump stations.

3.1 Model Data Sources

Several data sources provided by the City were used to develop the hydraulic model:

- The ArcGIS feature class of water mains and related features were provided by the City of Olathe on December 2, 2015. The data contained the nominal diameter, construction year, and material of water mains. Lines shown to be abandoned were removed from the data set.

- Transmission main record drawings were generally used for verifying line sizes and transmission main interconnections.

- A previous InfoWater model completed by Carollo Engineering for an earlier water master plan was a starting point to import pump station and tank information.

- Record drawings provided by City were used to verify pump station and tank information.

3.2 Model Development

3.2.1 Pipe Network and Facilities

Pipe network and junction information was imported into the model using the GIS information provided by the City. The model network tools were then used to verify the model piping network was complete and connected. After verifying network connectivity, elevations were assigned to junction nodes using city-wide elevation contour information. A previous InfoWater model completed by Carollo Engineering for an earlier water master plan used as a starting point to import pump station and tank information. Record drawings provided by City were then used to verify pump station and tank information. Reservoir information was entered into the model to simulate clear well elevations at WTP2.

3.2.2 Baseline and Future Demands

Average day and maximum day system demands using population projections were inputted into the model. See table 3.8 of the Water Master Plan Update for average day and maximum day demands. Baseline demands for 2014 average day were distributed to model nodes using meter data provided by City. Calibration day demands were then determined by applying a global factor to the baseline demands and AMI data for large customers to get the system total demand to match calibration day usage. Future demands were distributed using future land use areas to approximate where to place demands. All demands include a factor for unmetered (unaccounted) water. Scenarios were created for years 2014 (baseline), 2020, 2025 and 2035.
3.2.3 Fire Protection Demands

Fire supply scenarios were evaluated for the various scenario years using a maximum day steady state application. The model evaluated the maximum available fire flow for nodes assuming a minimum 20psi pressure to be maintained in the distribution system. Fire supply contours were then generated so fire supply could be compared to fire hydrant locations.

3.2.4 Diurnal Demand Patterns

Diurnal patterns for the system were created using SCADA information from September and August of 2014. Diurnals were also created for the top 11 large customers. See diurnal figures at end of report.

3.3 Steady-State Model Calibration

A steady state model assuming 2014 average day demands was created to calibrate pipe network C factors using hydrant flow tests conducted by City.

3.3.1 Field Testing

The City conducted twelve fire hydrant tests at various locations throughout the City to use for steady state model calibration. The locations were selected to get a representative test result for different sections of the City. Field test results can be seen in table at end of this summary.

3.3.2 Calibration Results

Using the results of the fire hydrant testing, model pipe C factors were adjusted so that the model flows and pressures matched the field test flows and pressures as closely as possible. The C factors were adjusted system-wide based on pipe material and diameter. There were three locations that the modeled results did not match the field test very well. The first location was in the Southwest Pressure Zone, which is a closed system with no elevated storage, and it is possible that the Blackbob pump reaction time in field did not match the model. The second location is in the Main Pressure Zone and it is possible there may be a closed valve in the system. The third location is near the New Century meter location, which has a history of causing pressure fluctuations (this test was disregarded). All other model tests location matched field tests closely after the C factors were adjusted and the model pipe network was considered representative of the existing distribution system to allow proceeding to the EPS calibration phase. Final C factor used can be seen in table at end of this summary.

3.4 Extended Period Simulation Calibration

After the C factors for the pipe network were adjust in the steady state calibration, a 24-hour extended period simulation calibration was conducted using SCADA information for a high use day of August 25, 2014.

3.4.1 Calibration Data

SCADA information used for calibration data was supplied by City for all tanks, pump stations and the two "pressure cans" that allow flow between the Main Pressure Zone and Southwest Pressure Zone. System usage and AMI data was used to create diurnal curves for the calibration day.
3.4.2 Model Controls

Controls for all pump stations were set up to mimic pump station outflows and tank inflows for the calibration day. PRV settings at the pressure cans were also adjusted to match pressures at those locations. Initial tank levels were set to match initial tank levels at the beginning of the calibration day.

3.4.3 Calibration Results

The Southwest Pressure Zone does not have elevated storage, which makes it a closed system. The Main Pressure Zone does have temporary elevated storage when the Blackbob fill valve is fully open and allows two-way flow of water into and out of the Blackbob tank. In situations were the Blackbob fill valve is closed the Main Pressure Zone becomes a closed system with no elevated storage. Since parts or all of the system is a closed system without elevated storage, it makes the model difficult to calibrate and stabilize. This is reflected in the operation of the system which requires significant oversight of pump controls to maintain system pressures and flows. Significant pump controls had to be placed in model to mimic calibration day operations that were not necessarily applicable to any standard operation day scenario.

Initial calibration runs had pump speed adjusted so model flows matched SCADA flows and tank levels were then used to make adjustments to model. Once the model tank levels matched SCADA the pumps were then set to match SCADA pump speeds to see if model pump flows matched SCADA flows. Pump flows that did not match closely indicate that the model pump curve does not match field conditions. Based on the analysis the WTP2 South High Service and Renner pump curves did not have a good match. The City performed additional pump testing on Renner Pump Station to obtain a more accurate pump curve that was entered in the model. City was not able to do testing for the WTP2 South High Service pumps so the curves from the 2011 model were selected since they matched SCADA results better. The Blackbob Pump Station pump speed SCADA information did not seem to be reporting accurately on calibration day so no evaluation was made of Blackbob pump curves.

3.5 Model Scenarios

Scenarios were created for years 2014 (baseline), 2020, 2025 and 2035. Each year had an EPS and steady-state scenario for average day and maximum day. Water quality scenarios were analyzed using an average day condition for a 15 day period. Fire flow was also analyzed for each year using a stead-state maximum day condition.

3.5.1 Pressure Evaluation

Maximum day EPS scenarios were used for each of the planning years to determine if pressures can be maintained above Olathe’s minimum target pressure of 40 psi. In general most areas of the city were able to maintain the minimum 40 psi pressure except the far southwest portion of the city. This was due to the lack of transmission mains in the area and three large wholesale customers (RWD#6, RWD#7 and New Century) also located in the same area. Consequently, it is recommended that a new transmission main be extended south from the Hedge Lane Pump Station, with the long term goal of eventually connecting the proposed transmission main to the Blackbob Station discharge transmission main in the southeast part of the city. In addition to the proposed Hedge Lane-to-Blackbob main, a new 16-inch main is also proposed along old Hwy 56 from the
proposed transmission main to the southwest to serve the area and help supply New Century and RWD#6.

As discussed previously the lack of elevated storage within the system requires significant oversight of pump controls to maintain system pressures and flows. It is recommended that elevated storage be added to the system. If the system is converted into a single pressure zone and the proposed transmission main from Hedge Lane to Blackbob is constructed then a single elevated storage tank with an overflow meeting the Blackbob 2 tank overflow level would allow both tanks to operate as elevated storage facilities and still allow occasional drawdown of Blackbob 2 for water quality improvement. However if two pressure zones are maintained then at least one elevated tank would be required for each zone.

AMI data indicates New Century takes water at excessively high rates for short time periods and causes high pressure fluctuations in the area. City staff indicates the meter at that location has a pressure sustaining valve, however the AMI data indicates it is either not set correctly or it is malfunctioning. The sustaining valve should be repaired or replaced so that pressure fluctuations are minimized while still allowing New Century to received water at contracted volumes.

3.5.2 Fire Flow Evaluation

Fire supply scenarios were evaluated for the various scenario years using a maximum day steady state application. The model evaluated the maximum available fire flow for nodes assuming a minimum 20psi pressure to be maintained in the distribution system. Fire supply contours were then generated so fire supply could be compared to fire hydrant locations. Olathe fire supply targets are 1000 gpm for residential areas and 3500 gpm for commercial and industrial areas.

The existing system is not able to provide required fire flow to the far southwest portion of the distribution system near the executive airport. However if the proposed capital improvements mentioned in 3.5.1 are made then the system can generally provide required fire flows.

Some isolated locations on dead end mains did have limited fire flow that was near or below targets for all scenarios regardless of system improvements due to the diameter and length of the dead end main. Those locations would require upsizing of the dead end main to increase fire flow capacity. See fire flow supply maps.

3.5.3 Water Age Evaluation

A 15-day water age scenario was run for each of the planning years. Olathe maximum water age target was 10 days (240 hours). In general most areas were able to meet the target if all of the tanks are regularly cycled. There were some isolated dead end mains that did not have demand loaded on them in the model that exceeded the limit. Those locations would require additional review on a case by case level to determine if the dead end main requires flushing or other improvements to improve water age.

Areas near the boundary between the Southeast and Main Pressure Zones had higher water age values. Consolidating the system into a single pressure zone improved water ages for the entire system. See water age maps for more information.
<table>
<thead>
<tr>
<th>Test #</th>
<th>Time</th>
<th>Pressure Hydrant</th>
<th>Static Pressure (psi)</th>
<th>Residual Pressure (psi)</th>
<th>Flow (gpm)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>10918</td>
<td>65</td>
<td>55</td>
<td>10</td>
<td>7</td>
</tr>
<tr>
<td>2</td>
<td>3:09 pm</td>
<td>9334</td>
<td>56</td>
<td>40</td>
<td>31</td>
<td>16</td>
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<tr>
<td>3</td>
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<td>11:23 pm</td>
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<tr>
<td>5</td>
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<td>11471</td>
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<td>82</td>
<td>77</td>
<td>12</td>
</tr>
<tr>
<td>6</td>
<td>10:18 am</td>
<td>8690</td>
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<td>58</td>
<td>30</td>
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<tr>
<td>7</td>
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<td>7538</td>
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### Calibrated Model C Factors

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<th>Material</th>
<th>Size</th>
<th>C Factor</th>
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</thead>
<tbody>
<tr>
<td>DIP</td>
<td>6&quot; &amp; Smaller</td>
<td>100</td>
</tr>
<tr>
<td>DIP</td>
<td>8&quot; - 20&quot;</td>
<td>110</td>
</tr>
<tr>
<td>DIP</td>
<td>24&quot; &amp; Larger</td>
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<tr>
<td>CIP</td>
<td>6&quot; &amp; Smaller</td>
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<tr>
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<td>130</td>
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<tr>
<td>HDPE</td>
<td>All</td>
<td>130</td>
</tr>
<tr>
<td>Unknown</td>
<td>All</td>
<td>100</td>
</tr>
</tbody>
</table>
Appendix G
Soil Corrosivity Evaluation
Soil Corrosivity Evaluation

Water Master Plan Update
Olathe, Kansas
DRAFT REPORT
July, 2016

HDR Project No. 10026759
Executive Summary

HDR Engineering, Inc. (HDR) performed a soil corrosivity evaluation throughout the limits of the system. Field testing was completed on April 22, 2016, for the Water Master Plan Update project at 17 discrete locations throughout Olathe’s water service area. The purpose of these tests was to determine the level of corrosivity of the soils and risk of damage and premature failure of underground metallic and non-metallic utility piping. It is intended to utilize the data from this study to help the City plan for corrosion control, waterline replacement or rehabilitation planning.

The scope of this study is limited to determination of soil corrosivity and general corrosion control recommendations for both existing and proposed piping systems. HDR understands that the typical depth of cover in the system is four feet and that there is no construction proposed at this time. The pipe material in the system consists of 70 percent ductile iron, 26 percent cast iron, 1 percent steel, and the remaining 3 percent is non-metallic.

Based on the information provided and the 0-15 foot average soil resistivity data collected, the soil within the area studied is classified as severely corrosive to ferrous metals. There is a 40 percent probability of a pipeline within the testing limits experiencing severely corrosive soil, 43 percent for corrosive soil, and the remaining 17 percent for moderately corrosive soil.

Looking only at the average pipe depth, the soil within the water mains limits is classified as corrosive to ferrous metals. There is a 36 percent probability that the pipeline is experiencing severely corrosive soil, 44 percent for corrosive soils, and the remaining 20 percent is for moderately corrosive soil.

Eight of the 17 test sites were shown to have severely corrosive soils. Contour maps depicting the soil corrosivity across the City were generated and are incorporated in the Appendix.

Soil corrosivity conclusions are based on interpolation between test points. Site specific testing should be performed for new construction.

The soil across the entire system is classified as moderately to severely corrosive and does necessitate corrosion control measures. In addition, cathodic protection should be installed and applied to metallic pipelines per NACE SP0169 in order to protect the existing infrastructure. It is recommended that pipeline joints should be welded or bonded and test stations installed for corrosion monitoring and the application of cathodic protection.
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Introduction

Field tests were completed for the Water Master Plan Update project on April 22, 2016. The purpose of these tests was to determine if the soils might have deleterious effects on underground metallic and non metallic utility piping. HDR Engineering, Inc. (HDR) worked with the City to select sites that were:

- Spatially representative of the system
- Representative of areas that have historically experienced a high density of main breaks, particularly where the main breaks have been attributed to corrosion
- Representative of areas that contain primarily metallic pipe

Two sites, Sites 3, 6, 9 and 15 represent areas that have not experienced main breaks for comparison against sites that have.

It is intended to utilize the data from this study to evaluate the probability of failure in the existing water mains system. The pipe in the system is 70 percent ductile iron, 26 percent cast iron, 1 percent steel, and the remaining 3 percent is non-metallic. HDR understands that the typical depth of cover in the piping is four feet and that there is no specific construction proposed at this time. The water table depth was not provided; therefore, its effect on site corrosivity could not be accounted for in this analysis and report.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for capital improvement and rehabilitation projects within the water system. HDR’s recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction.

Test and Data Analysis Procedures

The field test procedures and data analysis methods described below were followed while executing this contract.

Wenner Four Pin Field Resistivity

The electrical resistivity of the soil was measured in place at 17 locations using the Wenner Four Pin Method per the ASTM G57 test method depicted in Figure 1 below. This procedure gives the average resistivity to a depth equal to the spacing between the pins. Approximate pin spacings of 2.5, 5, 7.5, 10, and 15 feet were used so that variations with depth could be evaluated. Strata as resistivities were calculated from resistance data using the Barnes Procedure. The test results are shown in the enclosed Table 1 of Appendix A.
Geographical Information System (GIS) Survey

GPS coordinates were obtained at the test locations with a Trimble Geo XH, subfoot Global Positioning System (GPS) (See Figure 3). HDR used ESRI ArcMap10.1 GIS to create a database containing soil resistivity data and field notes linked to the appropriate test location. This report includes a shapefile (.shp) with soil testing data attributes that can be incorporated into the City’s GIS system. The database is enclosed in the accompanying compact disc. Maps of the water system area with soil corrosivity categories overlayed can be found in Appendix B.
A major factor in determining soil corrosivity is electrical resistivity, which is a measure of the soil’s resistance to the flow of electrical current. It is often reported in units of ohm-centimeter (Ω-cm). Corrosion of buried metal is an electrochemical process in which the rate of metal loss corrosion is directly proportional to the flow of direct electrical current from the metal into the soil or water. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. A correlation between electrical resistivity and corrosivity toward ferrous metals is:\(^1\)

<table>
<thead>
<tr>
<th>Soil Resistivity in Ω-cm</th>
<th>Corrosivity Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Greater than 10,000</td>
<td>Mildly Corrosive</td>
</tr>
<tr>
<td>2,001 to 10,000</td>
<td>Moderately Corrosive</td>
</tr>
<tr>
<td>1,001 to 2,000</td>
<td>Corrosive</td>
</tr>
<tr>
<td>Below 1,000</td>
<td>Severely Corrosive</td>
</tr>
</tbody>
</table>

Factors that influence corrosivity towards metals are moisture level, concentration of soluble salts (notably chloride ions), acidity (pH), soil types, aeration, anaerobic conditions, and site drainage. For comparison, seawater which rapidly corrodes carbon steel has a resistivity in the range of 20 to 40 Ω-cm.

Wenner Four Pin Testing

Average Wenner 4-pin resistivities resulting from the April, 2016 testing were in the moderately to severely corrosive categories. Severely corrosive soils were found in sites 5, 7, 8, 11, 12, 14, 16, and 17. Statistically, 47 percent of the average resistivities from 0-15 feet were in the severely corrosive category, 41 percent were in the corrosive category, and the remaining 12 percent were in the moderately corrosive category. The mean of measurements at this depth range was 1,357 ohm–cm. Average resistivities increased with increasing depth in testing locations 1, 2, 3, 6, and 11; decreased with increasing depth in testing locations 12 and 16; and demonstrated no correlation between soil resistivity and soil depth in testing locations 4, 5, 7, 8, 9, 10, 13, 14, 15, and 17. A map of the approximate test locations is provided above in Figure 4.

After verifying that the Wenner Four Pin average point resistivity data, from 0 to 15 feet, was normally distributed, normal distribution probabilities were calculated. Based on this normal distribution, the probability of a pipeline within the testing limits experiencing severely corrosive soil is 40 percent, 43 percent for corrosive soil, and the remaining 17 percent for moderately corrosive soil.

Stratum resistivities at normal pipe depths were predominantly in the corrosive category as shown in Table 1 below. The mean of measurements at this depth range was 1,435 ohm–cm.
Table 1- Soil at Pipe Depth Corrosivity Category

<table>
<thead>
<tr>
<th>Corrosive Category</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Severely corrosive</td>
<td>35</td>
</tr>
<tr>
<td>Corrosive</td>
<td>41</td>
</tr>
<tr>
<td>Moderately Corrosive</td>
<td>24</td>
</tr>
</tbody>
</table>

Statistically, 35 percent of the resistivities at normal pipe depth were in the severely corrosive category, 41 percent were in the corrosive category, and the remaining 24 percent were in the moderately corrosive category.

After transforming the stratum resistivity at pipe depth data into an approximately normally distributed dataset, normal distribution probabilities were calculated. The probability of the pipeline experiencing severely corrosive soils is 36 percent, 44 percent for corrosive soil, and the remaining 20 percent for moderately corrosive soil.

This soil is classified as severely corrosive to ferrous metals and corrosion control for buried pipelines should be incorporated during rehabilitation and new construction projects.

Geographic Information System (GIS) Survey

Using the data collected from the soil resistivity testing, a contour was mapped for the average corrosivity of the soil at 15 feet below grade. Contour maps were also created for the soil resistivity at the average pipe depth between 2.5 and 5, and the three deeper strata. These maps may be used to help identify areas where corrosion was more likely to occur. These isopleth maps are provided in Appendix B.

Conclusion

Based on the information provided and the data collected, the soil within the area studied is classified as severely corrosive to ferrous metals at a depth of 0-15 feet. There is a 40% probability of pipelines within the area studied being in contact with severely corrosive soil, 43% for corrosive soil, and the remaining 17% in moderately corrosive soil.

The soil at pipe depth within the water mains limits is classified as corrosive to ferrous metals. There is a 36 percent probability that the pipeline is experiencing severely corrosive soil, 44 percent for corrosive soils, and the remaining 20 percent is for moderately corrosive soil.

These conclusions are based on interpolation between test points. Site specific testing along pipe routes is recommended for new construction.

The severely corrosive soils were found in sites 5, 7, 8, 11, 12, 14, 16, and 17.

Despite some variance, the soil across the entire system is classified as severely corrosive. It is recommended that pipeline joints be welded or bonded and test stations installed for corrosion monitoring and the application of cathodic protection. In addition, cathodic protection should be installed and applied to steel, cast and ductile iron pipelines per NACE SP0169 in order to protect the existing infrastructure.
Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict with precision. What is more practical than trying to predict corrosion is to employ corrosion control methods that are proven to extend the service life of facilities that would otherwise be subject to significant corrosion and premature failure.

The following recommendations apply to both existing and proposed piping systems. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement all the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other mechanical type joints should be bonded for electrical continuity. Pipeline electrical continuity is necessary for corrosion monitoring and cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:
   
   a. At each end of the pipeline.
   
   b. At each end of all casings.
   
   c. On each side of major highways, river crossings and other major obstructions along the route.
   
   d. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per NACE SP0286 from:
   
   a. Dissimilar metals including copper electrical grounding
   
   b. Dissimilarly coated piping (cement-mortar vs. dielectric).
   
   c. Above ground steel pipe.

4. Apply a suitable dielectric coating intended for underground use such as:
   
   a. Polyurethane per AWWA C222 or
   
   b. Extruded polyethylene per AWWA C215 or
   
   c. A tape coating system per AWWA C214 or
d. Hot applied coal tar enamel per AWWA C203 or

e. Fusion bonded epoxy per AWWA C213.

5. Apply cathodic protection to steel piping as per NACE SP0169.

Iron Pipe

Implement all the following measures:

1. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:
   a. At each end of the pipeline.
   b. At each end of any casings.
   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Apply a suitable coating intended for underground use such as:
   a. Polyethylene encasement per AWWA C105; or
   b. Epoxy coating; or
   c. Polyurethane; or
   d. Wax tape.

   NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

5. Apply cathodic protection to cast and ductile iron piping as per NACE SP0169.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.

2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.
All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Closure

The analysis and recommendations presented in this report are based upon data obtained from the field tests conducted on April 22, 2016. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR’s services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.
Appendix A. Soil Resistivity Field Tests
Table 1 - Soil Resistivity Field Tests

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DEPTH (feet)</th>
<th>MEASURED RESISTANCE (ohms)</th>
<th>AVERAGE RESISTIVITY TO DEPTH (ohm-cm)</th>
<th>STRATUM RESISTIVITY (ohm-cm)</th>
</tr>
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<tbody>
<tr>
<td>SITE 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>(38.88243, -94.83769)</td>
<td>2.5</td>
<td>2.5</td>
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<td>1,240</td>
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<td>5.0</td>
<td>1.7</td>
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<tr>
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<td>7.5</td>
<td>1.6</td>
<td></td>
<td>27,716</td>
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<td></td>
<td>10</td>
<td>1.4</td>
<td></td>
<td>5,293</td>
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<tr>
<td></td>
<td>15</td>
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<td></td>
<td>14,091</td>
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<tr>
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<td>5.0</td>
<td>0.76</td>
<td></td>
<td>770</td>
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<tr>
<td>Across 900 Sheridan</td>
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<td>0.58</td>
<td></td>
<td>1,224</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>0.47</td>
<td></td>
<td>1,239</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>0.42</td>
<td></td>
<td>3,948</td>
</tr>
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<td>SITE 3</td>
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<td>(38.94088, -94.88531)</td>
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<td>2.1</td>
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<tr>
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<td></td>
<td>10</td>
<td>0.67</td>
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<td>1,355</td>
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<tr>
<td></td>
<td>15</td>
<td>0.55</td>
<td></td>
<td>3,071</td>
</tr>
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Table 1 - Soil Resistivity Field Tests

**OLATHE, KS**
**WATER MASTER PLAN UPDATE**
**HDR #10026759**
**22-Apr-16**

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DEPTH (feet)</th>
<th>MEASURED RESISTANCE (ohms)</th>
<th>AVERAGE RESISTIVITY TO DEPTH (ohm-cm)</th>
<th>STRATUM RESISTIVITY (ohm-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SITE 4</td>
<td>2.5</td>
<td>4.3</td>
<td>2,155</td>
<td>2,155</td>
</tr>
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<td></td>
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<td>2,520</td>
<td>3,034</td>
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<td></td>
<td>7.5</td>
<td>1.4</td>
<td>2,055</td>
<td>1,501</td>
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<td>10</td>
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<td>1,440</td>
<td>759</td>
</tr>
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<td></td>
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<td>644</td>
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<td>2.0</td>
<td>980</td>
<td>980</td>
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<td>0.78</td>
<td>780</td>
<td>648</td>
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<td></td>
<td>7.5</td>
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<td>885</td>
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<td></td>
<td>15</td>
<td>0.32</td>
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</tr>
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<td>1,925</td>
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<tr>
<td></td>
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<td>2,745</td>
<td>6,497</td>
</tr>
<tr>
<td></td>
<td>10</td>
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<td>2,820</td>
<td>3,072</td>
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<td></td>
<td>15</td>
<td>1.0</td>
<td>3,000</td>
<td>3,439</td>
</tr>
</tbody>
</table>

**CORROSIVITY LEGEND (FERROUS METALS)**
- **Mildly**
- **Moderately**
- **Corrosive**
- **Severely**
### Table 1 - Soil Resistivity Field Tests

**OLATHE, KS**  
**WATER MASTER PLAN UPDATE**  
**HDR #10026759**  
**22-Apr-16**

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DEPTH (feet)</th>
<th>MEASURED RESISTANCE (ohms)</th>
<th>AVERAGE RESISTIVITY TO DEPTH (ohm-cm)</th>
<th>STRATUM RESISTIVITY (ohm-cm)</th>
</tr>
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<tr>
<td>SITE 7</td>
<td>2.5</td>
<td>1.7</td>
<td>855</td>
<td>855</td>
</tr>
<tr>
<td></td>
<td>(38.86403, -94.77970)</td>
<td></td>
<td></td>
<td>637</td>
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<tr>
<td></td>
<td>5.0</td>
<td>0.73</td>
<td>730</td>
<td>554</td>
</tr>
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<td></td>
<td>7.5</td>
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<td>660</td>
<td>525</td>
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<td></td>
<td>10</td>
<td>0.31</td>
<td>620</td>
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<td></td>
<td>15</td>
<td>0.28</td>
<td>840</td>
<td>2,893</td>
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<tr>
<td>SITE 8</td>
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<td>2.4</td>
<td>1,195</td>
<td>1,195</td>
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<tr>
<td></td>
<td>(38.88946, -94.81611)</td>
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<td></td>
<td>660</td>
</tr>
<tr>
<td></td>
<td>5.0</td>
<td>0.85</td>
<td>850</td>
<td>670</td>
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<td></td>
<td>7.5</td>
<td>0.52</td>
<td>780</td>
<td>867</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>0.40</td>
<td>800</td>
<td>933</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>0.28</td>
<td>840</td>
<td></td>
</tr>
<tr>
<td>SITE 9</td>
<td>2.5</td>
<td>3.9</td>
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<td>1,960</td>
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<td></td>
<td>(38.88499, -94.76209)</td>
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<td>10</td>
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<tr>
<td></td>
<td>15</td>
<td>0.50</td>
<td>1,500</td>
<td></td>
</tr>
</tbody>
</table>

**CORROSIVITY LEGEND (FERROUS METALS)**  
- Green: Mildly Corrosive  
- Yellow: Moderately Corrosive  
- Red: Corrosive  
- Black: Severely Corrosive
## Table 1 - Soil Resistivity Field Tests

**OLATHE, KS**  
**WATER MASTER PLAN UPDATE**  
**HDR #10026759**  
**22-Apr-16**

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DEPTH (feet)</th>
<th>MEASURED RESISTANCE (ohms)</th>
<th>AVERAGE RESISTIVITY TO DEPTH (ohm-cm)</th>
<th>STRATUM RESISTIVITY (ohm-cm)</th>
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<td>1,230</td>
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**CORROSIVITY LEGEND (FERROUS METALS)**

- Green: Mildly Corrosive  
- Yellow: Moderately Corrosive  
- Orange: Corrosive  
- Black: Severely Corrosive

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431 West Baseline Road · Claremont, CA 91711  
Phone: 909.626.0967 · Fax: 909.626.3316  
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Table 1 - Soil Resistivity Field Tests

**OLATHE, KS**
**WATER MASTER PLAN UPDATE**
**HDR #10026759**
**22-Apr-16**

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DEPTH (feet)</th>
<th>MEASURED RESISTANCE (ohms)</th>
<th>AVERAGE RESISTIVITY TO DEPTH (ohm-cm)</th>
<th>STRATUM RESISTIVITY (ohm-cm)</th>
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<tbody>
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<td>0.49</td>
<td>1,470</td>
<td>2,091</td>
</tr>
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</table>
### Table 1 - Soil Resistivity Field Tests

**OLATHE, KS**
**WATER MASTER PLAN UPDATE**
**HDR #10026759**
**22-Apr-16**

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DEPTH (feet)</th>
<th>MEASURED RESISTANCE (ohms)</th>
<th>AVERAGE RESISTIVITY TO DEPTH (ohm-cm)</th>
<th>STRATUM RESISTIVITY (ohm-cm)</th>
</tr>
</thead>
</table>
| **SITE 16**  
(38.86913, -94.82458) | 2.5  | 6.0 | 3,000 | 3,000 |
| | 5.0  | 1.6 | 1,590 | 1,082 |
| | 7.5  | 0.60 | 900 | 482 |
| | 10  | 0.26 | 520 | 229 |
| | 15  | 0.12 | 360 | 223 |
| **SITE 17**  
(38.87110, -94.79990) | 2.5  | 1.5 | 740 | 740 |
| | 5.0  | 0.72 | 720 | 701 |
| | 7.5  | 0.55 | 825 | 1,165 |
| | 10  | 0.41 | 820 | 805 |
| | 15  | 0.33 | 990 | 1,691 |

**CORROSIVITY LEGEND (FERROUS METALS)**
- **Green**: Mildly
- **Yellow**: Moderately
- **Red**: Corrosive
- **Black**: Severely

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Appendix B. Soil Corrosivity Contour Maps
Figure 1
City of Olathe - Water Master Plan
Soil Corrosivity Evaluation

Soil Corrosivity Map (Based on average resistivity 2.5' - 5' below surface)
* Corrosivity of areas between test sites is interpolated.

2.5' - 5' Stratum Resistivity
- Highly Corrosive
- Corrosive
- Moderately Corrosive
- Mildly Corrosive

Soil Testing Sites

Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), MapmyIndia, © OpenStreetMap contributors, and the GIS User Community
Figure 2
City of Olathe - Water Master Plan
Soil Corrosivity Evaluation

Soil Corrosivity Map (Based on average resistivity 5' - 7.5' below surface)
* Corrosivity of areas between test sites is interpolated.

5' - 7.5' Stratum Resistivity
- Highly Corrosive
- Corrosive
- Moderately Corrosive
- Mildly Corrosive

Soil Testing Sites
Figure 3
City of Olathe - Water Master Plan
Soil Corrosivity Evaluation

Soil Corrosivity Map (Based on average resistivity 7.5’ - 10’ below surface)

* Corrosivity of areas between test sites is interpolated.
Soil Corrosivity Map (Based on average resistivity 10' - 15' below surface)

* Corrosivity of areas between test sites is interpolated.
Figure 5
City of Olathe - Water Master Plan
Soil Corrosivity Evaluation

Soil Corrosivity Map (Based on average resistivity to 15' below surface)
* Corrosivity of areas between test sites is interpolated.

Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), MapmyIndia, © OpenStreetMap contributors, and the GIS User Community