

## City of Norman, Oklahoma

## Update Distribution System Modeling



March 2018

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## List of Abbreviations

| APAI | Alan Plummer Associates, Inc. |
| :--- | :--- |
| AWWA | American Water Works Association |
| CIP | capital improvement plan |
| City | City of Norman, Oklahoma |
| DEQ | Oklahoma Department of Environmental Quality |
| DIP | ductile iron pipe |
| EPS | extended period simulation |
| EST | elevated storage tank |
| ft | feet |
| gal/day | gallons per day |
| gpcd | gallons per capita per day |
| gpm | gallons per minute |
| GPSFD | gallons per square foot of building area per day |
| hp | horsepower |
| in | inch |
| LF | linear foot |
| MFU | multi-family housing unit |
| MG | million gallons |
| MGD | million gallons per day |
| MDS | Main Distribution System |
| NUA | Norman Utilities Authority |
| OKC | Oklahoma City |
| OPCC | opinion of probable construction cost |
| PS | pump station |
| psi | pounds per square inch |
| PZ | Upper Pressure Zone |
| SA | service area |
| SCADA | supervisory control and data acquisition |
| SFU | single-family housing unit |
| OU | University of Oklahoma |
| VFD | variable frequency drive |
| WDM | water distribution model |
| WTP | water treatment plant |

## Executive Summary

Norman Utilities Authority (NUA) provides water and sewer service to the City of Norman, located in Cleveland County in central Oklahoma. NUA last updated its water distribution system model and master plan in 2003. NUA contracted with Alan Plummer Associates, Inc. to provide a Water Distribution Model Update.

The water distribution model for NUA was updated by utilizing current system GIS data, operational pumping controls and as-built facility data, and historical daily pumping and monthly billing data. A calibrated hydraulic model was used to aid in the analyses of the existing water system and recommendation of proposed improvements.

After calibrating the existing system, future demands were placed in the model based on growth projections in the 2025 Land Use Plan. The future model also includes 2 MGD of additional groundwater supply from a well expansion project that NUA is currently evaluating. The performance of NUA's distribution system under a future max day scenario was then evaluated using criteria for minimum water pressure, unit headloss through pipelines, and available fire flow from hydrants around the City. A list of Capital Improvement Plan (CIP) projects was developed to address areas of the City that did not achieve the minimum performance criteria.

The future maximum day (max day) demand scenario indicated that water pressure in the eastern region of the distribution system along $24^{\text {th }}$ Ave. SE was below the desired minimum pressure criteria. Additionally, the model predicted that a number of pipelines in the distribution system would experience elevated levels of headloss, requiring a capacity expansion. Finally, a number of fire hydrants were identified that did not meet the minimum desired available fire flow. A list of CIP projects was developed that, when implemented, will achieve the desired performance criteria under the future max day demand scenario. Key projects identified include a new elevated storage tank (EST) for the main distribution system pressure plane (MDS), additional pumping capacity in the MDS pump station, expansion of water mains along Robinson Street and $24^{\text {th }}$ Ave NE, and several pipeline renewal/maintenance projects targeting ductile iron lines that need replacing due to age or material incompatibility with soils.

The CIP project list was separated into six categories depending on the primary driver for the project, although there are multiple drivers for most projects that overlap between several categories. Prioritization of the projects was provided by NUA. CIP projects are displayed on Figure 5-1 (page 41), color coded by category. The CIP project categories included:

- Future Development. These projects are located in future development areas and would only be required when growth is experienced in these areas. Consequently, it is assumed that the developer will be responsible for the cost of these projects, not the City.
- Low Fire Flow. CIP projects in this category are required to increase available fire flows at hydrants throughout the distribution system.
- High Headloss. Pipelines experiencing a unit headloss approximately equal to or greater than $7 \mathrm{ft} / 1,000 \mathrm{ft}$ were identified in the model and a CIP project was created to increase the pipeline size to reduce headloss.
- Maintenance. CIP projects falling under the category of Maintenance have all been previously identified by NUA as pipelines that will require replacement in the near future due to pipe age or condition. In general, pipelines identified in the Maintenance category are sized appropriately for future flows and do not need to be replaced with larger lines, though there are some exceptions. These pipelines would be replaced to proactively prevent pipe failures in the future.
- Low Pressure. NUA desires to deliver a minimum pressure of 40 psi throughout the distribution system. Locations with a minimum pressure of 35 psi or less were addressed by recommending CIP projects to increase the pressure above 40 psi .
- High Water Age. This CIP project category includes projects that eliminate dead end water lines or create water loops to improve delivery efficiency and reduce water age.

Opinions of probable construction cost were developed for the water CIP projects. The combined OPCC for all of the recommended projects is approximately $\$ 95.5$ million. However, it is anticipated that approximately $\$ 6.4$ million of this total will be funded by developers. Table E-1 summarizes the total OPCC for each category in the CIP list. The complete list of CIP projects is presented in Appendix H .

Table E-1: CIP Project OPCC by Category

| Category | OPCC (millions) ${ }^{\text {A }}$ |
| :--- | :---: |
| Future Development | $\$ 6.4$ |
| Low Fire Flow | $\$ 10.9$ |
| High Headloss | $\$ 2.8$ |
| Maintenance | $\$ 66.5$ |
| Low Pressure | $\$ 4.0$ |
| High Water Age | $\$ 4.8$ |
| Total |  |
| City Responsibility | $\$ 95.5$ |

A. Costs are presented in 2017 dollars.

### 1.1 PURPOSE AND BACKGROUND

Norman Utilities Authority (NUA) provides water and sewer service to the City of Norman (City), located in Cleveland County in central Oklahoma. NUA last updated its water distribution system model and master plan in 2003. NUA contracted with Alan Plummer Associates, Inc. (APAI) to provide a Water Distribution Model (WDM) Update.

A primary objective of the WDM Update is to develop a water distribution system model including all pipes in the system, based on the City's GIS database. The City of Norman has also undergone significant growth since 2003, including improvements and changes to the water distribution system and water supply facilities. The purpose of this evaluation is to provide NUA with an updated water system model and to develop recommendations for system improvements through 2025 based on growth projections from the 2025 Land Use Plan.

### 1.2 SCOPE OF WORK

The scope of work defines the following major activities for this project.
A. Data collection from NUA and other sources necessary for model building and calibration.
B. Prepare population projections based upon the 2025 Land Use Plan ${ }^{1}$ (as amended) and the 2060 Strategic Water Supply Plan ${ }^{2}$.
C. Build an "all pipes" water model based on the City's GIS.
D. Calibrate the model using peak flow data, collected by NUA using pressure recording devices as provided by APAI.
E. Conduct a system performance evaluation.
F. Identify system improvements required for the maximum day future model scenario.
G. Recommend and develop a CIP list for the identified improvements.
H. Provide training to NUA's personnel on the hydraulic model.
I. Preparation and presentation of Model Update Report documenting the work conducted and the recommendations made.

[^0]
## 2 Water System Overview

NUA currently provides water service to the majority of citizens within its incorporated city limits. In conjunction with the 2025 Land Use Plan, NUA has mapped various growth area boundaries to describe level of service and categorize development density in the City (Figure 2-1, page 3). The 2025 Land Use Plan, and this WDM Update, includes potable water service to future developments within the Current Urban ${ }^{3}$ and Future Urban ${ }^{4}$ service areas. Projected water demands (see Section 3.2) will not include water service to the Suburban Residential ${ }^{5}$ or Country Residential ${ }^{6}$ areas.

The remainder of this section discusses the existing facilities owned and operated by NUA within the Current Urban service area.

### 2.1 DISTRIBUTION FACILITIES

NUA's existing water distribution system is comprised of approximately 597 miles of water mains ranging in size from 1 -inch to 36 -inches. The system includes two pressure planes, the Main Distribution System (MDS), which serves a majority of the city and the smaller Upper Pressure Zone (PZ) which serves approximately 4.4 square miles of northeastern Norman (See Figure 2-2, page 4). The MDS operates within a normal pressure range of 37 to 113 pounds per square inch (psi). The PZ operates within a normal pressure range of 49 to 108 psi . There are 17 existing isolation valves along the boundary between the two pressure planes. Also shown in Figure 2-2 (page 4) are the water treatment plant, which provides the main source of treated water and pumping capacity for the system; the active water towers which provide storage and pressure in the system; active groundwater wells; and the existing connection to Oklahoma City Water Utility.

[^1]


### 2.1.1 Pumping Capacity

There are two pump stations both located on the water treatment plant (WTP) property that distribute treated surface water to the distribution system. The larger MDS Pump Station (MDS PS) serves the Main Distribution System pressure plane and the PZ Pump Station (PZ PS) serves the Upper Pressure Zone. APAI provided pump testing services at the end of July and early August 2016 to define the current in situ pump curves to reflect accurate pumping conditions in the model. The pump testing protocol is provided in Appendix A and results of the testing are presented in Appendix B. The design flow and head of each pump are summarized in Table 2-1.

The MDS pump station includes four 250 horse power (hp) vertical turbine pumps, installed in 1982. Pumps 1 and 3 have variable frequency drives (VFDs). The PZ PS includes two 200 hp and two 125 hp vertical turbine pumps, installed in 1963 and 1993, respectively. All four PZ pumps are slated to be replaced within the next few years. The specifications of the selected future PZ PS pumps are described in Section 5.2.1.

Table 2-1: Pumping Facility Summary (Existing)

| Pressure <br> Plane | Pump <br> Number | Design Flow <br> (gpm) | Design Head <br> (ft) | Horsepower <br> (hp) |
| :---: | :---: | :---: | :---: | :---: |
| MDS | 1 | 3,600 | 231 | 250 |
|  | 2 | 3,500 | 231 | 250 |
|  | 3 | 3,500 | 231 | 250 |
|  | 4 | 3,500 | 231 | 250 |
| PZ | 1 | 2,083 | 288 | 200 |
|  | 2 | 2,083 | 288 | 200 |
|  | 3 | 1,388 | 288 | 125 |
|  | 4 | 1,388 | 288 | 125 |

### 2.1.2 Elevated Storage

NUA's distribution system includes five elevated storage tanks in the MDS (Lindsey Tower has been decommissioned and is no longer operating) and one in the PZ. The locations of the active tanks are shown in Figure 2-2 (page 4) and a summary of storage facility characteristics is provided in Table 2-2 (page 6). Additional information related to the elevated storage tanks is presented in Appendix C.

Table 2-2: Storage Facility Summary

| $\begin{array}{c}\text { Pressure } \\ \text { Plane }\end{array}$ | $\begin{array}{c}\text { Tower } \\ \text { Name }\end{array}$ | $\begin{array}{c}\text { Year } \\ \text { Built }\end{array}$ | $\begin{array}{c}\text { Storage } \\ \text { Volume } \\ \text { (MG) }\end{array}$ | $\begin{array}{c}\text { Ground } \\ \text { Surface } \\ \text { Elevation } \\ \text { (ft) }\end{array}$ | $\begin{array}{c}\text { Bottom of } \\ \text { Bowl (ft) }\end{array}$ | $\begin{array}{c}\text { Overflow } \\ \text { Elevation } \\ \text { (ft) }\end{array}$ | Notes |
| :---: | :--- | :---: | :---: | :---: | :---: | :---: | :--- |
|  | Cascade | 1999 | 2.0 | $1,189.50$ | $1,265.00$ | $1,315.00$ | Altitude valve. |
|  | Brookhaven | 1975 | 1.5 | $1,191.00$ | $1,272.60$ | $1,315.10$ | $\begin{array}{l}\text { MDS PS controls } \\ \text { off this tower. }\end{array}$ |
|  | Boyd | 1965 | 0.5 | $1,160.20$ | $1,280.00$ | $1,320.00$ | Altitude valve. |
|  | Robinson | 1954 | 0.5 | $1,190.20$ | $1,275.34$ | $1,315.00$ | $\begin{array}{l}\text { Has mixing } \\ \text { system installed } \\ \text { in tank and } \\ \text { altitude valve. }\end{array}$ |
|  |  | Lindsey | 1950 's | 0.5 | $1,153.20$ | $1,263.81$ | $1,312.01$ | \(\left.\begin{array}{l}Currently <br>

decommissioned <br>
due to location <br>
and changes in <br>
distribution <br>
system <br>
operations.\end{array}\right]\)

### 2.2 WATER SUPPLIES

NUA's main water source is surface water from Lake Thunderbird, which is pumped to the WTP via an existing 8 mile raw water pipeline comprised of 33 -inch and 30 -inch diameter segments. In 2014 NUA constructed a 48 -inch pipeline parallel to the 30 -inch segment to increase raw water conveyance capacity. NUA also has 30 to 36 active groundwater wells, the majority of which are located in the northeastern side of the City. In recent past years, NUA has decommissioned several of the wells due to water quality concerns. Those wells were located mostly in the central city area. Newer wells are located to the northeast of the City, which will also likely be the location of the well field expansion currently being evaluated in a separate study. For the purpose of this project, NUA provided APAI with the intersection of Tecumseh Rd. and $36^{\text {th }}$ Ave. NE as a representative location for the future well field point of entry in the distribution system model. It was assumed that an annual average supply of 2 million gallons per day (MGD) and maximum daily supply of 3 MGD would be available from the future well field. Additionally, NUA has an emergency connection to the Oklahoma City (OKC) distribution system that became operational in September 2000. NUA can control the amount of water received from OKC and prefers to limit usage to an average daily use of 1 MGD or less. During the week of calibration in August 2016, instantaneous flow from OKC varied between 0.52 and 1.12 MGD.

## 3 Water Demands

Historical water usage trends were estimated from daily pumping data and monthly billing data provided by NUA. Daily pumping data provides a historical record of the volume of water obtained from surface water (Lake Thunderbird to the WTP), groundwater wells, and the connection with OKC. A comparison of billed and pumped volumes also provides an estimate of non-revenue water used within the City.

Future water demands were projected based on historical water usage and projected land use trends.

### 3.1 HISTORICAL WATER DEMANDS

A six year period of production data (January 2010 through September 2016) was reviewed and normalized based on available demographic data and is summarized in Table 3-1. The ratio of maximum day demands to average day demands (max day factor) has historically ranged from 1.53 to 1.88 . The 2060 Strategic Water Supply Plan (Strategic Plan) ${ }^{7}$ used a max day factor of 1.9 for planning purposes. Based on the historical data, this same max day factor of 1.9 was used in this study. The annualized average percent non-revenue water (which includes real ${ }^{8}$ and apparent ${ }^{9}$ losses) is approximately $12.5 \%$.

Table 3-1: Historical Production Data

| Year | Service <br> Population | Total Production (1,000 gal/day) |  | Max <br> Day | Max Day <br> Date | Average <br> gpcd |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  | 6,792 | 12,225 | 22,242 | 1.82 | $8 / 9 / 2010$ | 124.7 |
| 2011 | 99,429 | 7,355 | 13,514 | 23,935 | 1.77 | $8 / 5 / 2011$ | 135.9 |
| 2012 | 100,782 | 7,315 | 13,231 | 24,822 | 1.88 | $7 / 23 / 2012$ | 131.3 |
| 2013 | 102,136 | 6,954 | 11,195 | 20,605 | 1.84 | $7 / 11 / 2013$ | 109.6 |
| 2014 | 103,489 | 7,014 | 13,113 | 20,692 | 1.58 | $7 / 8 / 2014$ | 126.7 |
| 2015 | 104,843 | 7,385 | 12,378 | 19,873 | 1.61 | $9 / 7 / 2015$ | 118.1 |
| 2016 | 106,197 | 7,523 | 11,931 | 18,254 | 1.53 | $8 / 15 / 2016$ | 112.3 |

[^2]Table 3-2: Top 10 Maximum Historical Production Days

| Rank | Year | Service <br> Population | Total Production (1,000 gal/day) |  | Max <br> Day | Max Day <br> Date |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  | Wells | OKC |  |  |  |  |
| 1 | 2012 | 100,782 | 13,327 | 8,251 | 3,244 | 24,822 | 1.88 | $7 / 23 / 2012$ |
| 2 | 2012 | 100,782 | 13,809 | 6,583 | 4,281 | 24,673 | 1.86 | $8 / 4 / 2012$ |
| 3 | 2012 | 100,782 | 14,741 | 8,281 | 1,541 | 24,563 | 1.86 | $7 / 22 / 2012$ |
| 4 | 2012 | 100,782 | 12,457 | 7,573 | 4,499 | 24,529 | 1.85 | $7 / 30 / 2012$ |
| 5 | 2012 | 100,782 | 13,463 | 6,598 | 4,417 | 24,478 | 1.85 | $8 / 3 / 2012$ |
| 6 | 2013 | 102,136 | 12,964 | 7,641 | - | 20,605 | 1.84 | $7 / 11 / 2013$ |
| 7 | 2012 | 100,782 | 14,069 | 8,084 | 1,973 | 24,126 | 1.82 | $7 / 27 / 2012$ |
| 8 | 2012 | 100,782 | 13,593 | 7,037 | 3,465 | 24,095 | 1.82 | $8 / 1 / 2012$ |
| 9 | 2012 | 100,782 | 13,237 | 8,385 | 2,458 | 24,080 | 1.82 | $7 / 20 / 2012$ |
| 10 | 2010 | 98,075 | 12,435 | 6,991 | 2,816 | 22,242 | 1.82 | $8 / 9 / 2010$ |

### 3.2 PROJECTED WATER DEMANDS

The following sections describe projection of annual water system demands and allocation of these demands to locations in the water system.

### 3.2.1 Projection of Annual Water System Demands

Land use projections in the Norman 2025 Land Use and Transportation Plan (Land Use Plan) ${ }^{10}$ begin in 2004. Between 2004 and 2025, the Land Use Plan projected an additional 10,032 single-family housing units (SFUs) and 3,034 multi-family housing units (MFUs) (Table 3-3, page 8).

Table 3-3: Projected Single-Family and Multi-Family Units

| Year | Single-- <br> Family <br> Units | Multi- <br> Family <br> Units |
| :---: | :---: | :---: |
| 2004 | 29,241 | 15,283 |
| Projected Increase | 10,032 | 3,034 |
| 2025 | 39,273 | 18,317 |

From the 2025 Land Use and Transportation Plan

[^3]In addition, NUA provided APAI with actual residential development data through 2015. An analysis of these data showed that recent growth has been different than projected in the Land Use Plan (Figure 3-1, page 9). Single-family development has occurred slightly more slowly than projected, and multi-family development has occurred more quickly than projected. The rapid multi-family development rate in recent years has resulted in overall more total housing units than predicted by the Land Use Plan.

Figure 3-1: Actual and Projected New Housing Units


In consultation with City staff, the following decisions were reached with respect to the population and water demand projections:

- Projected new housing units should be revised to account for actual development from 2004 through 2015. Revised projections for new housing units were developed by adding the projected new housing units for 2016 through 2025 to the actual 2015 new housing unit totals (Figure 3-2).
- Population densities should be based on those used in a previous wastewater modeling project performed by another consultant. The population densities for use in this study are 2.55 people per SFU and 1.91 people per MFU.
- The "High Service Area" (or "High SA") scenario from the Strategic Plan should be used for water system modeling and development of the CIP. ${ }^{11}$
- Water demand projections should be based on 145 gallons per capita per day (gpcd). This projection omits the 15 gpcd reserve supply and the passive water conservation savings discussed in the Strategic Plan.

Figure 3-2: Revised Projections of New Housing Units


Based on the projected number of new housing units, population projections are presented in Table 3-4 (page 11) and Figure 3-3 (page 11) and associated water demand projections are presented in Figure 3-4 (page 12). The decrease in the projected 2020 water demand from the 2060 Strategic Plan is caused by including passive water conservation savings in the projections (Figure 3-4, page 12). As described above, passive water conservation savings are not considered in the water demand projections for this study.

The projected 2025 water demand is 20,111 acre-feet per year (ac-ft/yr), for an average day water demand of 17.95 MGD.

[^4]Table 3-4: Revised Projections of New Housing Units and Population

| Year | Single- <br> Family <br> Units | Multi- <br> Family <br> Units | City <br> Population | High <br> Service <br> Area <br> Percentage | High <br> Service <br> Area <br> Population |
| :---: | :---: | :---: | :---: | :---: | :---: |
| 2004 | 29,241 | 15,283 | 103,101 | $85 \%$ | 90,305 |
| Projected Increase | 9,609 | 5,465 | 34,938 | $4.7 \%$ | 33,516 |
| 2025 | 38,850 | 20,748 | 138,039 | $89.7 \%$ | 123,821 |

Figure 3-3: Comparison of Population Projections to 2060 Strategic Water Supply Plan


Figure 3-4: Comparison of Annual Water Service Area Demand Projections to 2060 Strategic Water Supply Plan


### 3.2.2 Allocation of Projected Demands to Land Use Categories

Projected 2025 annual water demand in the water service area can be divided between existing water use and future development water use (Table 3-5, page 13). To assist in the allocation, City staff provided existing water use data for October 2015 through September 2016, including customer meter data, estimates of other uses, and estimated water loss. "Other uses" may include water used for fire-fighting, street cleaning, water main and sewer flushing, fire flow tests, and other unmetered uses. From October 2015 through September 2016, the City estimated the volume of other uses and water loss to be about 2.0 percent and 12.5 percent of total water use, respectively.

Table 3-5: Allocation of Projected Water Demands

| $\begin{gathered} 2025 \\ \text { Water } \\ \text { Use } \\ (17.95) \end{gathered}$ | Existing System Use <br> (16.06) | Metered Customer Use (13.73) |  |
| :---: | :---: | :---: | :---: |
|  |  | Other Uses (0.32) |  |
|  |  | Water Loss (2.01) |  |
|  | Future Development Use (1.89) | Metered <br> Flow for New Accounts (1.62) | Single-Family (0.76) |
|  |  |  | Multi-Family (0.15) |
|  |  |  | Office/Retail (0.25) |
|  |  |  | Industrial/Warehouse (0.38) |
|  |  |  | Parks (0.01) |
|  |  |  | Schools (0.001) |
|  |  |  | Other (0.07) |
|  |  | Other Uses (0.04) |  |
|  |  | Water Loss (0.24) |  |

Numbers represent approximate average day water demand in MGD.

The allocation of projected water demand among the different existing and future uses is described in detail in Appendix D. The primary assumptions in the allocation process are:

- Projections based on numbers of connections:
- 17.25 multi-family units per multi-family water connection. This was estimated from the average day water use for a multi-family connection (1,859 gallons per day) and the average day water use for an independently metered apartment (102 gallons per day), with adjustments for differences in irrigation between these types of connections.
- For each residential category, the unit water use was estimated to be the average of the 2015-16 average day water uses for all existing meters.
- Based on these procedures and the estimated population densities, these assumptions result in projected single-family water use of 77 gpcd and projected multi-family water use of 56 gpcd . Based on literature values and experience with other utilities, these are reasonable estimates.
- Projections based on information provided by the City
- The City provided information on parks and schools that are expected to be developed before 2025. For each new park and school, the City also identified an existing park or school with expected similar water use. Metered data from these comparable properties were used to estimate future water use at the new parks and schools.
- A 20-acre future OU development with 1,200-bed student housing and an office building was also identified by the City. Unit water use of 56 gallons per bed per day was assumed for student housing (same value as multi-family per capita water use). Projected water use for the office building is described in the next bulleted items.
- Projections based on land use acreage:
- For each category (office/retail and industrial/warehouse), the number of connections per acre was projected by identifying the existing total acreage of this land use and existing total number of meters for developed parcels with similar land use.
- Water use in the office/retail and industrial/warehouse land use categories is highly variable, depending on the property, with the average water use skewed by a few large water users (For each category, the average of the average day water use for all meters is about the 83rd percentile value). In addition, there are only 19 existing connections in the industrial/warehouse category that had metered 2015-16 water use. For these reasons, smaller percentile values were used that would also make the total allocated metered water use equal the amount projected based on the 2060 Strategic Water Supply Plan (Tables 2 and 3):
- 69th percentile average day water use for existing meters for the projections without water conservation and
- 63rd percentile average day water use for existing meters for the projections with water conservation.


### 3.2.3 Allocation of Projected Demands to Locations in the Water Service Area

Projected 2025 water demands were allocated in the model using a GIS shapefile showing areas of future development including active platted and preliminary platted areas as of 2016. The total future demand due to new metered accounts (1.62 MGD) plus future leakage \& water loss (0.28 MGD) was spread equally over approximately 440 nodes in areas of development in the water model.

Figure 3-5 (page 15) displays the areas of future development in the City where future demand was included in the water model. These areas are projected to develop by 2025.


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## 4 Model Development and Evaluation

A new water distribution system model was produced for this project, using the most current data available from NUA. The model was created from a GIS database provided by NUA and included all pipes in the system. All water mains were imported from GIS into the model, and the model auto-generated both junction and end nodes based simply on pipe spatial connectivity. The network-building feature of the Infoworks WS software was a useful, efficient method to quickly generate a base model network from existing GIS data. Digital elevation surfaces were built in the ArcGIS terrain format from 1-foot contours (covers majority of MDS and PZ system) and 2-foot contours (covers majority of well-field area) received from NUA. These terrain surfaces were used to assign the initial node elevations in the model, where each node was assigned the best available terrain surface ground elevation, minus 3.5 feet, to represent the node elevation below ground. In addition, some global model adjustments were required to build the functional model network: 1) the majority of pipes with length $<1$ foot were lengthened to 1 foot, 2) the pipes with diameter > length were extended so that length = diameter, and 3) additional junction nodes were inserted where needed to establish full connectivity of the water main network.

Storage and pumping facility data were obtained from as-built plans, NUA's SCADA system and staff knowledge and incorporated into the existing system model. Additionally, operational controls for pumps, wells, and valves were input to the model based on spreadsheet data and discussions with NUA staff. Existing demands were calculated and allocated using historical monthly City billing account information tied directly to billing addresses. Leakage demands representing the 12.5 percent of overall system water loss were then distributed uniformly across the system nodes to increase the total existing demand to production quantity.

This chapter discusses the following topics: Model Calibration, Extended Period Simulations, Performance Criteria, Model Analysis, and Water Quality.

### 4.1 CALIBRATION

Static and extended period simulation calibration runs were conducted on the newly constructed model for a week and a half from August 23, 2016 through September 2, 2016. Pressure and flow data were collected at 17 hydrant sites throughout Norman during this time period. Four hydrants, two recording pressure and two recording flow, were utilized at each site. Additionally, NUA SCADA data including flow and pressure from the MDS PS and PZ PS and elevated storage tank levels were also collected during the same period.

Figure 4-1 (page 17) displays the locations of the hydrants for each testing site. Appendix E contains detailed instructions that NUA followed for completing the flow tests as well as key maps for each specific site. In cases where pressure dropped significantly and approached a minimum pressure of 20 psi, NUA Staff flowed only one hydrant instead of two hydrants. (It was recommended that the system not be stressed below 20 psi.)


The hydrant flow tests were used to perform a static model calibration at each hydrant test site prior to flow being drawn through either hydrant. The goal of this calibration run is to accurately represent field pressure data in the model under the same conditions seen in the field at the time of data collection. This first level of calibration is useful in validating node elevations, tower (pressure head) elevations, and operational boundary conditions in the model.

The second calibration step (residual calibration) performed at each site involved opening hydrants to draw larger localized flow at each location. Flows in excess of $3,000 \mathrm{gpm}$ were measured in many areas when both of the test hydrants were flowing. These flows are higher at these pressure testing locations than would be expected during any peak hour demands. The higher flows used in the residual calibration tests assist in evaluating model pipe connectivity, pump operation, pipe roughness factors and system response times.

The hydrant test locations calibrated very well under static conditions and modeled pressures were within five psi of measured pressures at all test locations. The residual calibration was more difficult to refine. APAI and NUA Staff spent considerable time checking pipe sizes and connectivity and through this process discovered several lines or connections that were different in the model (GIS) than field conditions. With the investigation and resolution of each of these hydrant test areas, the residual calibration modeled conditions were brought closer to field measurements. APAI also refined and adjusted pipe friction factors (Hazen Williams C-Factors) as a final calibration step (Appendix F). Table 4-1 (page 19) provides a summary of the final calibration results and model notes according to each hydrant test site. Although calibration at several of the sites did not achieve the target agreement between measured and modeled results (<5 psi difference), significant improvements to the overall calibration were achieved after the initial calibration process, primarily through identification of pipe connectivity errors in the GIS/model. The modeled pressure results presented in Table 4-1 (page 19) are final results, after improvements were made to the model infrastructure. It is recommended that future model updates continue this process of improving GIS/model connectivity accuracy. Graphs showing detailed model and field calibration locations and output from the static and residual calibration are displayed in Appendix G.

Table 4-1: Model Calibration Summary

| Test | Reviewed by NUA? | Flow (gpm) | Date | P1 Gage Pressure Difference (psi) ${ }^{1}$ |  |  | P2 Gage Pressure Difference (psi) ${ }^{1}$ |  |  | \# of Hydrants Flowing | Notes |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  |  | Static (Before) | During <br> Flow <br> Test | Static <br> (After) | Static (Before) | During <br> Flow Test | Static <br> (After) |  |  |
| 1 | - | 3,760 | 8/23/16 | 1.6 | -6.0 | 0.3 | 0.7 | -4.0 | 0.4 | 2 flows | Calibration complete, test matches well. |
| 2 | Yes | 3,215 | 8/24/16 | 2.3 | -12.5 | 2.6 | 0.7 | -12.1 | 0.9 |  | Calibration complete. NUA \& APAI both checked lines thoroughly in this area. |
| 3 | Yes | 2,310 | 8/24/16 | 1.5 | -12.7 | 1.5 | 1.2 | -13.3 | 1.3 |  | Calibration complete. |
| 4 | Yes | 2,660 | 8/25/16 | 2.3 | -18.1 | 2.3 | 2.8 | -12.7 | 2.3 |  | Calibration complete. |
| 5 | Yes | 2,980 | 8/24/16 | 2.0 | -7.8 | 2.2 | 1.5 | -5.2 | 2.0 |  | Calibration complete. Made +5 psi improvements w/ addition of 12 -in line down $\mathrm{IH}-35$ frontage. |
| 6 | Yes | 5,430 | 8/25/16 | 2.9 | -5.3 | 1.6 | 3.0 | 2.7 | 2.0 | 2 flows | Calibration complete. |
| 7 | Yes | 1,540 | 8/25/16 | 2.8 | -14.4 | 0.5 | 1.0 | -20.1 | 1.0 |  | Calibration complete. City found an error along Franklin Rd. that will amplify the pressure issues. There is a CIP proposed by City that will help address pressure in this area, so no further adjustments at this time. |
| 8 | Site eliminated |  |  |  |  |  |  |  |  |  |  |
| 9 | Yes | 4,300 | 8/25/16 | 2.2 | -1.4 | 0.5 | 1.9 | 3.3 | -0.2 | 2 flows | Calibration complete. Plus $\sim 20$ psi for both hydrants. Test matches well now. |
| 10 | Yes | 3,170 | 8/25/16 | 2.4 | -12.7 | 3.4 | 2.3 | -18.4 | 2.4 |  | Calibration complete. Plus $\sim 5$ psi for both hydrants with NUA updates to GIS. Test matches better. |
| 11 | Yes | 3,350 | 8/30/16 | 1.6 | -1.5 | -0.1 | 1.9 | -9.8 | 1.0 |  | Calibration complete. Plus 4 psi for both hydrants from GIS updates. |
| 12 | - | 4,895 | 9/1/16 | 2.4 | -1.5 | 2.5 | 4.0 | -2.1 | 3.7 | 2 flows | Calibration complete, test matches well. |
| 13 | Yes | 5,630 | 9/1/16 | 4.0 | -4.3 | 1.5 | -0.2 | -6.3 | 1.7 | 2 flows | Calibration complete, test matches well. GIS update made slight improvement (+2 psi for P1, -0.4 psi for P2). |
| 14 | Yes | 3,080 | 8/30/16 | 2.5 | -6.2 | 1.5 | 2.6 | -4.3 | 1.6 |  | Calibration complete, test matches well. GIS update made slight improvement ( $+<1$ psi for P1 \& P2). |
| 15 | Yes | 2,660 | 8/30/16 | 2.1 | -11.6 | 1.7 | 1.7 | -15.1 | 1.3 |  | Calibration complete. APAI did verify wells are acting as they should. |
| 16 | Yes | 3,215 | 8/30/16 | 2.4 | -10.9 | 6.0 | 1.3 | -7.9 | 3.2 |  | Calibration complete. NUA \& APAI both checked lines thoroughly in this area. |
| 17 | - | 4,380 | 8/31/16 | 1.0 | -2.2 | -1.1 | 1.9 | -4.8 | -0.3 | 2 flows | Calibration complete, test matches well. |
| 18 | - | 4,718 | 8/31/16 | 3.8 | -0.2 | 3.3 | 4.8 | -2.8 | 2.8 |  | Calibration complete, test matches well. |

[^5]
### 4.2 EXTENDED PERIOD SIMULATIONS

Following calibration, APAI created an extended period simulation (EPS) to model a three day period of time for both existing and future conditions. As opposed to a steady-state simulation where there is no time variable, an EPS introduces time as a variable in the model for a more realistic evaluation of the distribution system. For example, an EPS can evaluate cycling of elevated storage tanks and the resulting water quality (age) over time. An EPS is also useful for determining pump efficiency by observing the percentage of time throughout the day that the pumps must be online to meet diurnal demands.

Two scenarios were constructed in the model: an existing conditions scenario and a future conditions scenario. Historical water demand was evaluated to determine an average day demand of 16.06 MGD with a max day factor of 1.9 for the existing conditions scenario (see Section 3.1). The future average day demand in 2025 was projected to be 17.95 MGD with 1.62 MGD of this additional flow allocated to future development and the remaining 0.28 MGD allocated to water loss \& leakage. The max day factor in the future conditions scenario was assumed to be equal to the max day factor used in the existing conditions scenario. Table 4-2 summarizes the demands used in the EPS scenarios.

Table 4-2: Extended Period Simulation Scenario Demands

| Modeling Scenario | Existing <br> Conditions | Future <br> Conditions |
| :--- | :---: | :---: |
| Design Year | FY 2015-2016 | 2025 |
| Average Day Demand (MGD) | 16.06 | 17.95 |
| Max Day Factor | 1.9 | 1.9 |
| Max Day Demand (MGD) | 30.52 | 34.11 |

Historical peak hour data was not provided for the system. However, for the model calibration period of August 23, 2016 through September 2, 2016 SCADA data was provided in a fine resolution. Production data in one minute increments for the MDS pumps (1 through 4), the Upper PZ, OKC connection, and active wells during the calibration period were plotted and used to determine the diurnal pattern of demands over time. Since this time frame was during summer, the peak hour demand factors were greater than average. The greatest factor was about 1.6 as shown on Figure 4-2 (page 21), which displays this data over a seven day period. The three day max day simulation began on a "Tuesday" and concluded at the end of the day on "Thursday" to capture the maximum peak on Thursday.

Results from the max day future conditions scenario were used to identify model improvements and recommend CIP projects to meet performance criteria outlined in Section 4.3. Additionally, the average day future conditions scenario was used to evaluate water age in the system since water quality issues typically occur during periods of lower demand. Results from the max day future conditions model run are presented in Section 4.4, and the water quality results are presented in Section 4.5.

Figure 4-2: Diurnal Pattern Measured during Calibration and Used in EPS Simulation


### 4.3 PERFORMANCE CRITERIA

A number of performance criteria were used to interpret results from the max day future demand scenario related to water pressure, available fire flow, and modeled headloss. The Oklahoma Department of Environmental Quality (DEQ) requires that a municipality provide a minimum water pressure of 25 psi throughout the distribution system, including during fire flow events. ${ }^{12}$ However, NUA preferred to improve on this standard by recommending a minimum water pressure of 40 psi, if possible. In the CIP recommendations, projects to improve pressure for any node experiencing 35 psi or less (during the maximum day scenario) were included.

For the minimum required available fire flow, the DEQ defers to requirements presented in publications and standard manuals of practice. ${ }^{13}$ The American Water Works Association (AWWA) has published a manual of practice for fire flow requirements in a distribution system, stating that the minimum available fire flow should be 500 gpm at a residual pressure of 20 psi. ${ }^{14}$ As a company practice, APAI generally recommends a minimum flow of $1,000 \mathrm{gpm}$ in residential areas and $>1,000 \mathrm{gpm}$ in commercial areas. NUA desired to have a minimum available fire flow of $1,500 \mathrm{gpm}$ at 25 psi , if possible. In the CIP recommendations, projects for hydrants with an available fire flow of less than $1,250 \mathrm{gpm}$ were included. APAI lowered the threshold from NUA's initial recommendation of a minimum flow of $1,500 \mathrm{gpm}$ to $1,250 \mathrm{gpm}$ based upon the large number of fire hydrant nodes that had available fire flows less than 1,500 gpm. A large number of them were in residential or newly developed areas where it did not make sense to upsize lines based upon this parameter alone. Lowering the threshold to 1,250 gpm eliminated a significant number of hydrants to address.

Finally, the maximum unit headloss through each pipe segment in the distribution system during the max day future conditions scenario was evaluated. Any segments with a unit headloss greater than $7 \mathrm{ft} / 1,000 \mathrm{ft}$ were recommended to be upsized. Some transmission mains were recommended to be upsized when unit headloss was less than $7 \mathrm{ft} / 1,000 \mathrm{ft}$ due to the potential for accumulated headloss along long stretches of larger diameter lines.

### 4.4 MODEL ANALYSIS

The calibrated water model was used to analyze the existing water distribution system for potential deficiencies. A projected 2025 max day demand scenario was applied to the calibrated model. Model results were evaluated for minimum node pressures, maximum line head losses, and maximum day available fire flow (numerical criteria given in Section 4.3). Each is described in more detail in the following sections.

### 4.4.1 Pressure

The existing system was initially run with existing max day demands to simulate the minimum pressures currently experienced throughout the distribution system on a day of maximum demands (Figure 4-3, page 24 and Figure 4-4, page 25).

[^6]In order to successfully run the 2025 max day demand scenario, the fourth MDS pump had to be turned on to provide enough supply to meet the increased max day demands. This means that all four MDS pumps are running and that the MDS PS will need a fifth pump to maintain firm capacity. With this update, the existing distribution system functions well under projected 2025 max day demands, with the majority of minimum pressures above 40 psi (Figure 4-5, page 26 and Figure 4-6, page 27). The exception is the boundary between the MDS and southwest side of the PZ, generally bounded by Highway 9 to the south, $36^{\text {th }}$ Ave. SE to the east, $12^{\text {th }}$ Ave. SE to the west, and E . Robinson St. to the north which experiences minimum pressures below 35 psi (Figure 4-7, page 28). The highest pressures in the system tend to be near the extremes of the system, especially on the southwest edge of the City along the Canadian River, where the ground surface elevation is at a minimum. The model predicts that approximately 420 out of over 27,000 nodes in the model will have a minimum pressure less than 35 psi .

CIP projects improving pressure at these identified locations are presented in Chapter 5. After implementing the recommended CIP projects, all 27,000 nodes had a minimum pressure of 35 psi or greater in the future max day scenario (Figure 4-8, page 29 and Figure 4-9, page 30). A detailed map showing the boundary between the MDS and the southwest area of the PZ is shown in Figure 4-10 (page 31).


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### 4.4.2 Fire Protection

Available fire flow was modeled in the existing system and the results are shown in Figure 4-11 (page 33). The existing distribution system also functions well under projected 2025 max day demands to generally provide adequate fire flow to all areas of the City (Figure 4-12, page 34). Without CIP improvements, hydrants below the minimum fire flow requirement of 1,250 gpm are not concentrated in any specific area, though small groupings can be identified in the City (Figure 4-13, page 35). Instead, hydrants not meeting the minimum flow requirements are generally scattered throughout the City. General examples of low flow hydrants include some on small diameter lines; older lines less than 6-inches in diameter. Also, hydrants located at the end of a cul-de-sac or dead end line sometimes exhibited low flow. Of the approximately 5,800 hydrants in the City, 119 of them are not able to provide at least $1,250 \mathrm{gpm}$ at 25 psi at the projected max day 2025 demands. The model predicts that approximately $98 \%$ of the City's hydrants will meet the fire flow requirements under future demands. CIP projects improving available fire flow at the remaining locations are presented in Chapter 5. Modeled available fire flow with the recommended CIP projects is shown in Figure 4-14 (page 36).

### 4.5 WATER QUALITY

Although a detailed evaluation of water quality was beyond the scope of this project, the model was used to evaluate water age in the system. While water age is not a significant concern, in itself, high water age can be an indicator of potential water quality issues, such as nitrification. Nitrification is a biological process where naturally occurring bacteria convert ammonia into nitrate. This can be a problem in distribution systems where ammonia may be present in the water (especially systems that maintain a chloramine residual such as Norman.) As residual chloramines degrade over time, ammonia is released, providing food for nitrifying bacteria. Not only does nitrification increase the concentration of nitrate in a distribution system, it can also reduce alkalinity, pH , and dissolved oxygen. These changes in water chemistry could affect the distribution system infrastructure (especially systems with lead and copper pipes), if not addressed. Furthermore, chlorine residual decreases as a result of nitrification, which could lead to bacterial regrowth in the distribution system. NUA noted that nitrification was recently observed in the distribution system between August 2015 and October 2015.

For the water age evaluation, the existing distribution system was modeled using existing and future average day demands. Water age in the existing system is displayed in Figure 4-15 (page 37). The predicted water age under future average day conditions without CIP projects is shown in Figure $4-16$ (page 38). The predicted water age after implementing the CIP projects recommended in Section 5 is shown in Figure 4-17 (page 39).

Appendix H presents modeling results for the water age experienced at each of the ESTs without and with the recommended CIP projects. In general, the simulated water age in the ESTs is acceptable. If water quality issues are observed, water age could be improved by installing mixers in the ESTs where there currently are none (Boyd, Brookhaven, and Cascade).








## 5 Water System Capital Improvements Plan

A water system capital improvements plan (CIP) has been created to meet the demands of projected growth in the City through 2025 and to fix existing system deficiencies. A total of 87 projects were identified (Table 5-1, page 41 and Figure 5-1, page 41). The majority of these projects are pipeline infrastructure projects. All together, these projects include construction of approximately 49 miles of water lines in the distribution system with a total opinion of probable cost of approximately $\$ 94$ million. A large version of Figure 5-1 has been included at the back of this report in the printed copies, following the appendices.

The CIP projects were separated into six categories depending on the main driver for the project, though most projects have benefits in multiple categories. The categories are defined as follows:

- Future Development. These projects are located in future development areas and would only be required when growth is experienced in these areas. Consequently, it is assumed that the developer will be responsible for the cost of these projects, not the City. The seven projects in this category account for approximately 4.1 miles of water line installation and approximately $\$ 6.4$ million.
- Low Fire Flow. CIP projects in this category were recommended to increase available fire flows at hydrants throughout the distribution system. This category accounts for the greatest number of projects on the CIP list; however, it does not represent the largest quantity of pipeline installation. The 48 projects identified for low fire flow concerns account for approximately 8.4 miles of pipeline and $\$ 10.9$ million.
- High Headloss. Pipelines experiencing a unit headloss approximately equal to or greater than $7 \mathrm{ft} / 1,000 \mathrm{ft}$ were identified in the model and a CIP project was created to increase the pipeline size to reduce headloss. There are five CIP projects identified as primarily caused by high headloss that account for 1.6 miles of water lines and approximately $\$ 2.8$ million.
- Maintenance. CIP projects falling under the category of Maintenance have all been previously identified by NUA as pipelines that will need replacement soon due to pipe age or condition. In general, pipelines identified in the Maintenance category are sized appropriately for future flows and do not need to be replaced with larger lines, though there are some exceptions. These pipelines would be replaced to proactively prevent pipe failures in the future. The Maintenance category comprises the majority of CIP projects in total length and cost, accounting for approximately 33 miles of pipelines and $\$ 66$ million.
- Low Pressure. NUA desires to deliver a minimum pressure of 40 psi throughout the distribution system. In the model, nodes with a minimum pressure of 35 psi or less were addressed by recommending CIP projects. Most of these CIP projects are not pipeline infrastructure projects. Instead they include unique projects such as expanding the Upper Pressure Zone, installing a new elevated storage tank, and adding a $5^{\text {th }}$ pump to the MDS PS. The opinion of probable cost for projects in this category is approximately $\$ 4$ million.
- High Water Age. The final CIP project category includes projects that eliminate dead end water lines or create water loops to improve delivery efficiency and reduce water age. These projects account for approximately 2.3 miles of pipeline replacement and $\$ 4.8$ million.

Table 5-1: List of CIP Projects

| Project Code |  |  | Description | Linear Feet of Pipe |  |  |  |  |  |  |  |  | Cost | Driver | Project <br> Priority |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  | 6" | 8" | 12" | 16" | 24" | 30" | 36" | 42" | Total |  |  |  |
| W |  | 5 |  | Water Line Segment D (Phase 4) | 0 | 0 | 0 | 0 | 8,500 | 0 | 0 | 0 | 8,500 | \$3,874,000 | $\begin{gathered} \text { High } \\ \text { Water Age } \end{gathered}$ | Highest |
| F |  | 39 | Upsize 8" Line to 12" along Meadowood Blvd | 0 | 1,000 | 1,030 | 0 | 0 | 0 | 0 | 0 | 2,030 | \$526,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \end{gathered}$ | High |
| H |  | 1 | Complete 12" Line Along 36th Ave. NE | 0 | 0 | 4,080 | 0 | 0 | 0 | 0 | 0 | 4,080 | \$1,147,000 | High Headloss | High |
| H |  | 3 | Upsize 6" Line to 12" at Alameda St. and Vicksburg Ave. | 0 | 0 | 105 | 0 | 0 | 0 | 0 | 0 | 105 | \$51,000 | High Headloss | High |
| H |  | 4 | Upsize Lines to Boyd Tower | 0 | 0 | 300 | 800 | 0 | 0 | 0 | 0 | 1,100 | \$390,000 | High Headloss | High |
| M |  | 5 | WL Replacement: Flood: Rock Creek to Venture | 0 | 0 | 3,400 | 6,400 | 0 | 0 | 0 | 0 | 9,800 | \$3,355,000 | Maint. | High |
| M |  | 7 | Robinson Waterline: 24th Ave. NE to 24th Ave. NW | 0 | 0 | 0 | 0 | 0 | 21,850 | 0 | 0 | 21,850 | \$11,576,000 | Maint. | High |
| M |  | 8 | Waterline Replacement: Interstate Drive | 0 | 5,680 | 0 | 0 | 0 | 0 | 0 | 0 | 5,680 | \$1,140,000 | Maint. | High |
| M |  | 11 | Water Line Replacement: Gray St. \& Tonhawa St. | 430 | 4,000 | 1,800 | 0 | 0 | 0 | 0 | 0 | 6,230 | \$1,002,000 | Maint. | High |
| M |  | 12 | Water Line Replacement: West of Campus | 8,150 | 1,550 | 0 | 0 | 0 | 0 | 0 | 0 | 9,700 | \$1,658,000 | Maint. | High |
| M |  | 13 | Alameda Waterline Replacement: S. Poncha Ave. to 24th Ave. NE | 0 | 0 | 0 | 0 | 8,500 | 200 | 0 | 0 | 8,700 | \$3,741,000 | Maint. | High |
| M |  | 15 | Robinson Waterline Replacement: WTP to 24th Ave NE | 0 | 0 | 0 | 0 | 80 | 0 | 0 | 2,600 | 2,680 | \$3,338,000 | Maint. | High |
| M |  | 17 | Replace Upper Pressure Zone Pumps | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | - | Maint. | High |
| P |  | 1 | Extend Upper PZ to Hollister Trail and Palomino Way | 0 | 425 | 0 | 0 | 0 | 0 | 0 | 0 | 425 | \$142,000 | Low Pressure | High |
| P |  | 4 | Include Meadowood Blvd in HPP | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | \$0 | Low Pressure | High |
| P |  | 5 | Future Elevated Storage Tank in MDS | 0 | 0 | 0 | 0 | 800 | 0 | 0 | 0 | 800 | \$3,638,000 | Low Pressure | High |
| F |  | 4 | Upsize 6" Line to 8" along Harriett Road | 0 | 1,160 | 0 | 0 | 0 | 0 | 0 | 0 | 1,160 | \$276,000 | Low Fireflow | Medium |
| F |  | 6 | Complete 6" loop along Thedford Drive | 425 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 425 | \$125,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \\ \hline \end{gathered}$ | Medium |
| F |  | 8 | Upsize 6" Line to 8" along Willow Creek Drive | 0 | 705 | 0 | 0 | 0 | 0 | 0 | 0 | 705 | \$200,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 9 | Extend the HPP to Redwood Drive | 0 | 600 | 0 | 0 | 0 | 0 | 0 | 0 | 600 | \$162,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 16 | Upsize 6" Line to 8" Along Eisenhower Rd | 500 | 2,010 | 0 | 0 | 0 | 0 | 0 | 0 | 2,510 | \$557,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F |  | 17 | Connect 6" dead end to 12" across N. Porter Ave. | 85 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 85 | \$39,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 25 | Upsize 6" Line to 8" along Pinebrooke Court | 0 | 590 | 0 | 0 | 0 | 0 | 0 | 0 | 590 | \$151,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 26 | Connect 6" Lines at Westport Dr. and Fairway Dr. | 700 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 700 | \$147,000 | Low Fireflow | Medium |
| F | - | 27 | Upsize 4" Line to 6" along Foreman Avenue | 1,150 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1,150 | \$254,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 28 | 8" Line along E Main St. Near Beacon Ave. | 0 | 1,180 | 0 | 0 | 0 | 0 | 0 | 0 | 1,180 | \$288,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 30 | Upsize 6" Line to 8" along Jean Marie Dr. | 0 | 1,875 | 0 | 0 | 0 | 0 | 0 | 0 | 1,875 | \$437,000 | $\begin{aligned} & \text { Low } \\ & \text { Fireflow } \end{aligned}$ | Medium |
| F | - | 32 | Extend 6 " line along Elm Avenue to W. Symmes St. | 220 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 220 | \$70,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 34 | Connect Dead-End 6" Line in The Pines Apartments | 450 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 450 | \$110,000 | Low Fireflow | Medium |
| F | - | 35 | Upsize 4" Lines to 6" along Justin Dr., Bill Carrol Dr., and Cara Jo Dr. | 650 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 650 | \$157,000 | Low Fireflow | Medium |
| F | - | 41 | Connect 6" Dead-End Line to McGee Drive | 600 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 600 | \$137,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 42 | Complete 6" Loop along Brookside Drive | 200 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 200 | \$85,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 43 | Upsize 6" Line to 8" along Rolling Hills Street | 0 | 820 | 0 | 0 | 0 | 0 | 0 | 0 | 820 | \$221,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Medium |
| F | - | 44 | Upsize 6" Line to 8" along Whispering Pines Drive | 0 | 460 | 0 | 0 | 0 | 0 | 0 | 0 | 460 | \$126,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \\ \hline \end{gathered}$ | Medium |
| H | - | 5 | Upsize 6" Line to 8" along Chautauqua Ave. | 0 | 400 | 0 | 0 | 0 | 0 | 0 | 0 | 400 | \$131,000 | High Headloss | Medium |
| M | - | 1 | WL Replacement: Classen/Flood: Hwy 9 to Indian Hills | 0 | 0 | 12,000 | 24,100 | 0 | 0 | 0 | 0 | 36,100 | \$11,975,000 | Maint. | Medium |
| M | - | 2 | Water Dist. System Improvements - Segment G | 0 | 0 | 7,280 | 0 | 0 | 0 | 0 | 0 | 7,280 | \$1,682,000 | Maint. | Medium |
| M | - | 3 | WL Replacement: Franklin: RR to 12th NW | 0 | 0 | 2,170 | 0 | 0 | 0 | 0 | 0 | 2,170 | \$584,000 | Maint. | Medium |


| Project Code |  |  | Description | Linear Feet of Pipe |  |  |  |  |  |  |  |  | Cost | Driver | Project Priority |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  | 6" | 8" | 12" | 16" | 24" | 30" | 36" | 42" | Total |  |  |  |
| M | - | 6 |  | Water Line Replacement: Hall Park, Phase 2 | 4,600 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4,600 | \$742,000 | Maint. | Medium |
| M | - | 9 | WL Replacement: W. Main: Berry to Interstate Drive | 0 | 5,170 | 6,830 | 0 | 0 | 0 | 0 | 0 | 12,000 | \$3,025,000 | Maint. | Medium |
| M | - | 10 | Waterline Replacement: Flood Avenue | 0 | 6,130 | 0 | 0 | 0 | 0 | 0 | 0 | 6,130 | \$1,505,000 | Maint. | Medium |
| M | - | 14 | 24th Ave NE Waterline Replacement: Alameda St. to Robinson St. | 0 | 0 | 0 | 0 | 0 | 0 | 5,200 | 0 | 5,200 | \$3,920,000 | Maint. | Medium |
| M | - | 16 | Robinson PZ Waterline Replacement: WTP to 24th Ave NE | 0 | 0 | 0 | 0 | 2,590 | 0 | 0 | 0 | 2,590 | \$1,177,000 | Maint. | Medium |
| P | - | 3 | Expand Upper PZ to Include Crest Place | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | \$0 | Low Pressure | Medium |
| W | - | 2 | New 12" pipe on Nantucket Blvd | 0 | 0 | 240 | 0 | 0 | 0 | 0 | 0 | 240 | \$81,000 | High Water Age | Medium |
| F | - | 1 | Loop 6" Line on Della St NW and NW Sterling Ct | 2,495 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2,495 | \$547,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \\ \hline \end{gathered}$ | Low |
| F | - | 10 | Upsize 6" Line to 8" along Briarcliff Rd | 0 | 1,170 | 0 | 0 | 0 | 0 | 0 | 0 | 1,170 | \$53,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Low |
| F | - | 12 | Upsize 6" Line to 8" along Hillside Drive | 0 | 910 | 0 | 0 | 0 | 0 | 0 | 0 | 910 | \$240,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Low |
| F | - | 14 | Upsize 6" Line to 8" along Valley Ridge Road | 0 | 1,250 | 0 | 0 | 0 | 0 | 0 | 0 | 1,250 | \$301,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Low |
| F | - | 20 | Upsize 6" Line to 8" along Wheaton Dr | 0 | 300 | 0 | 0 | 0 | 0 | 0 | 0 | 300 | \$99,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Low |
| F | - | 22 | Upsize 6" Line to 8" along Hunter's Hill Road | 0 | 1,440 | 0 | 0 | 0 | 0 | 0 | 0 | 1,440 | \$357,000 | $\begin{aligned} & \text { Low } \\ & \text { Fireflow } \end{aligned}$ | Low |
| F | - | 24 | Upsize 6" Line to 8" along Cedar Ridge Drive | 0 | 470 | 0 | 0 | 0 | 0 | 0 | 0 | 470 | \$127,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Low |
| F | - | 31 | Upsize 6" Line to 8" along McFarland St. | 0 | 530 | 0 | 0 | 0 | 0 | 0 | 0 | 530 | \$139,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Low |
| F | - | 36 | Upsize 6" Lines to 8" along Brandon Cr., Sheffield Dr., Chamblee Dr., Surrey Dr., \& Village Dr. | 0 | 1,725 | 0 | 0 | 0 | 0 | 0 | 0 | 1,725 | \$416,000 | Low Fireflow | Low |
| F | - | 37 | Upsize 6" Line to 8" along Columbia Cr., Atlanta Cr., Montgomery Cr., Raleigh Cr., and Mobile Cr. | 0 | 1,705 | 0 | 0 | 0 | 0 | 0 | 0 | 1,705 | \$511,000 | Low Fireflow | Low |
| F | - | 38 | Upsize 6" Line to 8" along Peppertree Ct. | 0 | 680 | 0 | 0 | 0 | 0 | 0 | 0 | 680 | \$195,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Low |
| F | - | 40 | Upsize 6" Line to 8" South of Briggs St. | 0 | 410 | 0 | 0 | 0 | 0 | 0 | 0 | 410 | \$132,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \end{gathered}$ | Low |
| F | - | 45 | Upsize 6" Line to 8" along Holly Cir. | 0 | 50 | 0 | 0 | 0 | 0 | 0 | 0 | 50 | \$43,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Low |
| F | - | 46 | Extend 6" Line Along Twin Creek Village Apartments | 360 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 360 | \$95,000 | Low Fireflow | Low |
| H | - | 2 | Upsize 12" Line to 16 " along Robinson from WTP to 36th Ave. NE | 0 | 0 | 0 | 2,730 | 0 | 0 | 0 | 0 | 2,730 | \$1,073,000 | High <br> Headloss | Low |
| M | - | 4 | Waterline Improvement: OKC Second Feed | 0 | 0 | 0 | 0 | 31,680 | 0 | 0 | 0 | 31,680 | \$16,077,000 | Maint. | Low |
| P | - | 2 | Add 5th 250 HP Pump to MDS PS | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | \$260,000 | Low Pressure | Low |
| W | - | 1 | Complete 6" loop along Teton Oval culdesac | 120 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 120 | \$53,000 | High Water Age | Low |
| W | - | 3 | Upsize 6" Line to 8" along Shrill St. | 0 | 2,890 | 25 | 0 | 0 | 0 | 0 | 0 | 2,915 | \$683,000 | High Water Age | Low |
| W | - | 4 | Connect 6" Lines at NW corner of 24th Avenue NW and W. Main Street | 540 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 540 | \$144,000 | High Water Age | Low |
| F | - | 2 | Upsize 6" Line to 8" along Moor Drive and Nicole Place | 0 | 790 | 0 | 0 | 0 | 0 | 0 | 0 | 790 | \$215,000 | Low Fireflow | Very <br> Low |
| F | - | 3 | Upsize 6" Line to 8" along Nicole Circle | 0 | 675 | 0 | 0 | 0 | 0 | 0 | 0 | 675 | \$184,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Very <br> Low |
| F | - | 5 | Upsize 6" Line to 8" along Bright St., Glisten Ct., Ripple Ave., \& Glisten St. | 0 | 1,615 | 0 | 0 | 0 | 0 | 0 | 0 | 1,615 | \$395,000 | $\begin{aligned} & \text { Low } \\ & \text { Fireflow } \end{aligned}$ | Very <br> Low |
| F | - | 7 | Upsize 6" Line to 8" along Sloane St., Shipley Dr., Bishop's Ct., \& Victoria Dr. | 0 | 1,600 | 0 | 0 | 0 | 0 | 0 | 0 | 1,600 | \$392,000 | $\begin{aligned} & \text { Low } \\ & \text { Fireflow } \end{aligned}$ | Very <br> Low |
| F | - | 11 | Upsize 6" Line to 8" off of Brookhaven Blvd | 0 | 345 | 0 | 0 | 0 | 0 | 0 | 0 | 345 | \$101,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \end{gathered}$ | Very Low |
| F | - | 13 | Upsize 6" Line to 8" on Northhampton Court | 334 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 334 | \$108,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \end{gathered}$ | Very <br> Low |
| F | - | 15 | Upsize 6" Line to 8" along Warwick Dr. and Waverly Dr. | 0 | 1,970 | 0 | 0 | 0 | 0 | 0 | 0 | 1,970 | \$473,000 | Low Fireflow | Very Low |
| F | - | 18 | Upsize 6" Line to 8" along Wind Hill Rd | 0 | 400 | 0 | 0 | 0 | 0 | 0 | 0 | 400 | \$119,000 | Low Fireflow | Very Low |
| F | - | 19 | Upsize 6" Line to 8" along Ridgemont Circle | 0 | 460 | 0 | 0 | 0 | 0 | 0 | 0 | 460 | \$131,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Very Low |


| Project Code |  |  | Description | Linear Feet of Pipe |  |  |  |  |  |  |  |  | Cost | Driver | Project Priority |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  | 6" | 8" | 12" | 16" | 24" | 30" | 36" | 42" | Total |  |  |  |
| F | - | 21 |  | Upsize 6" Line to 8" along Sundance Ct. | 0 | 360 | 0 | 0 | 0 | 0 | 0 | 0 | 360 | \$105,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \end{gathered}$ | Very Low |
| F | - | 23 | Upsize 6" Line to 8" along Innsbrook Court | 0 | 350 | 0 | 0 | 0 | 0 | 0 | 0 | 350 | \$102,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \\ \hline \end{gathered}$ | Very Low |
| F | - | 29 | Upsize 6" Line to 8" along Riverwalk Ct. | 0 | 825 | 0 | 0 | 0 | 0 | 0 | 0 | 825 | \$206,000 | $\begin{gathered} \hline \text { Low } \\ \text { Fireflow } \\ \hline \end{gathered}$ | Very Low |
| F | - | 33 | Upsize 6" Line to 8" along Schulze Dr. and Creston Way | 0 | 1,425 | 0 | 0 | 0 | 0 | 0 | 0 | 1,425 | \$337,000 | $\begin{aligned} & \text { Low } \\ & \text { Fireflow } \end{aligned}$ | Very Low |
| F | - | 47 | Upsize 6" Lines to 8" along White Oak Cir., Oak Vista Cir., \& Bois-de-arc Cir. | 0 | 1,170 | 0 | 0 | 0 | 0 | 0 | 0 | 1,170 | \$286,000 | Low Fireflow | Very Low |
| F | - | 48 | Loop 6" Line along Black Locust Ct \& Black Locust Place | 985 | 1,055 | 0 | 0 | 0 | 0 | 0 | 0 | 2,040 | \$459,000 | $\begin{gathered} \text { Low } \\ \text { Fireflow } \end{gathered}$ | Very <br> Low |
| D | - | 1 | 12" Loop along 48th Avenue NW | 0 | 1,175 | 6,240 | 0 | 0 | 0 | 0 | 0 | 7,415 | \$1,877,000 | Fut. Dev. | - |
| D | - | 2 | Install 12" line along 48th Ave NW between W Rock Creek Rd and Las Colinas Ln | 0 | 0 | 2,475 | 0 | 0 | 0 | 0 | 0 | 2,475 | \$663,000 | Fut. Dev. | - |
| D | - | 3 | Waterline Segment H | 0 | 0 | 1,500 | 0 | 0 | 0 | 0 | 0 | 1,500 | \$368,000 | Fut. Dev. | - |
| D | - | 4 | Add 6" line near Wyckham PI. | 675 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 675 | \$169,000 | Fut. Dev. | - |
| D | - | 5 | Add 6" Line Along Kingswood Dr | 340 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 340 | \$89,000 | Fut. Dev. | - |
| D | - | 6 | Extend 8" Lines to Harbor Dr. and Lyric St. | 0 | 1,335 | 0 | 0 | 0 | 0 | 0 | 0 | 1,335 | \$335,000 | Fut. Dev. | - |
| D | - | 7 | 16" Destin Landing Development | 0 | 0 | 0 | 8,000 | 0 | 0 | 0 | 0 | 8,000 | \$2,853,000 | Fut. Dev. | - |



### 5.1 CIP LIST AND OPINION OF PROBABLE CONSTRUCTION COST

This section summarizes the list of CIP projects identified during the modeling process along with assumptions and methodology used to develop an opinion of probable construction cost (OPCC) for each project. Table 5-2 summarizes the total number of recommended CIP projects and combined OPCC for each category. All costs are presented in 2017 dollars.

Table 5-2: Summary List of 2025 CIP Projects

| Category | Number of Projects | OPCC (million) ${ }^{\boldsymbol{A}}$ |
| :--- | :---: | :---: |
| Future Development | 7 | $\$ 6.4$ |
| Low Fire Flow | 48 | $\$ 11$ |
| High Headloss | 5 | $\$ 2.8$ |
| Maintenance | 17 | $\$ 66$ |
| Low Pressure | 5 | $\$ 4.0$ |
| High Water Age | 5 | $\$ 4.8$ |
| $r$ Total | $\mathbf{8 7}$ | $\$ 95.5$ |
| City Responsibility | $\mathbf{8 0}$ | $\$ 89.1$ |

A. Costs are presented in 2017 dollars.

A planning level OPCC was prepared for each CIP project using actual pipeline costs from previous NUA projects along Robinson St., Lindsey St., and Berry Road. Additionally, NUA had prepared planning level OPCCs for a number of projects previously identified (projects in the Maintenance category). APAI maintained several assumptions that NUA used to prepare these previous OPCCs, updating them when necessary. Table 5-3 presents the pipeline unit costs that were used to prepare OPCCs for each CIP project. Additionally, ancillary pipeline costs were included in each OPCC (Table 5-4, page 46). The complete list of recommended CIP projects along with a detailed OPCC for each project is provided in Appendix I. The projects are sorted according to NUA's prioritization.

Table 5-3: Planning Level Pipeline Costs

| Diameter <br> (in) | Trenched <br> Unit Cost <br> $(\$ / L F)$ | Boring and <br> Casing Unit <br> Cost (\$/LF) |
| :---: | :---: | :---: |
| 6 | $\$ 53$ | $\$ 246$ |
| 8 | $\$ 68$ | $\$ 296$ |
| 12 | $\$ 84$ | $\$ 371$ |
| 16 | $\$ 138$ | $\$ 468$ |
| 24 | $\$ 166$ | $\$ 628$ |
| 30 | $\$ 230$ | $\$ 1,194$ |
| 36 | $\$ 300$ | $\$ 1,719$ |
| 42 | $\$ 350$ | $\$ 2,340$ |

Table 5-4: Additional Cost Assumptions

| Item | Value |
| :--- | :---: |
| ROW Width | 15 ft |
| ROW Cost | $\$ 3 / \mathrm{ft}^{2}$ |
| Mobilization and Insurance | $5 \%$ of Subtotal |
| Contingency | $30 \%$ of (Subtotal + Mobilization and Insurance) |
| OPCC | Subtotal + Mobilization and Insurance + Contingency |
| Design and Inspection <br> During Construction | $15 \%$ of OPCC |

### 5.2 MAJOR PROJECTS DESCRIPTION

There are a number of major CIP projects that warrant additional discussion. These projects were selected for discussion because of their unique project components or large OPCC. The following sections elaborate on these projects.

### 5.2.1 WTP Pump Station Projects

NUA is planning to replace the existing four Upper Pressure Zone (PZ) pumps within the next few years (CIP project $\mathrm{M}-17$ ) with three new pumps. These pumps have already been selected by NUA and the future pump design specifications were used in the model for future scenarios (Table 5-5). The cost for these pumps was not included in the CIP because the pumps have already been selected.

Table 5-5: Future PZ Pump Specifications

| Speed Control | Fixed |
| :--- | :--- |
| Pump Speed, rpm | 1,770 |
| Design Flow, gpm | 1,725 |
| Design Head, ft | 253 |
| Minimum Efficiency at Design Point | $84 \%$ |

Additionally, a CIP project was created to add a fifth pump to the MDS PS (CIP project P-2) with identical design parameters to the existing four (Table 2-1, page 5). The model showed that minimum pressures cannot be met in the future max day scenario without 4 MDS pumps operating concurrently. The fifth pump would be required for firm capacity.

### 5.2.2 Upper Pressure Zone Boundary Changes

It is recommended that NUA extend the PZ boundary in four locations to include areas that are currently served by the MDS due to low water pressure or available fire flow in these areas (Figure 5-2, page 47). Two of these areas can be incorporated into the PZ simply by the addition of new isolation valves or manipulation of existing isolation valves. Two areas (CIP projects P-1 and F-9) would also require short pipelines. Detailed maps for each of the five recommended boundary changes are provided in Appendix J.


### 5.2.3 Water Supply Expansion

There are two future water supplies that were considered in this WDM update: a second connection to the OKC distribution system and an expansion of NUA's groundwater well network. The groundwater expansion project is currently being planned and designed, but the second connection to OKC is a water supply project that NUA will consider in the future.

The future supply from the groundwater project was simulated in the future conditions water model and is expected to provide 2 MGD of annual average supply and 3 MGD of supply for max day. This future supply was added into the model at the intersection of E. Tecumseh Rd. and $36^{\text {th }}$ Ave. NE, at the direction of NUA.

The second feed from OKC is identified as CIP project M-4, which would allow NUA to purchase up to 6 MGD of additional treated water from OKC through a new six mile pipeline. This project was identified in NUA's 2060 Strategic Water Supply Plan as a viable water supply to meet future demands. This project is not necessary to meet the projected 2025 demands, but is included as a potential future option for NUA.

### 5.2.4 Major Maintenance Projects

There are a number of significant CIP projects in the Maintenance category that should be highlighted. The two overall highest cost CIP projects (not including the project to add a second feed from OKC) are CIP projects M-7 and M-1. Additionally, there are a collection of four related projects that would replace major water lines near the WTP (M-13, M-14, M-15, and M-16). These six projects were all previously identified by NUA due to maintenance reasons, but some of these projects also have a capacity benefit as they will replace lines with a larger size.

Project M-7 would install approximately 4 miles of 30" PVC pipeline along Robinson Street from $24^{\text {th }}$ Ave. NE to $24^{\text {th }}$ Ave. NW, replacing an existing $16^{\prime \prime}$ ductile iron pipe (DIP) for the majority of this route. However, this project would not replace a short segment of 16 " pipe that exists underneath the railroad tracks near N. Flood Ave. since that project was completed in recent years. This project increases water pressure on the western side of the City and improves water age near the Westwood Park Golf Course by providing a more direct route to the western part of the City from the WTP. This project also provides a third main path for water to be fed by the WTP to the rest of the City. This helps reduce headloss along the transmission mains and allow the MDS pumps to remain working within their most efficient points.

Project M-1 would replace approximately 7 miles of 12 " pipeline along Classen Blvd., James Garner Ave., and N Flood Ave. generally from Highway 9 to Well \#20 just north of Indian Hills Road. A subset of this route has been assigned as a separate CIP project (M-5) that would replace approximately 2 miles of existing 16 " and 12 " lines along N Flood Ave. from Rock Creek Road to Venture Drive. These projects are required primarily for maintenance reasons. The DIP material used along this route is not compatible with the corrosive clay soils in the area and has ruptured in some areas causing extensive damage to driveways, streets, and yards. These lines would be replaced with PVC pipe, which is not as susceptible to corrosion.

Projects $\mathrm{M}-13, \mathrm{M}-14, \mathrm{M}-15$, and $\mathrm{M}-16$ would replace major waterlines near the City's WTP along Robinson St., $24^{\text {th }}$ Ave. NE, and Alameda St for maintenance and capacity reasons. Combined,
these four projects would replace approximately 3.6 miles of pipelines and cost approximately $\$ 12$ million. Project M-13 would replace 1.5 miles of 24 " pipeline in the MDS along Alameda St. from $24^{\text {th }}$ Ave NE to S. Ponca Ave. with 24 " PVC for maintenance reasons. Project M-14 would replace 1 mile of 30 " pipeline in the MDS along $24^{\text {th }}$ Ave NE from Alameda St. to Robinson St. with 36 " PVC line. Project M-15 would replace the existing 30 " MDS pipeline along Robinson St. between the WTP and $24^{\text {th }}$ Ave NE with a $42^{\prime \prime}$ line. The water model predicts that this existing 30 " MDS line along Robinson St. will experience slightly elevated velocities ( $\sim 6.5 \mathrm{ft} / \mathrm{s}$ ) under the future 2025 max day demand scenario. This line will be upsized to a 42" pipe to reduce headloss through this section of line and to provide additional capacity for future growth beyond 2025. Additionally, project $\mathrm{M}-15$ would replace a short section of the 16 " PZ pipeline crossing under Robinson St. at the intersection of Robinson St. and $24^{\text {th }}$ Ave NE with a $24^{\prime \prime}$ line. The City plans to widen this intersection and the existing line will need to be relocated farther east. Finally, project $\mathrm{M}-16$ would replace an existing 24 " PZ line between the WTP and $24^{\text {th }}$ Ave NE south of Robinson St. with a new 24 " PVC line for maintenance reasons. After replacement this line would be repurposed for use in the MDS instead of the PZ, as a parallel redundant line to the line replaced in project $\mathrm{M}-15$. Valves will be required to isolate this line from the PZ system once repurposed. The CIP projects at the intersection of Robinson St. and $24^{\text {th }}$ Ave NE are displayed in Figure 5-3.

Figure 5-3: CIP Projects at Intersection of Robinson St. and $\mathbf{2 4}^{\text {th }}$ Ave NE


### 5.2.5 Future MDS Elevated Storage Tank (EST)

To achieve the minimum water pressure criterion of 35 psi along 24 Ave SE in the future max day modeling scenario, it is necessary to add another EST in the MDS (project P-5). Due to the condition of the existing Lindsey EST and site constraints, a new tank would likely need to be located elsewhere in the MDS, instead of rehabilitating the existing tank. At the request of NUA, APAI evaluated a number of alternative locations for the new EST (Figure 5-4, page 51). These possible locations are preliminary and a detailed siting study would be conducted prior to the final selection of a future EST location. It is recommended that the water model be used during the siting study to better inform the selection process. A comparison of the initially identified possible future EST locations based on a number of factors is provided in Table 5-6.

Table 5-6: Comparison of Possible Future EST Locations

| Location | 1 | 2 | 3 |
| :--- | :---: | :---: | :---: |
| Description | Eastridge <br> Park | Highway 9 and <br> $24^{\text {th }}$ Ave SE | Saxon Park |
| Ground Elevation (ft) | 1,191 | 1,183 | 1,160 |
| Overflow Elevation (ft) | 1,320 | 1,320 | 1,320 |
| Height (ft) | 129 | 137 | 160 |
| Volume (MG) | Fully Mixed | Fully Mixed | Fully Mixed |
| Mixing Status | 800 | 150 | 150 |
| Length of Pipeline <br> Required (ft) | 24 | 24 | 24 |
| Diameter of Connecting <br> Pipeline (in) | 33 | 36 | 70 |
| Average Water Age in <br> EST (hrs) | Yes | Yes | Yes |
| Minimum Pressures are <br> Met in System? | Yes | Yes | Yes |
| Minimum Fire Flows are <br> Met in System? |  |  |  |



In summary, all three of the locations will allow the distribution system to meet the performance criteria for minimum water pressure and available fire flow. However, there are slight differences between the locations that should be noted:

1. The ground surface elevation at Location 3 is approximately 30 ft lower than the elevation at Locations 1 and 2, meaning that the EST at Location 3 will need to be physically taller to achieve the same overflow elevation. It should be noted that there are other locations in Saxon Park (Location 3) with a higher ground surface elevation where the EST could be located, instead, but the length of the connecting pipeline would increase as a result.
2. In general, Locations 1,2 , and 3 are all located close to a large water line ( $\geq 12^{\prime \prime}$ ). Location 1 would connect into a 30 " line, Location 2 would connect into a $12^{\prime \prime}$ line, and Location 3 would connect into a future 16 " line along the Destin Landing development. Location 2 could connect to a 30 " water line running east/west along Highway 9, but this connection would require crossing under Highway 9. The 12" line that Location 2 connects into experiences a slightly greater unit headloss (up to $4.7 \mathrm{ft} / 1,000 \mathrm{ft}$ ), which would add flow to an already hydraulically stressed pipe. Connecting Location 2 to the 30 " line along Highway 9 would eliminate the elevated headloss, but would be more expensive to bore under Highway 9. All three of these locations would require a connecting pipeline length of less than 200 ft . However, Location 1 is set back from a major water line and would require approximately 800 ft of pipeline to connect to the 30 " line running north/south along $24^{\text {th }}$ Ave SE.
3. The average age of water in the EST at Location 1 and 2 is in the range of 30 to 40 hours. However, the average water age in the EST at Location 3 is approximately 70 hours. This is likely due to the fact that Location 3 is positioned on the fringe of the distribution system.
4. Finally, Location 3 is positioned in an area of the City anticipated to be developed in the future. It is possible that a future developer would be willing to contribute towards the cost of a new EST.

It is recommended that a detailed siting study be conducted prior to selecting a final EST location. This preliminary evaluation does not consider critical site selection components such as land acquisition cost, public opposition to location, or site conditions affecting constructability. The future max day modeling scenario assumes that the new EST is positioned at Location 1, as directed by NUA.

## Appendix A - Pump Testing Protocol

DRAFT
TECHNICAL MEMORANDUM

## City of Norman <br> Water Distribution System Hydraulic Modeling Pump Testing Protocol

## Project No.: 1115-004-01

## Date: <br> July 8, 2016

Prepared For: Mark Daniels, P.E., Utilities Engineer, Department of Utilities - City of Norman
Prepared By: Brian K. McDonald, P.E., APAI
Sarah Seamands, P.E., APAI

Alan Plummer Associates, Inc. (APAI) is building a new hydraulic model of the City of Norman's water distribution system as part of the updated Water Master Plan. Part of this project includes developing revised pump curves that are representative of current operating conditions for all high service and booster pumps greater than five years old that serve the main distribution system (MDS) or the upper pressure zone (PZ) (Table 1).

Table 1: City of Norman High Service Pumps

| Pressure <br> Plane | Pump No. | Nominal <br> Flowrate <br> (gpm) | Horsepower | Installation <br> Date | Status |
| :---: | :---: | :---: | :---: | :---: | :---: |
| MDS | 1 | 3,600 | 250 | 1982 | To be evaluated |
|  | 2 | 3,500 | 250 | $1982^{*}$ | To be evaluated |
|  | 3 | 3,500 | 250 | 1982 | To be evaluated |
|  | 4 | 3,500 | 250 | 1982 | To be evaluated |
|  | PZ | Total Capacity is 20.3 mgd at 100 psi. Firm Capacity is 15.1 mgd at 100 psi. |  |  |  |
|  | 5 | 2,083 | 200 | 1963 | To be replaced |
|  | 6 | 2,083 | 200 | 1963 | To be replaced |
|  | 7 | 1,388 | 125 | 1993 |  |
|  | 8 | 1,388 | 125 | 1993 |  |
|  | Total Capacity is 10.0 mgd at 125 psi. Firm Capacity is 7.0 mgd at 125 psi. |  |  |  |  |

* Rehabilitated in 2003.

The two largest pumps serving the PZ are scheduled to be replaced as part of an upcoming water treatment plant (WTP) improvements project ${ }^{1}$, but per Chris Mattingly with Norman Utilities Authority (NUA), they will likely not be replaced for another two years. APAI recommends that these two pumps are tested this summer along with the others so that pump curves can be developed and used in the model calibration. If NUA knows what type of pump will be specified or if the manufacturer pump curves for the new pumps is available, this information should be provided to APAI for inclusion in the future model runs.

The pumps that serve the MDS are scheduled to be evaluated for repair or replacement as part of the WTP improvements project, but it is anticipated this project will occur well after the hydraulic water model is developed and calibrated. At the present time, it is anticipated that pumps 1-4 for the MDS plane will each be individually tested. If NUA knows what type of pumps will be specified or if the manufacturer pump curves for new pumps is available, this information should be provided to APAI for inclusion in the future model runs.

Section 1 of this memorandum discusses the field testing protocol for individual pumps. Section 2 describes how all pumps will be included in testing to develop the system curves for each pump station.

## SECTION 1. FIELD TESTING PROTOCOL FOR INDIVIDUAL PUMPS

Prior to pump testing, the City should measure discharge shut-off head and lowest discharge head for each pump. Use the following steps to test each pump individually for the development of new pump performance curves:

1. Record the following general items:
a. Pump station floor elevation (from record drawings),
b. Centerline elevation of the discharge,
c. Distance from centerline of the discharge to centerline of the pressure gauge,
d. Diameter of discharge piping at the pressure gauge, and
e. Pump nameplate information.
2. Perform testing while the City's elevated storage tanks are at their lowest level (when the static head on the pumps is lowest and will allow the pumps to be tested closer to the run-out condition). Lowest levels in the elevated storage tanks typically occur after the morning peak demand.

[^7]3. Synchronize the clocks for the various recording equipment (or at a minimum, document the differences in their recorded time). This includes the water treatment plant supervisory control and data acquisition (SCADA) system, the Shermco electrical data recorder, and the manual pressure and wetwell level recordings. This is critical so that data taken for each test can be grouped together correctly.
4. Turn off all pumps in the pump station. For the pump to be tested, install the pressure gauge on the discharge and connect the electrical data recording equipment at the motor control center (MCC). Attach a piece of reflective tape on a visible portion of the pump shaft so that the laser tachometer can read the pump RPM.
5. Verify that personnel and equipment are ready to record the data for Step 8.

Data will be collected at up to five different flowrates. The flowrates will be varied by opening/closing the pump discharge manual butterfly valve. Five data points will be taken, evenly spaced between the lowest discharge head (run-out) and the shut-off head. The measurement data points are at: (1) lowest discharge head, (2) 25 percent of the head range, (3) 50 percent of the head range, (4) 75 percent of the head range, (5) shut-off head.
6. Fully open the pump discharge manual butterfly valve.
7. Turn on the test pump.

When the pump receives the signal to turn on, the SCADA system will start the pump. Once the pump reaches its operating speed and the discharge pressure reaches a set value, the SCADA system will open the automatic valve. With the manual valve open, the pump will operate at its lowest discharge head.
8. Once the pressure gauge reading stabilizes, record the following information:
a. Pressure (from APAI pressure gauge),
b. Flowrate (from City flowmeter),
c. Wetwell level (from City ultrasonic gauge),
d. Elevated storage tank levels (from City remote instruments),
e. Pump RPM (from APAI laser tachometer),
f. Voltage and current for each phase of the motor (Shermco recorder),
g. Power factor (Shermco recorder), and
h. Power usage in kilowatts (Shermco recorder).

Record the time for each measurement.
9. Adjust the manual discharge valve until the discharge pressure reaches the next desired point in the head range. Repeat steps 8 and 9 until the desired points in the head range have been tested.
10. Turn off the test pump; remove the pressure gauge, electrical recording equipment and reflective tape; and install them on the next pump (use a new piece of tape).
11. Repeat steps 4 through 10 until each pump in both pump stations have been tested.

Warning: Testing the shut-off head should be done quickly so that the WTP SCADA will not alarm and automatically shut-off the pump.

## SECTION 2. FIELD TESTING PROTOCOL FOR COMBINED PUMPS

Use the following steps to test the pumps at each pump station (MDS and PZ) in combination, by sequentially turning on an additional pump, in order to create the system curve. Only the flowrate, pressure, and tank levels will be collected during this testing. No electrical data will be recorded.

1. Perform the testing while the City's elevated storage tanks are at their lowest level. This is typically after the morning peak demand.
2. Turn off all the pumps in the pump station.
3. Install the pressure gauge on the combined pump discharge header. The discharge header experiences system pressure and can be used to develop the system curve. If there is no tap on the combined header, discuss with APAI options for where to install the pressure gauge.
4. Verify that personnel and equipment are ready to record the data for Steps 6 and 8.
5. With its manual discharge valve fully open, turn on the smallest pump.
6. Once the pressure reading stabilizes, record the time, pressure, flowrate, clearwell level, wetwell level, and elevated storage tank levels.
7. With its manual discharge valve fully open, turn on the next smallest pump.
8. Once the pressure reading stabilizes, record the time, pressure, flowrate, clearwell level, wetwell level, and elevated storage tank levels.
9. Repeat steps 7 and 8 until all pumps in the pump station are running and the data have been collected.
10. Once the testing at the pump station is complete, resume normal operations.

## Appendix B - Pump Testing Results

## Summary of Main Distribution Pump Station (MDS) Pump Testing

On July 28, 2016, Christopher Haeckler and Robert Weinert performed an in situ pump test at the Main Distribution Pump Station (MDS). The in situ pump curves will be used in the water distribution model to reflect actual pumping conditions. The MDS pump station, located at the Norman Water Treatment Plant, includes four 250 horse power (hp) vertical turbine pumps installed in 1982. Pumps 1 and 3 include vertical frequency drives (VFDs).

Discharge pressure and run time were measured with a pressure transducer attached to each pump's air release valve and connected to a logging software. The butterfly valve was actuated by Andy Bruehl, who also read the flow measurements being recorded by magmeters installed on the outlet of each pump. Speed was measured with a laser tachometer. Input power was measured by attaching a multimeter directly to the electronic components of the pump.

All pumps were run at full speed $(\sim 1770 \mathrm{RPM})$ during the entire pump test. The test began with the controlling butterfly valve on the outlet completely open, in order to measure the pump at the design point. Once discharge pressure leveled out, measurements were taken of time, flow, discharge pressure, shaft speed, and input power at as close to the same point in time as possible. The valve was then closed slightly in order to increase the pressure by $10-15 \mathrm{psi}$, and another set of data points taken. This process was repeated until five data points had been captured for each pump, enough to create a reasonably accurate pump curve. The pressure, flow, and power were corrected for the design shaft speed of 1770 rpm using the affinity law and then plotted.

## Summary of Upper Pressure Zone (PZ) Pump Testing

On August 10, 2016, Robert Weinert returned to test the Upper Pressure Zone Pump Station (PZ). The PZ pump station, also located at the Norman Water Treatment Plant, includes two 200 hp and two 125 hp vertical turbine pumps, installed in 1963 and 1993, respectively. The pump testing process intended to follow the same process outlined in the first paragraph above, with one exception: Instead of flow being measured with a magmeter, it was measured with a Venturi flowmeter that was connected to a SCADA system through which data could be accessed after each test. The flowmeter had been calibrated the previous day by Neal Engleman, using drawn down tests and volumetric calculations in the clearwell. However, the flowmeter didn't return reasonable results on the tests of the first pump. Instead of the $\sim 800-1450$ GPM range that should have been measured, the results were instead between $\sim 1225-1350$ GPM. From this it was concluded that the flowmeter would not return accurate data.

Therefore, two main solutions were proposed:

1. Install an ultrasonic flowmeter to measure results, then correlate the data to a drawn down test done at a later date in order to provide a correction factor. This correction factor was necessary because the pumps did not have the 15 pipe diameter lengths ( 10 upstream and 5 downstream) of undisturbed piping requisite to return accurate data.
2. Run draw down tests for each individual pump. This test would return accurate data, but would require a minimum of four days of labor for Norman employees.

In the end, it was determined that both results would require too much time and effort for the quality and importance of information being obtained. This was especially true because all four pumps are slated to be replaced within the next few years.



Appendix C - Elevated Storage Tank Information

Elevated Storage Tank Summary Table

|  | Brookhaven | Boyd | Cascade | Lindsey | Robinson | HPP |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Transmitter | Microcomm | Rosemount | Microcomm | Rosemount | Rosemount | Unknown |
| Capacity (MG) | 1.5 | 0.5 | 2.0 | 0.5 | 0.5 | 1.0 |
| Bowl Depth (feet) | 42.5 | 40 | 50 | 48.2 | 40 | 40 |
| Diameter (feet) | N/A | N/A | N/A | N/A | 50 | 74 |
| Overflow Elevation (feet MSL) | 1315 | 1320 | 1315 | 1312 | 1315 | 1381 |
| Bottom of Bowl Elevation (feet MSL) | 1273 | 1280 | 1265 | 1264 | 1275 | 1341 |
| Ground Elevation (feet MSL) | 1191 | 1160 | 1190 | 1153 | 1190 | 1186 |
| Height above ground (ft to overflow) | 124 | 160 | 126 | 159 | 125 | 196 |
| Gauge reading at overflow (psi) | 53.7 | 69.3 | 54.3 | 68.8 | 54.1 | 84.7 |
| Height above ground (overall) | 136 | 162 | 130 | N/A | 135 | 205 |
| Gallons per foot capacity ${ }^{1}$ | 35,294 | 12,500 | 46,710 | 10,373 | 12,607 | 32,173 |
| Top elevation | N/A | N/A | N/A | N/A | 1325 | 1391 |
| Year Completed | 1975 | 1965 | 1999 | 1950's | 1954 | 2016 |
| Notes | MDS PS controls off this tower. | Altitude valve. | Altitude valve. | Currently decommissioned due to location and changes in distribution system. | Has mixing system installed in tank and altitude valve. | - |

1. Calculated from the bottom of the bowl elevation to the overflow elevation

## Brookhaven Tower



Appendix C-2


## Boyd Tower



## Cascade Tower



## Robinson Tower



Robinson Water Tower Sketch


 mecruwcol:


 4. Remonaik sif iop simL be 6 MGHES msclumeous rove:







 Coverfow wer-size per specifactors




High Pressure Plane Tower
foovaratov:

Appendix D - Allocation of Future Water Demand Projections

As described in Section 3.2.1, the projected 2025 average day water demand is 17.95 MGD. To model 2025 conditions, this demand must be allocated among existing and future customers and land uses. Projected 2025 annual water demand in the water service area can be divided between existing water use and future development water use (Table 1). The letters from Table 1 correspond to the columns in the next tables in this appendix.

Table 1: Classification of Current and Future Water Uses

| [A] 2025 Water Use | October 2015 - <br> September 2016 <br> Existing System Use | [B] Customer Use (Metered) |  |
| :---: | :---: | :---: | :---: |
|  |  | [C] Other Uses |  |
|  |  | [D] Water Loss |  |
|  | Future Development Use | [E] Metered Flow for New Accounts | Single-Family |
|  |  |  | Multi-Family |
|  |  |  | Office/Retail |
|  |  |  | Industrial/Warehouse |
|  |  |  | Parks |
|  |  |  | Schools |
|  |  |  | Other |
|  |  | [F] Other Uses |  |
|  |  | [G] Water Loss |  |

City staff provided existing water use data for October 2015 through September 2016. As presented in Table 2 (page 2), average day water demands included usage measured by customer meters (10.26 MGD), estimates of other uses ${ }^{1}$ ( 0.24 MGD ), and estimated water loss (1.50 MGD). During this time period, actual average water demands were 108.3 gpcd; however, for planning purposes, the projected water demand is based on 145 gpcd of water use.

The first step in the demand allocation is to multiply the 2015-2016 water demands by a scale factor ( $145 \mathrm{gpcd} / 108.3 \mathrm{gpcd}=1.34$ ) to bring water use for the existing system to 145 gpcd . This results in an estimated water use of 16.06 MGD for the existing water system. Subtracting this from the total projected 2025 water demand, the additional demand developed between 2016 and 2025 is projected to be 1.89 MGD (Table 3, page 2).

The second step in the demand allocation is to assume that water use from future development will occur in the same proportions as water use in the existing system. From October 2015 through September 2016, the City estimated the volume of metered customer uses, other uses, and water loss to be about 85.5 percent, 2.0 percent, and 12.5 percent of total water use, respectively. The total projected demand for future development is allocated by multiplying 1.89 MGD by these percentages (Table 4, page 2).

[^8]Table 2: 2025 Demand Allocation Starting Point

| Labels | Total [A] | Metered [B] | Other [C] | $\begin{gathered} \text { Loss } \\ \text { [D] } \end{gathered}$ | Total Existing $[\mathrm{B}]+[\mathrm{C}]+[\mathrm{D}]$ | Total New $[\mathrm{E}]+[\mathrm{F}]+[\mathrm{G}]$ | Metered [E] | Other [F] | $\begin{gathered} \text { Loss } \\ \text { [G] } \end{gathered}$ |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Base Year (1) | 2025 | 2015-16 | 2015-16 | 2015-16 | 2015-16 | New | New | New | New |
| Projected Base year Population (2) | 123,821 |  |  |  | 110,765 |  |  |  |  |
| Average Day Water Demand, MGD (3) | 17.95 | 10.26 | 0.24 | 1.50 | 12.00 |  |  |  |  |
| Average Day Water Demand, gpcd | 145.0 |  |  |  | 108.3 |  |  |  |  |
| Historical/Current Scale Factor (5) |  |  |  |  |  |  |  |  |  |
| Scaled Average Day Water Demand, MGD (6) |  |  |  |  |  |  |  |  |  |
| Scaled Average Day Water Demand, gpcd (7) |  |  |  |  |  |  |  |  |  |
| Percentage of Existing Water Use (8) |  |  |  |  |  |  |  |  |  |
| Distribute Future Development Use (9) |  |  |  |  |  |  |  |  |  |

Table 3: 2025 Demand Allocation Step 1

| Labels | Total [A] | Metered [B] | Other [C] | $\begin{gathered} \text { Loss } \\ \text { [D] } \end{gathered}$ | Total Existing $[\mathrm{B}]+[\mathrm{C}]+[\mathrm{D}]$ | Total New $[\mathrm{E}]+[\mathrm{F}]+[\mathrm{G}]$ | Metered [E] | Other [F] | $\begin{gathered} \text { Loss } \\ \text { [G] } \end{gathered}$ |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Base Year (1) | 2025 | 2015-16 | 2015-16 | 2015-16 | 2015-16 | New | New | New | New |
| Projected Base year Population (2) | 123,821 |  |  |  | 110,765 |  |  |  |  |
| Average Day Water Demand, MGD (3) | 17.95 | 10.26 | 0.24 | 1.50 | 12.00 |  |  |  |  |
| Average Day Water Demand, gpcd (4) | 145.0 |  |  |  | 108.3 |  |  |  |  |
| Historical/Current Scale Factor (5) |  | 1.34 | 1.34 | 1.34 |  |  |  |  |  |
| Scaled Average Day Water Demand, MGD (6) | 17.95 | 13.73 | 0.32 | 2.01 | 16.06 | 1.89 |  |  |  |
| Scaled Average Day Water Demand, gpcd (7) | 145.0 |  |  |  | 145.0 |  |  |  |  |
| Percentage of Existing Water Use (8) |  |  |  |  |  |  |  |  |  |
| Distribute Future Development Use (9) |  |  |  |  |  |  |  |  |  |

Table 4: 2025 Demand Allocation Step 2

| Labels | Total [A] | Metered [B] | Other [C] | Loss [D] | Total Existing $[\mathrm{B}]+[\mathrm{C}]+[\mathrm{D}]$ | Total New [E]+[F]+[G] | Metered [E] | Other [F] | Loss [G] |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Base Year (1) | 2025 | 2015-16 | 2015-16 | 2015-16 | 2015-16 | New | New | New | New |
| Projected Base year Population (2) | 123,821 |  |  |  | 110,765 |  |  |  |  |
| Average Day Water Demand, MGD (3) | 17.95 | 10.26 | 0.24 | 1.50 | 12.00 |  |  |  |  |
| Average Day Water Demand, gpcd (4) | 145.0 |  |  |  | 108.3 |  |  |  |  |
| Historical/Current Scale Factor (5) |  | 1.34 | 1.34 | 1.34 |  |  |  |  |  |
| Scaled Average Day Water Demand, MGD (6) | 17.95 | 13.73 | 0.32 | 2.01 | 16.06 | 1.89 |  |  |  |
| Scaled Average Day Water Demand, gpcd (7) | 145.0 |  |  |  | 145.0 |  |  |  |  |
| Percentage of Existing Water Use (8) |  | 85.5\% | 2.0\% | 12.5\% |  |  | 85.5\% | 2.0\% | 12.5\% |
| Distribute Future Development Use (9) |  |  |  |  |  |  | 1.62 | 0.04 | 0.24 |

The third step in the demand allocation is to fill in Table 5 with results from the previous step.
Table 5: 2025 Demand Allocation Step 3

| Labels | Total [A] | Metered [B] | Other [C] | $\begin{gathered} \text { Loss } \\ \text { [D] } \\ \hline \end{gathered}$ | Total Existing $[\mathrm{B}]+[\mathrm{C}]+[\mathrm{D}]$ | $\begin{gathered} \text { Total New } \\ {[\mathrm{E}]+[\mathrm{F}]+[\mathrm{G}]} \end{gathered}$ | Metered [E] | Other [F] | $\begin{gathered} \text { Loss } \\ \text { [G] } \\ \hline \end{gathered}$ |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Base Year (1) | 2025 | 2015-16 | 2015-16 | 2015-16 | 2015-16 | New | New | New | New |
| Projected Base year Population (2) | 123,821 |  |  |  | 110,765 |  |  |  |  |
| Average Day Water Demand, MGD (3) | 17.95 | 10.26 | 0.24 | 1.50 | 12.00 |  |  |  |  |
| Average Day Water Demand, gpcd (4) | 145.0 |  |  |  | 108.3 |  |  |  |  |
| Historical/Current Scale Factor (5) |  | 1.34 | 1.34 | 1.34 |  |  |  |  |  |
| Scaled Average Day Water Demand, MGD (6) | 17.95 | 13.73 | 0.32 | 2.01 | 16.06 | 1.89 |  |  |  |
| Scaled Average Day Water Demand, gpcd (7) | 145.0 |  |  |  | 145.0 |  |  |  |  |
| Percentage of Existing Water Use (8) |  | 85.5\% | 2.0\% | 12.5\% |  |  | 85.5\% | 2.0\% | 12.5\% |
| Distribute Future Development Use (9) | 17.95 | 13.73 | 0.32 | 2.01 | 16.06 | 1.89 | 1.62 | 0.04 | 0.24 |

To model the water distribution system for 2025 conditions, the projected metered use for new development was allocated to different locations in the water service area. The Land Use Plan, existing developments, and future (platted) developments were reviewed to project future numbers of water connections and future acreage by land uses. The City also provided information about planned parks, schools, and a University of Oklahoma (OU) development. The projected metered use was then allocated for new development to the land use categories as shown in Table 6 (page 4).

The primary assumptions in the allocation process are:

- Projections based on numbers of connections:
- 17.25 multi-family units per multi-family water connection. This was estimated from the average day water use for a multi-family connection (1,859 gallons per day) and the average day water use for an independently metered apartment (102 gallons per day), with adjustments for differences in irrigation between these types of connections.
- For each category, the unit water use was estimated to be the average of the 2015-16 average day water uses for all existing meters.
- Based on these procedures and the estimated population densities, these assumptions result in projected single-family water use of 77 gpcd and projected multi-family water use of 56 gpcd . Based on literature values and experience with other utilities, these are reasonable estimates.
- Projections based on information provided by the City
- The City provided information on parks and schools that are expected to be developed before 2025. For each new park and school, the City also identified an existing park or school with expected similar water use. Metered data from these comparable properties were used to estimate future water use at the new parks and schools.

Table 6: Allocation of Projected Average Water Demand by Land Use Category

| Projections Based on Numbers of Connections |  |  |  |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Land Use Category | Projected <br> 2016-2025 <br> New <br> Housing <br> Units | Estimated <br> Housing <br> Units per <br> Connection | Projected <br> 2016-2025 <br> New <br> Connections | 2015-16 <br> Category <br> Average <br> Usage* <br> (gal/conn/day) | Projected <br> Add'I <br> Average <br> Demand (mgd) | Projected Add'I Average Demand (\%) | People per Unit | Per Capita Use (gpcd) |
| Single-Family | 3,892 | 1.00 | 3,892 | 195 | 0.759 | 46.93\% | 2.55 | 77 |
| Multi-Family | 1,367 | 17.25 | 79 | 1,859 | 0.147 | 9.10\% | 1.91 | 56 |
| Subtotal |  |  |  |  | 0.907 | 56.03\% |  |  |
| Projections Based on Information from City |  |  |  |  |  |  |  |  |
| Land Use Category | Identified <br> Potential <br> Acreage | Projected <br> 2016-2025 <br> New <br> Acreage | Connections per Acre | $\begin{gathered} \text { 2015-16 } \\ \text { Category } \\ \text { Usage } \\ \text { (gal/conn/day) } \end{gathered}$ | Projected Add'I <br> Average Demand (mgd) | Projected Add'I Average Demand (\%) |  |  |
| Parks | 379.3 | 153.0 |  |  | 0.011 | 0.67\% |  |  |
| Schools | 20.0 | 40.0 |  |  | 0.001 | 0.07\% |  |  |
| OU Jenkins/Timberdell | 20.0 | 20.0 |  |  | 0.068 | 4.18\% |  |  |
| Subtotal |  |  |  |  | 0.080 | 4.92\% |  |  |
| Projections Based on Acreage |  |  |  |  |  |  |  |  |
| Land Use Category | Identified <br> Potential <br> Acreage | Projected <br> 2016-2025 <br> New <br> Acreage | Connections per Acre | 2015-16 <br> Category 69th Percentile Usage* (gal/conn/day) | Projected <br> Add'I <br> Average <br> Demand (mgd) | Projected Add'I Average Demand (\%) |  |  |
| Office/Retail | 745.40 | 436.5 | 1.421 | 396 | 0.246 | 15.20\% |  |  |
| Industrial/Warehouse | 220.68 | 98.7 | 0.443 | 8,817 | 0.386 | 23.85\% |  |  |
| Subtotal |  |  |  |  | 0.632 | 39.05\% |  |  |
| TOTAL |  |  |  |  | 1.618 |  |  |  |

- A 20-acre future OU development with 1,200-bed student housing and an office building was also identified by the City. Unit water use of 56 gallons per bed per day was assumed for student housing (same value as multi-family per capita water use). Projected water use for the office building is described in the next bulleted items.
- Projections based on land use acreage:
- For each category (office/retail and industrial/warehouse), the number of connections per acre was projected by identifying the existing total acreage of this land use and existing total number of meters for developed parcels with similar land use.
- Water use in the office/retail and industrial/warehouse land use categories is highly variable, depending on the property, with the average water use skewed by a few large water users (For each category, the average of the average day water use for all meters is about the 83rd percentile value). In addition, there are only 19 existing connections in the industrial/warehouse category that had metered 2015-16 water use. For these reasons, smaller percentile values were used that would also make the total allocated metered water use equal the amount projected based on the 2060 Strategic Water Supply Plan (Tables 2 and 3).:
- 69th percentile average day water use for existing meters for the projections without water conservation and
- 63rd percentile average day water use for existing meters for the projections with water conservation.

Appendix E - Hydrant Flow and Pressure Testing Procedures and Locations

## 1 Hydraulic Model Calibration Procedures

This memorandum addresses Task B-3c from the project scope. It provides Norman Utilities Authority (NUA) with protocols to use for system flow and pressure testing during the summer of 2016. Pressure testing should be performed after the main distribution system (MDS) is separated from the new upper pressure zone.

### 1.1 INTRODUCTION

A successfully calibrated hydraulic model can be leveraged to evaluate the impact of proposed operational changes on the water system and to guide capital improvement decisions. To achieve an acceptable calibration, comparison and refinement of the model results based on measured field conditions is fundamental. Typically, a model is first calibrated to steady-state conditions, or a fixed set of conditions such as the peak demand hour on a maximum demand day. Then the model is calibrated to reflect extended-period simulation (EPS) conditions, or variable demand conditions that capture diurnal patterns during a period of high demand. Both steady-state and EPS simulations use historical conditions and field data for calibration. After calibration, a correlation between field data and model results of 5 to 10 percent is desirable ${ }^{1}$.

### 1.2 FLOW AND PRESSURE TESTING

Flow and pressure testing will be conducted to support the static and residual calibration of a newly created water distribution model for NUA. Testing will be performed during the summer of 2016 at approximately eighteen hydrant sites throughout the City. Four fire hydrants will be utilized at each site. Two fire hydrants (Q1, Q2) will be used to measure flow with a hydrant Pitot gauge. This device measures flow by recording the velocity head in the flow stream. Two additional fire hydrants will be used to measure pressure. A data logger and pressure gauge will be installed at both of the pressure hydrants. NUA supervisory control and data acquisition (SCADA) equipment, including all pump station flows and pressures and tank levels, will be used to collect additional system data while each test is being conducted. Figure 1 displays the proposed locations for hydrant testing. Appendix A provides a blank hydrant test paper form with pertinent information that should be collected during each hydrant test.

Each flow test is estimated to take one hour, which includes installing the equipment, running the test under five flow conditions, and disassembling the equipment. A crew of three can quickly and safely perform the testing procedure at each site. Generally one person is stationed at each of the two flow gauges, and a third person is stationed at one of the pressure gauges.

### 1.2.1 Calibration of Testing Equipment

Before beginning the flow and pressure testing, all testing equipment should be gathered and proper operation should be verified. The hydrant flow gauge (Pitot gauge) that will be used to measure flow must be calibrated. Calibration of the Pitot gauge is most effective when a 'dead-weight' tester is used. Following gauge calibration, comparison of gauge pressure to the dead-weight tester pressure should fall within 0.5 psi .

The timekeeping of all recording equipment needs to be synchronized (or at a minimum, the difference in their recorded time must be known). This includes all data logger clocks as well as staff watches. This is critical so that the data recorded for each test can be grouped by time-step correctly.

[^9]
### 1.2.2 Installation of Testing Equipment Basic Site Information

The proposed hydrant test sites are displayed on a map in Figure 1, and site detail maps with specific hydrant assignments are provided in Appendix B. At the hydrant test site, the two pressure test hydrants should each be outfitted with a data logger (set for data readings of 30 -seconds or finer) and a pressure gauge. Each of the flow hydrants should be outfitted with a calibrated Pitot gauge. The use of a flow diffuser (wire-cage or similar) after the Pitot gauge is recommended, and the environment around the hydrant flow trajectory should be checked and modified as needed to prevent site damage and erosion. Note that the flow hydrants must be unique from the pressure hydrants and should never be used to measure system pressure - friction loss in the hydrant valve and barrel will cause errors.

### 1.2.3 Basic Site Information

Once the testing equipment is installed, fill out the hydrant test paper form with the basic site and gauge information. At the pressure hydrants (P1 and P2), use a tape measure to record the height above ground level for each gauge. Take a photo at each of the flow hydrants (Q1, Q2) to document initial site conditions, and record the photo number. At all hydrants, collect a GPS coordinate. Verify that a hydrant wrench of the correct size is available at each of the flow hydrants.

### 1.2.4 Static Condition Measurement

The static condition represents the pressure during typical demands with no hydrants flowing. Static pressure is measured for two 5-minute periods at each pressure test hydrant, once before the three flow condition tests, and once after the flow tests are completed. During the static pressure measurements, both flow hydrants (Q1, Q2) remain closed and only pressure is measured at the two pressure hydrants ( $\mathrm{P} 1, \mathrm{P} 2$ ). The goal of the static pressure measurement is to accurately record field pressure data under baseline conditions, when no additional demand is present. It is used to validate major connectivity and operational boundary conditions in the model like the head in elevated tanks. The second static pressure measurement checks that the system returns to the conditions obtained before the test was initiated.

### 1.2.5 Residual Condition Measurement

The residual condition represents the pressure while the flow hydrant(s) are open. After the first static condition measurement, flow and pressures are measured for 5-minute periods for three different flow conditions:

1. Water is released from flow hydrant Q1. Meanwhile, flow is recorded at hydrant Q1 and pressure is recorded continuously at pressure hydrants P1 and P2. Confirm that pressure drop from static conditions at P1 and P2 is sufficient. ${ }^{2}$ Verify that measured pressure at P1 and P2 does not drop below 20 psi.
2. Water is released from Q1 and Q2 simultaneously. Meanwhile, flow is recorded at Q1 and Q2, and pressure is recorded at P1 and P2. Confirm that the maximum pressure drop is sufficient during this flow condition. ${ }^{2}$ Verify that the measured pressure at P1 and P2 does not drop below 20 psi.
3. Hydrant Q1 is closed and flow is stopped. Water continues to be released from Q2. Meanwhile, flow is recorded at Q2 and pressure is recorded at P1 and P2. Verify that measured pressure at P1 and P2 does not drop below 20 psi.

After the three flow tests are complete, make the second static condition measurement at P1 and P2. Confirm that the second static measurement is near the first static measurement.

[^10]

The goal of the residual pressure measurements is to accurately record field pressure data under active flow conditions, when an outflow higher than any peak hour demand is present. The flow and pressure data will be used for calibration of the model under the same conditions seen in the field at the time the tests were performed.

The hydrant flow and pressure testing procedure is conceptually displayed in Table 1. Hydrant flows that are higher than any peak hour demand will typically be produced when each test hydrant is flowing. The flows from the residual calibration tests will be used to evaluate the hydraulic model for connectivity (both pipes and valve status), pump operation, roughness factors, and system response times. Pipe friction factors (Hazen Williams CFactors) will be adjusted so that the model represents data collected in the field.

Table 1: Hydrant Flow Test Procedures

| Location | Time Periods |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: |
|  | Time 1 | Time 2 | Time 3 | Time 4 | Time 5 |
|  | Prior to Hydrant Test | 0-5 Minutes | 5-10 Minutes | $10-15$ <br> Minutes | After Hydrant Test |
| Pressure Hydrant P1 | Record Static Pressure (Start) | Record <br> Residual <br> Pressure | Record <br> Residual <br> Pressure | Record <br> Residual <br> Pressure | Record Static <br> Pressure (End) |
| Pressure Hydrant P2 | Record Static Pressure (Start) | Record <br> Residual <br> Pressure | Record <br> Residual <br> Pressure | Record <br> Residual <br> Pressure | Record Static <br> Pressure (End) |
| $\begin{gathered} \text { Flow } \\ \text { Hydrant Q1 } \end{gathered}$ | OFF | ON - Record Residual Flow | ON - Record Residual Flow | OFF | OFF |
| Flow Hydrant Q1 | OFF | OFF | ON - Record Residual Flow | ON - Record Residual Flow | OFF |

### 1.3 SUMMARY

The use of continuously-recording pressure data loggers in conjunction with simultaneously releasing flow from multiple flow hydrants is recommended by the American Water Works Association (AWWA) and others as an efficient and accurate way to conduct hydrant flow test. Benefits of this approach for the City of Norman, OK include faster tests, a feasible crew size, budget-minded equipment costs, and more data that can be used for model calibration.

Performing flow testing during the summer, higher demand period, is ideal. It should also be performed after the upper pressure zone is separated from the MDS in order to produce an up to date model. APAI will also need SCADA data for the period when hydrant flow testing was performed for accurate model calibration. Required SCADA data includes the following:

- Elevated and ground storage tank levels,
- Pump on/off status and flows, and
- Well on/off status and flows.


## APPENDIX A

Hydrant test form

|  Hydra <br> City <br> Water S <br> Date: $\longrightarrow$ <br> Test Site ID:  <br> Notes:  | Test Data Form Norman, OK stem Master Plan <br> Site Description: |
| :---: | :---: |
| Flow Hydrant - Q1 <br> Hydrant ID: <br> Location: <br> GPS Lat/Lon: <br> Photo ID: <br> Time 1: <br> Pressure 1: $\qquad$ psi <br> Time 2: $\qquad$ am / pm <br> Pressure 2: $\qquad$ psi <br> Time 3: $\qquad$ am / pm <br> Pressure 3: $\qquad$ psi <br> Time 4: $\qquad$ am / pm <br> Pressure 4: $\qquad$ psi <br> Time 5: $\qquad$ am / pm <br> Pressure 5: $\qquad$ psi <br> Notes: | Pressure Hydrant - P1 <br> Hydrant ID: $\qquad$ <br> Location: $\qquad$ <br> GPS Lat/Lon: $\qquad$ <br> Photo ID: $\qquad$ <br> Gauge height above ground (ft): $\qquad$ <br> Data Logger ID: $\qquad$ <br> Time ON: $\qquad$ am / pm <br> Begin Static Pressure: $\qquad$ psi <br> Time OFF: $\qquad$ am / pm <br> End Static Pressure: $\qquad$ psi <br> Notes: |
| Flow Hydrant - Q2 <br> Hydrant ID: <br> Location: $\qquad$ <br> GPS Lat/Lon: $\qquad$ <br> Photo ID: <br> Time 1: $\qquad$ am / pm <br> Pressure 1: $\qquad$ psi <br> Time 2: $\qquad$ am / pm <br> Pressure 2: $\qquad$ psi <br> Time 3: $\qquad$ am / pm <br> Pressure 3: $\qquad$ psi <br> Time 4: $\qquad$ am / pm <br> Pressure 4: $\qquad$ psi <br> Time 5: $\qquad$ am / pm <br> Pressure 5: $\qquad$ psi <br> Notes: | Pressure Hydrant - P2 <br> Hydrant ID: $\qquad$ <br> Location: $\qquad$ <br> GPS Lat/Lon: $\qquad$ <br> Photo ID: $\qquad$ <br> Gauge height above ground (ft): $\qquad$ <br> Data Logger ID: $\qquad$ <br> Time ON: $\qquad$ am / pm <br> Begin Static Pressure: $\qquad$ psi <br> Time OFF: $\qquad$ am / pm <br> End Static Pressure: $\qquad$ psi <br> Notes: |

## APPENDIX B

## Site detail maps

## Test Site 1 <br> (Land Use = RES)




















> Appendix F - Friction Factors used in Water Model

Friction Factors used in Model (Hazen-Williams C-Factors)

| Pipe Material | Diameter (inches) |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | 1 | 2 | 3 | 4 | 6 | 8 | 10 | 12 | 14 | 15 | 16 | 24 | 30 | 36 | 42 |
| Asbestos Cement | - | - | - | 142 | 145 | 145 | 147 | 147 | - | - | - | - | - | - | - |
| Cast Iron | 100 | 100 | 100 | 100 | 102 | 107 | 107 | 107 | 109 | - | 109 | 110 | 110 | - | - |
| Concrete | - | - | - | - | - | - | 120 | 120 | 120 | 120 | 120 | 120 | 120 | - | - |
| Copper | 140 | 140 | - | - | 149 | - | - | - | - | - | - | - | - | - | - |
| Ductile Iron | 130 | 130 | 130 | 130 | 133 | 133 | 133 | 138 | 138 | - | 138 | 140 | 140 | - | 140 |
| Galvanized Pipe | 120 | 120 | - | - | 120 | - | - | - | - | - | - | - | - | - | - |
| High Density Polyethylene | 150 | 150 | - | - | 150 | 150 | 150 | 150 | 150 | - | 150 | 150 | 150 | - | - |
| Polyvinyl Chloride | 134 | 134 | 142 | 142 | 145 | 145 | 147 | 147 | 147 | 147 | 147 | 150 | 150 | - | 150 |
| Reinforced Concrete Pipe | - | - | - | - | 120 | - | - | - | - | - | - | 120 | - | - | - |
| Steel | - | 120 | - | 120 | - | - | - | - | - | - | - | - | - | - | - |

## Appendix G - Calibration Detailed Results

Test Site: 001

| TEST TYPE <br> NODE | $\begin{gathered} P_{2708} \\ 2770 \end{gathered}$ | P AVG (psi) P MAX (psi) P MIN (psi) |  | Model <br> 43.9 <br> 49.2 <br> 16.7 | Observed Diff (\%; abs) |  | TEST TYPENODE ID | $\begin{gathered} P_{2} 2711 \\ 2701 \end{gathered}$ | P AVG (psi) P MAX (psi) P MIN (psi) |  | Model43.249.715.3 | Observed Diff \%; abs) |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  |  |  | 1\% | 42.1 |  |  |  |  | 3\% |
|  |  |  |  | 52.9 | -3.8 | 49.7 |  |  |  |  | -0.1 |
|  |  |  |  | 22.3 | -5.6 |  |  | 19.0 |  |  | -3.7 |
|  |  |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  | Pressure (0s) |  |  |  |  |  |  |
| ${ }_{\substack{\text { Pressure }}}^{\substack{\text { Pr }}}$ |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |  |  |  |  |  |
|  | tive | ${ }_{\text {120. }}^{10.00}$ |  |  |  |  |  | o930 |  | 10.00 |  | ${ }^{1100}$ |  |
|  |  | Model |  |  | Difference |  |  |  | Model | Observed |  | Difference |  |
|  | Date/Time | Pressure (psi) | Observed Pressure (psi) | filter | (psi) |  |  | Date/Time | Pressure (psi) | Observed Pressure (psi) | filter |  |  |
|  | 8/23/20169:24 | ${ }^{48.55}$ | 47.9 | 1 | 0.65 |  |  | 8/23/20169:24 | 49.12 |  |  |  |  |
|  | 8/23/20169:26 | 48.56 4.57 | 47.1 | 1 | 1.46 |  |  | 8/23/20169:26 | 49.12 |  |  |  |  |
|  | 8/23/20169:28 | 48.57 | 43.9 | 1 | 4.67 |  |  | 8/23/20169:28 | 49.14 | 48.7 | 1 | 0.44 |  |
|  | 8/23/20169:30 | 48.57 | 46.5 | 1 | 2.07 |  |  | 8/23/20169:30 | 49.14 | 49.4 | 1 | -0.26 |  |
|  | 8/23/20169:32 | 48.58 | 47 | 1 | ${ }^{1.58}$ |  |  | 8/23/20169:32 | 49.15 | 48.3 | 1 | 0.85 |  |
|  | 8/23/20169:34 | 48.58 | 47.9 | 1 | 0.68 |  |  | 8/23/20169:34 | 49.15 | 49.2 | 1 | -0.05 |  |
|  | 8/23/20169:36 | 48.59 | 46.6 | 1 | 1.99 |  |  | 8/23/20169:36 | 49.15 | 48.2 | 1 | 0.95 |  |
|  | 8/23/20169:38 | 48.6 | 47.2 | 1 | 1.4 |  |  | 8/23/20169938 | ${ }_{4}^{49.16}$ | 46.3 | 1 | ${ }^{2.86}$ |  |
|  | 8/23/20169:40 | 48.61 | 47 | 1 | 1.61 |  |  | 8/23/20169:40 | 49.17 | 49.1 | 1 | 0.07 |  |
|  | 8/23/20169:42 | 48.62 | 47.5 | 1 | 1.12 |  |  | 8/23/20169:42 | 49.18 | 49 | 1 | 0.18 |  |
|  | 8/23/20169:44 | 48.62 | 45.9 | 1 | 2.72 |  |  | 8/23/201699:44 | 49.18 | ${ }^{48.6}$ | 1 | 0.58 |  |
|  | 8/23/20169:46 | 48.63 | 46.4 | 1 | 2.23 |  |  | 8/23/20169:46 | 49.2 | 48.5 | 1 | 0.7 |  |
|  | 8/23/20169:48 | 48.63 | 47.8 | 1 | ${ }^{0.83}$ |  |  | 8/23/20169:48 | 49.2 | 47.7 | 1 | 1.5 |  |
|  | 8/23/20169:50 | 48.65 | 48.5 | 1 | 0.15 |  |  | 8/23/20169:50 | 49.22 | 48.4 | 1 | 0.82 |  |
|  | 8/23/20169:52 | 48.65 48.65 | 46.4 478 | 1 | 2.25 0.85 1 |  |  | $8 / 23 / 201699: 52$ $8 / 23 / 2016954$ | 49.21 | 48.8 48 | 1 | 0.41 1.22 |  |
|  | $8 / 23 / 20169: 54$ $8 / 23 / 2016956$ | 48.65 48.66 | 47.8 46.9 | 1 | 0.85 1.76 1.75 |  |  | $8 / 23 / 201699.54$ $8 / 23 / 2016956$ | 49.22 49.23 | 48 48.7 | 1 | 1.22 0.53 |  |
|  | 8/23/20169:98 | ${ }_{48.67}$ | 46.9 47.5 | 1 | ${ }_{1.17}^{1.16}$ |  |  | 8/23/20169:56 | 49.23 49.24 | ${ }_{49.7}^{48.7}$ | 1 | 0.53 -0.46 |  |
|  | 8/23/2016 10:00 | 48.67 | 47.4 | 1 | 1.27 |  |  | 8/23/2016 10:00 | 49.24 | 48.3 | 1 | 0.94 |  |
|  | 8/23/2016 10:02 | 48.68 | 48.1 | 1 | 0.58 |  |  | 8/23/2016 10:02 | 49.25 | 41 | 1 | 8.25 |  |
|  | 8/23/2016 10:04 | 33.13 | ${ }^{35}$ | 1 | -1.87 |  |  | 8/23/2016 10:04 | 32.81 | 33.7 | 1 | -0.89 |  |
|  | 8/23/2016 10:06 | 33.12 | 34.3 | 1 | ${ }^{-1.18}$ |  |  | 8/23/2016 10:06 | 32.81 | 33.4 | 1 | -0.59 |  |
|  | 8/23/2016 10:08 | 33.1 | 34.6 | 1 | -1.5 |  |  | 8/23/2016 10:08 | 32.78 | 33.4 | 1 | -0.62 |  |
|  | 8/23/2016 10:10 | 33.06 | 34.3 | 1 | -1.24 |  |  |  | 32.74 156 | ${ }^{23}$ | 1 | 9.74 |  |
|  | 8/23/2016 10:12 | 16.8 | 23.7 | 1 | -6.9 |  |  | 8/23/2016 10:12 | ${ }^{15.36}$ | 19.3 | 1 | -3.94 |  |
|  | 8/23/2016 10:14 | 16.77 16.76 | 22.6 22.3 | 1 | -5.83 |  |  | 8/23/201610:14 | 15.33 1532 |  | 1 | -3.67 |  |
|  | 8/83/2016 10:16 | 16.76 16.71 | ${ }_{22.3}^{22.3}$ | 1 | $\stackrel{-5.54}{-5.59}$ |  |  | 8/8/2/2016 10:16 | 15.32 15.28 | 19.3 19.5 | 1 | -3.98 -4.22 |  |
|  | 8/23/2016 10:20 | 42.45 | 41.9 | 1 | 0.55 |  |  | 8/23/2016 10:20 | 42.24 | 34.2 | 1 | 8.04 |  |
|  | 8/23/2016 10:22 | 42.44 | 37.2 | 1 | 5.24 |  |  | 8/23/2016 10:22 | 42.23 | 34.3 | 1 | 7.93 |  |
|  | 8/23/2016 10:24 | 42.45 | 37.3 | 1 | 5.15 |  |  | 8/23/2016 10:24 | 42.24 | 37.5 | 1 | 4.74 |  |
|  | 8/23/2016 10:26 | 42.46 | 37.9 | 1 | 4.56 |  |  | 8/23/2016 10:26 | 42.25 | 35.3 | 1 | 6.95 |  |
|  | 8/23/2016 10:28 | 48.64 | 52.9 | 1 | -4.26 |  |  | 8/23/2016 10:28 | 49.21 | 48.4 | 1 | 0.81 |  |
|  | 8/23/2016 10:30 | 48.69 | 48.2 | 1 | 0.49 |  |  | 8/23/2016 10:30 | 49.26 | 0.3 |  |  |  |
|  | 8/23/2016 10:32 | 48.7 | 47.5 | 1 | 1.2 |  |  | 8/23/2016 10:32 | 49.27 | 0.2 |  |  |  |
|  | 8/23/2016 10:34 | 48.73 | 47.9 | 1 | 0.83 |  |  | 8/23/2016 10:34 | 49.3 | 0.2 |  |  |  |
|  | 8/23/2016 10:36 | 48.74 | 49.3 | 1 | ${ }^{-0.56}$ |  |  | 8/23/2016 10:36 | 49.31 | 49.2 | 1 | 0.11 |  |
|  | 8/23/2016 10:38 | 48.77 | 48.4 | 1 | 0.37 |  |  | 8/23/2016 10:38 | 49.34 | 48.7 | 1 | 0.64 |  |
|  | 8/23/2016 10:40 | 48.8 | 48.3 | 1 | 0.5 |  |  | 8/23/2016 10:40 | 49.37 | 48.4 | 1 | 0.97 |  |
|  | 8/23/2016 10:42 | 48.82 | 48.3 | 1 | 0.52 |  |  | 8/23/2016 10:42 | 49.38 | 49.6 | 1 | -0.22 |  |
|  | 8/23/2016 10:44 | 48.84 | 48.6 | 1 | 0.24 |  |  | 8/23/2016 10:44 | 49.4 | 49.1 | 1 | 0.3 |  |
|  | 8/23/2016 10:46 | 48.87 | 48.3 | 1 | 0.57 |  |  | 8/23/2016 10:46 | 49.44 | 49 | 1 | 0.44 |  |
|  | 8/23/2016 10:48 | 49.08 | 48.3 | 1 | 0.78 |  |  | 8/23/2016 10:48 | 49.65 | 49.6 | 1 | 0.05 |  |
|  |  | 49.1 49.15 | 47.1 48.3 | 1 | $\underset{0.85}{2}$ |  |  | 8/8/23/2016 10:50 | 49.67 49.72 |  |  |  |  |
|  | 8/23/2016 10:52 | 49.15 | 48.3 | 1 | 0.85 |  |  | 8/23/2016 10:52 | 49.72 |  |  |  |  |

## Test Site: 002



|  |  |  | Model | Observed | Diff (\%; abs) |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| TEST TYPE | P1 | P AVG (psi) | 42.4 | 45.0 | $-6 \%$ |
| NODE ID | 26964 | P MAX (psi) | 49.8 | 48.8 | 1.0 |
|  |  | P MIN (psi) | 23.9 | 36.1 | -12.2 |

Run 054 - 042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1\#4


|  | Model | Observed |  | 24-Aug | 12:35 |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Date/Time | Pressure (psi) | Observed Pressure (psi) | filter |  | 13:10 |
| 8/24/2016 12:34 | 49.77 | -0.4 |  |  |  |
| 8/24/2016 12:36 | 49.77 | -0.4 |  |  |  |
| 8/24/2016 12:38 | 49.78 | -0.4 |  |  |  |
| 8/24/2016 12:40 | 49.78 | 48.2 | 1 | 1.58 |  |
| 8/24/2016 12:42 | 49.79 | 48 | 1 | 1.79 |  |
| 8/24/2016 12:44 | 49.79 | 48 | 1 | 1.79 |  |
| 8/24/2016 12:46 | 49.8 | 48.8 | 1 | 1 |  |
| 8/24/2016 12:48 | 49.8 | 48.4 | 1 | 1.4 |  |
| 8/24/2016 12:50 | 49.8 | 48.4 | 1 | 1.4 |  |
| 8/24/2016 12:52 | 23.92 | 36.3 | 1 | -12.38 |  |
| 8/24/2016 12:54 | 23.91 | 36.4 | 1 | -12.49 |  |
| 8/24/2016 12:56 | 23.89 | 36.1 | 1 | -12.21 |  |
| 8/24/2016 12:58 | 23.89 | 37.7 | 1 | -13.81 |  |
| 8/24/2016 13:00 | 49.72 | 48.2 | 1 | 1.52 |  |
| 8/24/2016 13:02 | 49.73 | 48.3 | 1 | 1.43 |  |
| 8/24/2016 13:04 | 49.73 | 48.2 | 1 | 1.53 |  |
| 8/24/2016 13:06 | 49.74 | 48.4 | 1 | 1.34 |  |




|  | Model | Observed |  | 24-Aug | 12:45 |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Date/Time | Pressure (psi) | Observed Pressure (psi) | filter |  | 13:05 |
| 8/24/2016 12:34 | 66.22 | 64.7 | 1 | 1.52 |  |
| 8/24/2016 12:36 | 66.23 | 65.1 | 1 | 1.13 |  |
| 8/24/2016 12:38 | 66.24 | 65.2 | 1 | 1.04 |  |
| 8/24/2016 12:40 | 66.24 | 65 | 1 | 1.24 |  |
| 8/24/2016 12:42 | 66.25 | 65.3 | 1 | 0.95 |  |
| 8/24/2016 12:44 | 66.25 | 64.9 | 1 | 1.35 |  |
| 8/24/2016 12:46 | 66.26 | 65.5 | 1 | 0.76 |  |
| 8/24/2016 12:48 | 66.26 | 64.8 | 1 | 1.46 |  |
| 8/24/2016 12:50 | 66.26 | 64.7 | 1 | 1.56 |  |
| 8/24/2016 12:52 | 40.3 | 53.1 | 1 | -12.8 |  |
| 8/24/2016 12:54 | 40.28 | 53.4 | 1 | -13.12 |  |
| 8/24/2016 12:56 | 40.26 | 53.2 | 1 | -12.94 |  |
| 8/24/2016 12:58 | 40.26 | 54.5 | 1 | -14.24 |  |
| 8/24/2016 13:00 | 66.18 | 64.8 | 1 | 1.38 |  |
| 8/24/2016 13:02 | 66.18 | 64.9 | 1 | 1.28 |  |
| 8/24/2016 13:04 | 66.19 | 65 | 1 | 1.19 |  |
| 8/24/2016 13:06 | 66.2 | 65.4 | 1 | 0.8 |  |

## Test Site: 004

|  |  |  | Model | Observed | Diff (\%; abs) |
| :---: | :---: | :---: | :---: | :---: | :---: |
| TEST TYPE | P1 | P AVG (psi) | 61.7 | 65.7 | $-6 \%$ |
| NODE ID | 26979 | P MAX (pss) | 78.1 | 76.2 | 2 |
|  |  | P MIN (psi) | 24.9 | 42.8 | -18 |

Run 054 - 042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1\#4
$\square \square$


Date/Time Pressure (psi) Observed Pressure (psi) filter 25-Aug
75.5
76.2
76.1
75.6
75.5
75.6
42.8
43.1
43.3
43.5
76.2
75.8
75.1


| 75.2 | 1 | 1.89 |
| :--- | :--- | :--- |
| 76.1 | 1 | 1.99 |


| 75.6 | 1 | 1.99 |
| :--- | :--- | :--- |
| 75.5 | 1 | 2.49 |


| 75.5 | 1 | 2.59 |
| :--- | :--- | :--- |



- 43.1

Test Site: 005

|  |  |  | Model | Observed | Diff (\%; abs) |
| :---: | :---: | :---: | :---: | :---: | :---: |
| TEST TYPE | P1 | P AVG (psi) | 53.0 | 54.2 | $-2 \%$ |
| NODE ID | 26968 | P MAX (psi) | 59.0 | 57.5 | 1.5 |
|  |  | P MIN (psi) | 41.1 | 47.5 | -6.4 |





Test Site: 007


Test Site: 009

|  |  |  | Model | Observed | Diff (\%; abs) |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| TEST TYPE | P1 | P AVG (psi) | 53.5 | 53.7 | $0 \%$ |
| NODE ID | 26983 | P MAX (psi) | 61.8 | 60.4 | 1 |
|  |  | P MIN (psi) | 43.2 | 44.1 | -1 |

Run 054 - 042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1*4
08/23/2016 00:00:00 Hydrant 26983 Run 054-042127; wells FLow; MDS and PZ AUTO; 4/3/17 GIS Edits

$\left.\begin{array}{ccccc}\text { Date/Time } & \begin{array}{l}\text { Model } \\ \text { Pressure (psi) }\end{array} & \begin{array}{l}\text { Observed } \\ \text { Observed }\end{array} & \\ \text { Pressure (psi) }\end{array}\right)$

Test Site: 010


Test Site: 011

|  |  |  | Model | Observed | Diff (\%; abs) |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| TEST TYPE | P1 | P AVG (psi) | 72.3 | 72.3 | 0\% |
| NODE ID | 26956 | P MAX (psi) | 76.7 | 81.4 | -4.7 |
|  |  | P MIN (psi) | 65.3 | 66.0 | -0.7 |

Run 054 - 042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1*4


| Date/Time | Model | Observed | filter |  |
| :---: | :---: | :---: | :---: | :---: |
|  | Pressure (psi) | Observed Pressure (psi) |  |  |
| 8/30/2016 9:12 | 76.63 | -0.3 |  |  |
| 8/30/2016 9:14 | 76.65 | -0.3 |  |  |
| 8/30/2016 9:16 | 76.63 | 75.5 | 1 | 1.13 |
| 8/30/2016 9:18 | 76.65 | 75.1 | 1 | 1.55 |
| 8/30/2016 9:20 | 76.65 | 75 | 1 | 1.65 |
| 8/30/2016 9:22 | 76.67 | 74.3 | 1 | 2.37 |
| 8/30/2016 9:24 | 76.67 | 75.3 | 1 | 1.37 |
| 8/30/2016 9:26 | 65.36 | 67.8 | 1 | -2.44 |
| 8/30/2016 9:28 | 65.35 | 66 | 1 | -0.65 |
| 8/30/2016 9:30 | 65.33 | 66.8 | 1 | -1.47 |
| 8/30/2016 9:32 | 65.31 | 67 | 1 | -1.69 |
| 8/30/2016 9:34 | 65.31 | 66.5 | 1 | -1.19 |
| 8/30/2016 9:36 | 76.64 | 81.4 | 1 | -4.76 |
| 8/30/2016 9:38 | 76.64 | 74.8 | 1 | 1.84 |
| 8/30/2016 9:40 | 76.66 | 74.1 | 1 | 2.56 |

Test Site: 012

| P1 |
| :--- |
| TEST TYPE <br> NODE ID |

Date/Time Model Observed
9/1/2016 8:48 9/1/2016 8:50 9/1/2016 8:52 9/1/2016 8:52 9/1/2016 8:54 9/1/2016 8:56 9/1/2016 8:58 9/1/2016 9:00 9/1/2016 9:02 9/1/2016 9:04 9/1/2016 9:06 9/1/2016 9:08 9/1/2016 9:10 9/1/2016 9:12 9/1/2016 9:14 9/1/2016 9:14 9/1/2016 9:16 9/1/2016 9:18 9/1/2016 9:20 9/1/2016 9:22 9/1/2016 9:24

| essure (psi) | Observed Pressure (psi) | filter |  |
| :---: | :---: | :---: | :---: |
| 62.62 | 60.3 | 1 | 2.32 |
| 62.69 | 59.8 | 1 | 2.89 |
| 62.62 | 60.1 | 1 | 2.52 |
| 62.71 | 60.2 | 1 | 2.51 |
| 62.72 | 60.9 | 1 | 1.82 |
| 53.33 | 55.4 | 1 | -2.07 |
| 53.33 | 55 | 1 | -1.67 |
| 53.31 | 53.4 | 1 | -0.09 |
| 46.84 | 48.9 | 1 | -2.06 |
| 46.84 | 48.6 | 1 | -1.76 |
| 46.77 | 47.7 | 1 | -0.93 |
| 46.4 | 47.5 | 1 | -1.1 |
| 58.28 | 58.4 | 1 | -0.12 |
| 58.38 | 55.7 | 1 | 2.68 |
| 58.57 | 55.7 | 1 | 2.87 |
| 58.47 | 56.6 | 1 | 1.87 |
| 62.05 | 59.5 | 1 | 2.55 |
| 62.1 | 59.6 | 1 | 2.5 |
| 62.07 | 59.5 | 1 | 2.57 |


| Model | Observed | Diff (\%; abs) |
| :---: | :---: | :---: |
| 54.3 | 52.2 | $4 \%$ |
| 61.8 | 58.0 | 3.8 |
| 42.4 | 44.0 | -1.6 |

Run 054 - 042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1\#4

## 08/23/2016 00:00:00



Mod
Observed
9/1/2016 8:48
9/1/2016 8:50
9/1/2016 8:52
9/1/2016 8:52
9/1/2016 8:54
9/1/2016 8:56
9/1/2016 8:58
9/1/2016 9:00
9/1/2016 9:02
9/1/2016 9:04
9/1/2016 9:06
9/1/2016 9:08
9/1/2016 9:10
9/1/2016 9:12
9/1/2016 9:12
9/1/2016 9:14
9/1/2016 9:16
9/1/2016 9:18
9/1/2016 9:20
9/1/2016 9:22
9/1/2016 9:24

|  |  |
| :---: | :---: |
| 61.74 | Observed Pressure (psi) |
| 61.81 | -1.3 |
| 61.74 | -1.6 |
| 61.83 | 57.6 |
| 61.84 | 58 |
| 52.31 | 57.9 |
| 52.31 | 52.9 |
| 52.28 | 52.5 |
| 42.82 | 50.7 |
| 42.81 | 44.7 |
| 42.75 | 44 |
| 42.37 | 45.4 |
| 56.35 | 45.2 |
| 56.45 | 52 |
| 56.64 | 53.1 |
| 56.54 | 51.2 |
| 61.17 | 50.4 |
| 61.22 | 57.7 |
| 61.19 | 57.7 |
|  | 57.2 |

Test Site: 013

|  |  |  | Model | Observed | Diff (\%; abs) |
| :---: | :---: | :---: | :---: | :---: | :---: |
| TEST TYPE | P1 | P AVG (psi) | 62.7 | 61.5 | $2 \%$ |
| NODE ID | 27000 | P MAX (psi) | 70.4 | 66.1 | 4.3 |
|  |  | P MIN (psi) | 52.0 | 55.7 | -3.7 |


|  |  |  | Model | Observed | Diff (\%; abs) |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| TEST TYPE | P2 | P AVG (psi) | 69.5 | 71.5 | $-3 \%$ |
| NODE ID | 27003 | P MAX (psi) | 76.4 | 76.2 | 0.2 |
|  |  | P MIN (psi) | 60.7 | 64.9 | -4.2 |

Run 054 - 042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1\#4
Qun 054-042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1\#4


| Date/Time | Model | Observed | filter |  |
| :---: | :---: | :---: | :---: | :---: |
|  | Pressure (psi) | Observed Pressure (psi) |  |  |
| 9/1/2016 10:02 | 69.05 | 64.3 | 1 | 4.75 |
| 9/1/2016 10:04 | 69.05 | 64.6 | 1 | 4.45 |
| 9/1/2016 10:06 | 69.04 | 66.1 | 1 | 2.94 |
| 9/1/2016 10:08 | 69.04 | 65.2 | 1 | 3.84 |
| 9/1/2016 10:10 | 58.91 | 58.3 | 1 | 0.61 |
| 9/1/2016 10:12 | 58.89 | 60.4 | 1 | -1.51 |
| 9/1/2016 10:14 | 58.87 | 60.7 | 1 | -1.83 |
| 9/1/2016 10:16 | 52.03 | 55.9 | 1 | -3.87 |
| 9/1/2016 10:18 | 52.01 | 56.5 | 1 | -4.49 |
| 9/1/2016 10:20 | 51.97 | 56.9 | 1 | -4.93 |
| 9/1/2016 10:22 | 52.7 | 55.7 | 1 | -3 |
| 9/1/2016 10:24 | 52.74 | 58 | 1 | -5.26 |
| 9/1/2016 10:26 | 65.02 | 63 | 1 | 2.02 |
| 9/1/2016 10:28 | 64.99 | 61.9 | 1 | 3.09 |
| 9/1/2016 10:30 | 65 | 61.3 | 1 | 3.7 |
| 9/1/2016 10:32 | 70.35 | 62.7 | 1 | 7.65 |
| 9/1/2016 10:34 | 70.35 | 65.2 | 1 | 5.15 |
| 9/1/2016 10:36 | 70.38 | 65.3 | 1 | 5.08 |
| 9/1/2016 10:38 | 70.37 | 65.8 | 1 | 4.57 |

Test Site: 014

|  |  |  | Model | Observed | Diff (\%; abs) |
| :---: | :---: | :---: | :---: | :---: | :---: |
| TEST TYPE | P1 | P AVG (psi) | 47.9 | 49.2 | $-3 \%$ |
| NODE ID | 27006 | P MAX (psi) | 57.3 | 63.2 | -6.0 |
|  |  | P MIN (psi) | 34.0 | 40.3 | -6.3 |

Run 054-042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1*4


|  | Model | Observed |  |  |
| :---: | :---: | :---: | :---: | :---: |
| Date/Time | Pressure (psi) | Observed Pressure (psi) | filter |  |
| 8/30/2016 14:06 | 56.54 | -0.3 |  |  |
| 8/30/2016 14:08 | 56.54 | -0.3 |  |  |
| 8/30/2016 14:10 | 56.52 | -0.3 |  |  |
| 8/30/2016 14:12 | 56.53 | 56.6 | 1 | -0.07 |
| 8/30/2016 14:14 | 56.53 | 56 | 1 | 0.53 |
| 8/30/2016 14:16 | 56.54 | 55.6 | 1 | 0.94 |
| 8/30/2016 14:18 | 56.55 | 47.9 | 1 | 8.65 |
| 8/30/2016 14:20 | 56.61 | 54.4 | 1 | 2.21 |
| 8/30/2016 14:22 | 34.04 | 40.5 | 1 | -6.46 |
| 8/30/2016 14:24 | 34.6 | 41.3 | 1 | -6.7 |
| 8/30/2016 14:26 | 34.57 | 41.1 | 1 | -6.53 |
| 8/30/2016 14:28 | 34.58 | 40.5 | 1 | -5.92 |
| 8/30/2016 14:30 | 34.57 | 40.5 | 1 | -5.93 |
| 8/30/2016 14:32 | 34.57 | 40.3 | 1 | -5.73 |
| 8/30/2016 14:34 | 57.18 | 63.2 | 1 | -6.02 |
| 8/30/2016 14:36 | 57.21 | 52.9 | 1 | 4.31 |
| 8/30/2016 14:38 | 57.23 | 53.2 | 1 | 4.03 |
| 8/30/2016 14:40 | 57.25 | 53.6 | 1 | 3.65 |



Run 054-042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1\#4



$\left.\begin{array}{ccccc}\begin{array}{c}\text { Date/Time }\end{array} & \begin{array}{l}\text { Model } \\ \text { Pressure (psi) }\end{array} & \begin{array}{c}\text { Observed } \\ \text { Observed }\end{array} \\ \text { Pressure (psi) }\end{array}\right)$

Test Site: 015


Re Run 054-042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1*4


|  | Model | Observed |  |  |
| :---: | :---: | :---: | :---: | :---: |
| Date/Time | Pressure (psi) | Observed Pressure (psi) | filter |  |
| 8/30/2016 10:30 | 58.84 | 57 | 1 | 1.84 |
| 8/30/2016 10:32 | 58.87 | 57.1 | 1 | 1.77 |
| 8/30/2016 10:34 | 58.88 | 56.9 | 1 | 1.98 |
| 8/30/2016 10:36 | 58.91 | 57.3 | 1 | 1.61 |
| 8/30/2016 10:38 | 58.93 | 56.1 | 1 | 2.83 |
| 8/30/2016 10:40 | 58.95 | 55.9 | 1 | 3.05 |
| 8/30/2016 10:42 | 58.97 | 56.5 | 1 | 2.47 |
| 8/30/2016 10:44 | 58.99 | 57 | 1 | 1.99 |
| 8/30/2016 10:46 | 59.02 | 56.4 | 1 | 2.62 |
| 8/30/2016 10:48 | 59.68 | 57.1 | 1 | 2.58 |
| 8/30/2016 10:50 | 58.15 | 56.6 | 1 | 1.55 |
| 8/30/2016 10:52 | 58.16 | 56.7 | 1 | 1.46 |
| 8/30/2016 10:54 | 58.16 | 55.9 | 1 | 2.26 |
| 8/30/2016 10:56 | 58.17 | 55.6 | 1 | 2.57 |
| 8/30/2016 10:58 | 58.2 | 56.7 | 1 | 1.5 |
| 8/30/2016 11:00 | 58.22 | 56.1 | 1 | 2.12 |
| 8/30/2016 11:02 | 58.24 | 55.6 | 1 | 2.64 |
| 8/30/2016 11:04 | 58.26 | 56.8 | 1 | 1.46 |
| 8/30/2016 11:06 | 28.77 | 37.2 | 1 | -8.43 |
| 8/30/2016 11:08 | 28.76 | 41.3 | 1 | -12.54 |
| 8/30/2016 11:10 | 28.77 | 40.4 | 1 | -11.63 |
| 8/30/2016 11:12 | 28.77 | 40.5 | 1 | -11.73 |
| 8/30/2016 11:14 | 28.76 | 40 | 1 | -11.24 |
| 8/30/2016 11:16 | 28.77 | 40.2 | 1 | -11.43 |
| 8/30/2016 11:18 | 28.76 | 40.5 | 1 | -11.74 |
| 8/30/2016 11:20 | 28.76 | 42.7 | 1 | -13.9 |
| 8/30/2016 11:22 | 58.29 | 55.6 | 1 | 2.69 |
| 8/30/2016 11:24 | 58.28 | 57 | 1 | 1.28 |
| 8/30/2016 11:26 | 58.33 | 56.1 | 1 | 2.23 |
| 8/30/2016 11:28 | 58.35 | 57.8 | 1 | 0.55 |

Observed Pr
8

$\square$

图Run 054 - 042127; wells FLOW; MDS and PZ AUTO; 4/3/17 GIS Edits> Control_1*4 13.1 26.7


Date/Time Mo $\begin{array}{clll}\text { Date/Time } & \begin{array}{l}\text { Model } \\ \text { Pressure (psi) }\end{array} & \begin{array}{l}\text { Observed } \\ \text { Observed Pressure (psi) }\end{array} & \text { filt }\end{array}$ 8/30/2016 10:30
8/30/2016 10:32 8/30/2016 10:34 8/30/2016 10:34 8/30/2016 10:36
8/30/2016 10:38 8/30/2016 10:38
$8 / 30 / 2016$
$10: 40$ 8/30/2016 10:40
8/30/2016 10:42 8/30/2016 10:44 8/30/2016 10:46 8/30/2016 10:46 8/30/2016 10:50 8/30/2016 10:52 8/30/2016 10:54 8/30/2016 10:56
8/30/2016 $10: 58$ 8/30/2016 10:58
$8 / 30 / 201611: 00$ 8/30/2016 11:02 8/30/2016 11:04 8/30/2016 11:06 8/30/2016 11:06
8/30/2016 11:08 8/30/2016 11:08
8/30/2016 11:10 8/30/2016 11:10
$8 / 30 / 2016 ~ 11: 12$ 8/30/2016 11:12 8/30/2016 11:14 $8 / 30 / 2016$ 11:16
$8 / 30 / 201611 \cdot 18$ 8/30/2016 11:18
$8 / 30 / 201611: 20$ 8/30/2016 11:22 $8 / 30 / 2016$ 11:24
$8 / 30 / 201612126$ $8 / 30 / 2016$ 11:26
$8 / 30 / 201611 \cdot 28$ 8/30/2016 11:28
45.53
4.56
45.57
45.6
45.62
45.65
45.66
45.68
4.71
46.37
44.84
4.45
44.85
44.87
44.89
4.91
44.93
44.95
13.14
13.14
13.15
13.14
13.14
13.15
13.14
13.14
44.99
44.97
4.52
45.04

5 . 6

8
1
7
$\qquad$
-1.9
43.7
43.8
43
43.9
43.6
43.2
43.2
44
43.1
43.9
43.9
43.4
44.7
43.5
43.7
43.1
43
43.8
36.6
27.2
27
26.7
27
27
26.7
26.7
27.7
43.3
43.2




Test Site: 016


Test Site: 017


Test Site: 018


17 GIS Edits> Control_1*4


| Date/Time | Model | Observed | filter |  |
| :---: | :---: | :---: | :---: | :---: |
|  | Pressure (psi) | Observed Pressure (psi) |  |  |
| 8/31/2016 11:06 | 86.49 | 82.7 | 1 | 3.79 |
| 8/31/2016 11:08 | 86.57 | 83.9 | 1 | 2.67 |
| 8/31/2016 11:10 | 86.53 | 84.1 | 1 | 2.43 |
| 8/31/2016 11:12 | 86.52 | 84.5 | 1 | 2.02 |
| 8/31/2016 11:14 | 86.51 | 84 | 1 | 2.51 |
| 8/31/2016 11:16 | 86.52 | 84 | 1 | 2.52 |
| 8/31/2016 11:18 | 86.57 | 83.2 | 1 | 3.37 |
| 8/31/2016 11:20 | 86.6 | 83.7 | 1 | 2.9 |
| 8/31/2016 11:22 | 86.61 | 83.7 | 1 | 2.91 |
| 8/31/2016 11:24 | 86.64 | 79.4 | 1 | 7.24 |
| 8/31/2016 11:26 | 86.86 | 81.5 | 1 | 5.36 |
| 8/31/2016 11:28 | 86.78 | 82.3 | 1 | 4.48 |
| 8/31/2016 11:30 | 86.75 | 82 | 1 | 4.75 |
| 8/31/2016 11:32 | 86.76 | 81.9 | 1 | 4.86 |
| 8/31/2016 11:34 | 86.79 | 82.2 | 1 | 4.59 |
| 8/31/2016 11:36 | 86.72 | 82.3 | 1 | 4.42 |
| 8/31/2016 11:38 | 69.76 | 64.3 | 1 | 5.46 |
| 8/31/2016 11:40 | 70.48 | 72.7 | 1 | -2.22 |
| 8/31/2016 11:42 | 70.04 | 70.8 | 1 | -0.76 |
| 8/31/2016 11:44 | 70.08 | 70.5 | 1 | -0.42 |
| 8/31/2016 11:46 | 70.06 | 71.2 | 1 | -1.14 |
| 8/31/2016 11:48 | 70.03 | 70.7 | 1 | -0.67 |
| 8/31/2016 11:50 | 70.04 | 72 | 1 | -1.96 |
| 8/31/2016 11:52 | 87.16 | 88 | 1 | -0.84 |
| 8/31/2016 11:54 | 86.29 | 81.1 | 1 | 5.19 |
| 8/31/2016 11:56 | 86.76 | 81.3 | 1 | 5.46 |
| 8/31/2016 11:58 | 86.73 | 6.8 |  |  |



NODE ID
Run 054 - 042127; wells FLOW; MDS and PZ AUTO; $4 / 3 / 17$ GIS Edits> Control_1 $1 * 4$
Model Observed

Date/Time

| 8/31/2016 11:06 | 117.32 |
| :---: | :---: |
| 8/31/2016 11:08 | 117.42 |
| 8/31/2016 11:10 | 117.37 |
| 8/31/2016 11:12 | 117.35 |
| 8/31/2016 11:14 | 117.33 |
| 8/31/2016 11:16 | 117.35 |
| 8/31/2016 11:18 | 117.4 |
| 8/31/2016 11:20 | 117.43 |
| 8/31/2016 11:22 | 117.44 |
| 8/31/2016 11:24 | 117.49 |
| 8/31/2016 11:26 | 117.74 |
| 8/31/2016 11:28 | 117.65 |
| 8/31/2016 11:30 | 117.6 |
| 8/31/2016 11:32 | 117.62 |
| 8/31/2016 11:34 | 117.65 |
| 8/31/2016 11:36 | 117.56 |
| 8/31/2016 11:38 | 96.5 |
| 8/31/2016 11:40 | 97.86 |
| 8/31/2016 11:42 | 97.07 |
| 8/31/2016 11:44 | 97.16 |
| 8/31/2016 11:46 | 97.14 |
| 8/31/2016 11:48 | 97.11 |
| 8/31/2016 11:50 | 97.12 |
| 8/31/2016 11:52 | 118.13 |
| 8/31/2016 11:54 | 117.03 |
| 8/31/2016 11:56 | 117.63 |
| 8/31/2016 11:58 | 117.59 |


| -1 |  |  |
| :---: | :---: | :---: |
| 114.3 | 1 | 3.14 |
| 109.6 | 1 | 7.89 |
| 112.4 | 1 | 5.34 |
| 113.2 | 1 | 4.45 |
| 113.2 | 1 | 4.4 |
| 112.9 | 1 | 4.72 |
| 113.8 | 1 | 3.85 |
| 113.2 | 1 | 4.36 |
| 96.9 | 1 | -0.32 |
| 102.1 | 1 | -4.24 |
| 10.1 | 1 | -3.03 |
| 99.8 | 1 | -2.64 |
| 100.1 | 1 | -2.96 |
| 99.5 | 1 | -2.39 |
| 10.3 | 1 | -4.18 |
| 120.6 | 1 | -2.47 |
| 112.1 | 1 | 4.93 |
| 113.1 | 1 | 4.53 |
| 113.2 | 1 | 4.39 |

## Appendix H - EST Water Age Modeling Results








Appendix I - Capital Improvements Plan Detailed Opinions of Probable Construction Cost

| $\begin{array}{\|c\|} \hline \text { Project } \\ \text { Code } \end{array}$ |  | Description | Linear Feet of Pipe |  |  |  |  |  |  |  |  | City Cost | DeveloperCost | Driver | Project Priority |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  | ${ }^{\text {" }}$ | $8^{\prime \prime}$ | 12" | 16" | $24{ }^{\prime \prime}$ | 30" | 36" | 42" | Total |  |  |  |  |
|  |  |  | Water Line Segment D (Phase 4) | 0 | 0 | 0 | 0 | 8,500 | 0 | 0 | 0 | 8,500 | 3,874,000 |  | High Water Age | Highest |
|  |  | Upsize $8^{\prime \prime}$ Line to 12 " along Meadowood Blvd | 0 | 1,000 | 1,030 | 0 | 0 | 0 | 0 | 0 | 2,030 | 526,000 |  | Low Fireflow | High |
| H | 1 | Complete 12" Line Along 36th Ave. NE | 0 | 0 | 4,080 | 0 | 0 | 0 | 0 | 0 | 4,080 | 1,147,000 |  | High Headloss | High |
| H | 3 | Upsize 6" Line to 12 " at Alameda St. and Vicksburg Ave. | 0 | 0 | 105 | 0 | 0 | 0 | 0 | 0 | 105 | 51,000 |  | High Headloss | High |
| H | 4 | Upsize Lines to Boyd Tower | 0 | 0 | 300 | 800 | 0 | 0 | 0 | 0 | 1,100 | 390,000 |  | High Headloss | High |
| M | 5 | WL Replacement: Flood: Rock Creek to Venture | 0 | 0 | 3,400 | 6,400 | 0 | 0 | 0 | 0 | 9,800 | 3,355,000 |  | Maintenance | High |
| M | 7 | Robinson Waterline: 24th Ave. NE to 24th Ave. NW | 0 | 0 | 0 | 0 | 0 | 21,850 | 0 | 0 | 21,850 | 11,576,000 |  | Maintenance | High |
| M | 8 | Waterine Replacement: Interstate Drive | 0 | 5,680 | 0 | 0 | 0 | , | 0 | 0 | 5,680 | 1,140,000 |  | Maintenance | High |
| M | 11 | Water Line Replacement: Gray St. \& Tonhawa St. | 430 | 4,000 | 1,800 | 0 | 0 | 0 | 0 | 0 | 6,230 | 1,002,000 |  | Maintenance | High |
| M | 12 | Water Line Replacement: West of Campus | 8,150 | 1,550 | 0 | 0 | 0 | 0 | 0 | 0 | 9,700 | 1,658,000 |  | Maintenance | High |
| M | 13 | Alameda Waterline Replacement: S. Poncha Ave. to 24th Ave. NE | 0 | 0 | 0 | 0 | 8,500 | 200 | 0 | 0 | 8,700 | 3,741,000 |  | Maintenance | High |
| M | 15 | Robinson Waterline Replacement: WTP to 24th Ave NE | 0 | 0 | 0 | 0 | 80 | 0 | 0 | 2,600 | 2,680 | 3,338,000 |  | Maintenance | High |
| M | 17 | Replace Upper Pressure Zone Pumps | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |  |  | Maintenance | High |
| P | 1 | Extend Upper PZ to Hollister Trail and Palomino Way | 0 | 425 | 0 | 0 | 0 | 0 | 0 | 0 | 425 | 142,000 |  | Low Pressure | High |
| P | 4 | Include Meadowood Blvd in HPP | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |  | Low Pressure | High |
| P | 5 | Future Elevated Storage Tank in MDS | 0 | 0 | 0 | 0 | 800 | 0 | 0 | 0 | 800 | 3,638,000 |  | Low Pressure | High |
| F | 4 | Upsize 6"L Line to 8" along Harriett Road | 0 | 1,160 | 0 | 0 | 0 | 0 | 0 | 0 | 1,160 | 276,000 |  | Low Fireflow | Medium |
| F |  | Complete 6 " loop along Thedford Drive | 425 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 425 | 125,000 |  | Low Fireflow | Medium |
| F | 8 | Upsize 6" Line to 8" along Willow Creek Drive | 0 | 705 | 0 | 0 | 0 | 0 | 0 | 0 | 705 | 200,000 |  | Low Fireflow | Medium |
| F | 9 | Extend the HPP to Redwood Drive | 0 | 600 | 0 | 0 | 0 | 0 | 0 | 0 | 600 | 162,000 |  | Low Fireflow | Medium |
| F | 16 | Upsize 6" Line to 8" Along Eisenhower Rd | 500 | 2,010 | 0 | 0 | 0 | 0 | 0 | 0 | 2,510 | 557,000 |  | Low Fireflow | Medium |
| F | 17 | Connect 6" dead end to 12 " across N . Porter Ave | 85 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 85 | 39,000 |  | Low Fireflow | Medium |
| F | 25 | Upsize 6 " Line to 8 " along Pinebrooke Court | 0 | 590 | 0 | 0 | 0 | 0 | 0 | 0 | 590 | 151,000 |  | Low Fireflow | Medium |
| F | 26 | Connect 6" Lines at Westport Dr. and Fairway Dr. | 700 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 700 | 147,000 |  | Low Fireflow | Medium |
| F | 27 | Upsize 4" Line to 6 " along Foreman Avenue | 1,150 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1,150 | 254,000 |  | Low Fireflow | Medium |
| F | 28 | 8"Line along E Main St. Near Beacon Ave. | 0 | 1,180 | 0 | 0 | 0 | 0 | 0 | 0 | 1,180 | 288,000 |  | Low Fireflow | Medium |
| F | 30 | Upsize $6^{\prime \prime}$ Line to 8 " along Jean Marie Dr. | 0 | 1,875 | 0 | 0 | 0 | 0 | 0 | 0 | 1,875 | 437,000 |  | Low Fireflow | Medium |
| F | 32 | Extend 6" line along Elm Avenue to W. Symmes St. | 220 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 220 | 70,000 |  | Low Fireflow | Medium |
| F | 34 | Connect Dead-End 6"Line in The Pines Apartments | 450 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 450 | 110,000 |  | Low Fireflow | Medium |
| F | 35 | Upsize 4" Lines to 6" along Justin Dr., Bill Carrol Dr., and Cara Jo Dr. | 650 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 650 | 157,000 |  | Low Fireflow | Medium |
| F | 41 | Connect 6" Dead-End Line to McGee Drive | 600 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 600 | 137,000 |  | Low Fireflow | Medium |
| F | 42 | Complete 6" Loop along Brookside Drive | 200 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 200 | 85,000 |  | Low Fireflow | Medium |
|  |  | Upsize 6 " Line to 8 " along Rolling Hills Street | 0 | 820 | 0 | 0 | 0 | 0 | 0 | 0 | 820 | 221,000 |  | Low Fireflow | Medium |
| F | 44 | Upsize 6 " Line to 8 " along Whispering Pines Drive | 0 | 460 | 0 | 0 | 0 | 0 | 0 | 0 | 460 | 126,000 |  | Low Fireflow | Medium |
| H | 5 | Upsize 6 " Line to 8" along Chautauqua Ave. | 0 | 400 | 0 | 0 | 0 | 0 | 0 | 0 | 400 | 131,000 |  | High Headloss | Medium |
| M | 1 | WL Replacement: Classen/Flood: Hwy 9 to Indian Hills | 0 | 0 | 12,000 | 24,100 | 0 | 0 | 0 | 0 | 36,100 | 11,975,000 |  | Maintenance | Medium |
| M | 2 | Water Dist. System Improvements - Segment G | 0 | 0 | 7,280 | 0 | 0 | 0 | 0 | 0 | 7,280 | 1,682,000 |  | Maintenance | Medium |
| M | 3 | WL Replacement: Frankin: RR to 12th NW | 0 | 0 | 2,170 | 0 | 0 | 0 | 0 | 0 | 2,170 | 584,000 |  | Maintenance | Medium |
| M | 6 | Water Line Replacement: Hall Park, Phase 2 | 4,600 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4,600 | 742,000 |  | Maintenance | Medium |
| M | 9 | WL Replacement: W. Main: Berry to Interstate Drive | 0 | 5,170 | 6,830 | 0 | 0 | 0 | 0 | 0 | 12,000 | 3,025,000 |  | Maintenance | Medium |
| M | 10 | Waterline Replacement: Flood Avenue | 0 | 6,130 | 0 | 0 | 0 | 0 | 0 | 0 | 6,130 | 1,505,000 |  | Maintenance | Medium |
| M | 14 | 24th Ave NE Waterline Replacement: Alameda St. to Robinson St. | 0 | 0 | 0 | 0 | 0 | 0 | 5,200 | 0 | 5,200 | 3,920,000 |  | Maintenance | Medium |
| M | 16 | Robinson PZ Waterline Replacement: WTP to 24th Ave NE | 0 | 0 | 0 | 0 | 2,590 | 0 | 0 | 0 | 2,590 | 1,177,000 |  | Maintenance | Medium |
| P | 3 | Expand Upper PZ to Include Crest Place | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |  | Low Pressure | Medium |
| w | 2 | New 12" pipe on Nantucket Blvd | 0 | 0 | 240 | 0 | 0 | 0 | 0 | 0 | 240 | 81,000 |  | High Water Age | Medium |
| F | 1 | Loop 6" Line on Della St NW and NW Sterling Ct | 2,49 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2,495 | 547,000 |  | Low Fireflow | Low |
| F | 10 | Upsize 6" Line to 8 " along Briarcliff Rd | 0 | 1,170 | 0 | 0 | 0 | 0 | 0 | 0 | 1,170 | 53,000 |  | Low Fireflow | Low |
| F | 12 | Upsize 6" Line to 8" along Hillside Drive | 0 | 910 | 0 | 0 | 0 | 0 | 0 | 0 | 910 | 240,000 |  | Low Fireflow | Low |
| F |  | Upsize 6 " Line to 8 " along Valley Ridge Road | 0 | 1,250 | 0 | 0 | 0 | 0 | 0 | 0 | 1,250 | 301,000 |  | Low Fireflow | Low |
| F | 20 | Upsize $6^{\prime \prime}$ Line to $8^{\prime \prime}$ along Wheaton Dr | 0 | 300 | 0 | 0 | 0 | 0 | 0 | 0 | 300 | 99,000 |  | Low Fireflow | Low |
|  | 22 | Upsize $6^{\prime \prime}$ Line to 8 " along Hunter's Hill Road | 0 | 1,440 | 0 | 0 | 0 | 0 | 0 | 0 | 1,440 | 357,000 |  | Low Fireflow | Low |
| F | 24 | Upsize 6 " Line to 8 " along Cedar Ridge Drive | 0 | 470 | 0 | 0 | 0 | 0 | 0 | 0 | 470 | 127,000 |  | Low Fireflow | Low |
| F | 31 | Upsize $6^{\prime \prime}$ Line to $8^{\prime \prime}$ along McFarland St. | 0 | 530 | 0 | 0 | 0 | 0 | 0 | 0 | 530 | 139,000 |  | Low Fireflow | Low |
| F | 36 | Upsize 6" Lines to 8" along Brandon Cr., Sheffield Dr., Chamblee Dr., Surrey Dr., \& Village Dr. | 0 | 1,725 | 0 | 0 | 0 | 0 | 0 | 0 | 1,725 | 416,000 |  | Low Fireflow | Low |
| F | 37 | Upsize 6" Line to 8" along Columbia Cr., Atlanta Cr., Montgomery Cr., Raleigh Cr., and Mobile Cr. | 0 | 1,705 | 0 | 0 | 0 | 0 | 0 | 0 | 1,705 | 511,000 |  | Low Fireflow | Low |
| F | 38 | Upsize 6 " Line to 8 " along Peppertree Ct. | 0 | 680 | 0 | 0 | 0 | 0 | 0 | 0 | 680 | 195,000 |  | Low Fireflow | Low |
| F | 40 | Upsize $6^{\prime \prime}$ Line to $8^{\prime \prime}$ " South of Briggs St. | 0 | 410 | 0 | 0 | 0 | 0 | 0 | 0 | 410 | 132,000 |  | Low Fireflow | Low |
| F | 45 | Upsize $6^{\prime \prime}$ Line to 8 " along Holly Cir. | 0 | 50 | 0 | 0 | 0 | 0 | 0 | 0 | 50 | 43,000 |  | Low Fireflow | Low |
| F | 46 | Extend 6" Line Along Twin Creek Village Apartments | 360 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 360 | 95,000 |  | Low Fireflow | Low |
| H | 2 | Upsize 12" Line to 16" along Robinson from WTP to 36th Ave. NE | 0 | 0 | 0 | 2,730 | 0 | 0 | 0 | 0 | 2,730 | 1,073,000 |  | High Headloss | Low |
| M | 4 | Waterline Improvement: OKC Second Feed | 0 | 0 | 0 | 0 | 31,680 | 0 | 0 | 0 | 31,680 | 16,077,000 |  | Maintenance | Low |
|  |  | Add 5th 250 HP Pump to MDS PS | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 260,000 |  | Low Pressure | Low |
| w | 1 | Complete $6^{\prime \prime}$ loop along Teton Oval culdesac | 120 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 120 | 53,000 |  | High Water Age | Low |
| w | 3 | Upsize $6^{\prime \prime}$ Line to $8^{\prime \prime}$ along Shrill St. | 0 | 2,890 | 25 | 0 | 0 | 0 | 0 | 0 | 2,915 | 683,000 |  | High Water Age | Low |
| w | 4 | Connect 6" Lines at NW corner of 24th Avenue NW and W. Main Street | 540 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 540 | 144,000 |  | High Water Age | Low |
| F | 2 | Upsize 6" Line to 8" along Moor Drive and Nicole Place | 0 | 790 | 0 | 0 | 0 | 0 | 0 | 0 | 790 | 215,000 |  | Low Fireflow | Very Low |
| F | 3 | Upsize $6^{\prime \prime}$ Line to 8" along Nicole Circle | 0 | 675 | 0 | 0 | 0 | 0 | 0 | 0 | 675 | 184,000 |  | Low Fireflow | Very Low |
| F | 5 | Upsize 6" Line to 8" along Bright St., Glisten Ct., Ripple Ave., \& Glisten St. | 0 | 1,615 | 0 | 0 | 0 | 0 | 0 | 0 | 1,615 | 395,000 |  | Low Fireflow | Very Low |
| F | 7 | Upsize 6" Line to 8" along Sloane St., Shipley Dr., Bishop's Ct., \& Victoria Dr. | 0 | 1,600 | 0 | 0 | 0 | 0 | 0 | 0 | 1,600 | 392,000 |  | Low Fireflow | Very Low |
| F | 11 | Upsize $6^{\prime \prime}$ Line to 8 " off of Brookhaven Blvd | 0 | 345 | 0 | 0 | 0 | 0 | 0 | 0 | 345 | 101,000 |  | Low Fireflow | Very Low |
| F | 13 | Upsize 6" Line to 8" on Northhampton Court | 334 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 334 | 108,000 |  | Low Fireflow | Very Low |
| F | 15 | Upsize 6" Line to 8" along Warwick Dr. and Waverly Dr. | 0 | 1,970 | 0 | 0 | 0 | 0 | 0 | 0 | 1,970 | 473,000 |  | Low Fireflow | Very Low |
| F | 18 | Upsize 6" Line to 8" along Wind Hill Rd | 0 | 400 | 0 | 0 | 0 | 0 | 0 | 0 | 400 | 119,000 |  | Low Fireflow | Very Low |
| - | 19 | Upsize 6" Line to 8 " along Ridgemont Circle | 0 | 460 | 0 | 0 | 0 | 0 | 0 | 0 | 460 | 131,000 |  | Low Fireflow | Very Low |
| F |  | Upsize 6"Line to $8^{\text {" }}$ along Sundance Ct. | 0 | 360 | 0 | 0 | 0 | 0 | 0 | 0 | 360 | 105,000 |  | Low Fireflow | Very Low |
| F | 23 | Upsize $6^{\prime \prime}$ Line to $8^{\text {" }}$ along Innsbrook Court | 0 | 350 | 0 | 0 | 0 | 0 | 0 | 0 | 350 | 102,000 |  | Low Fireflow | Very Low |
| F | 29 | Upsize 6" Line to 8" along Riverwalk Ct. | 0 | 825 | 0 | 0 | 0 | 0 | 0 | 0 | 825 | 206,000 |  | Low Fireflow | Very Low |
| F | 33 | Upsize 6" Line to 8" along Schulze Dr. and Creston Way | 0 | 1,425 | 0 | 0 | 0 | 0 | 0 | 0 | 1,425 | 337,000 |  | Low Fireflow | Very Low |
| F | 47 | Upsize 6" Lines to 8" along White Oak Cir., Oak Vista Cir., \& Bois-de-arc Cir. | 0 | 1,170 | 0 | 0 | 0 | 0 | 0 | 0 | 1,170 | 286,000 |  | Low Fireflow | Very Low |
| $F$ | 48 | Loop 6" Line along Black Locust Ct \& Black Locust Place | 985 | 1,055 | 0 | 0 | 0 | 0 | 0 | 0 | 2,040 | 459,000 |  | Low Fireflow | Very Low |
| D | 1 | 12" Loop along 48th Avenue NW | 0 | 1,175 | 6,240 | 0 | 0 | 0 | 0 | 0 | 7,415 | 0 | 1,877,000 | Future Development |  |
| D | 2 | Install 12" line along 48th Ave NW between W Rock Creek Rd and Las Colinas Ln | 0 | 0 | 2,475 | 0 | 0 | 0 | 0 | 0 | 2,475 | 0 | 663,000 | Future Development | - |
| D | 3 | Waterline Segment H | 0 | 0 | 1,500 | 0 | 0 | 0 | 0 | 0 | 1,500 | 0 | 368,000 | Future Development | - |
| D | 4 | Add 6" line near Wyckham PI. | 675 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 675 | 0 | 169,000 | Future Development | - |
| D | 5 | Add 6" Line Along Kingswood Dr | 340 | , | 0 | 0 | 0 | 0 | 0 | 0 | 340 | 0 | 89,000 | Future Development | - |
| D | 6 | Extend 8"Lines to Harbor Dr. and Lyric St. | 0 | 1,335 | 0 | 0 | 0 | 0 | 0 | 0 | 1,335 | 0 | 335,000 | Future Development | - |
| D |  | $16 "$ Destin Landing Development | 0 | 0 | 0 | 8,000 | 0 | 0 | 0 | 0 | 8,000 | 0 | 2,853,000 | Future Development | - |




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## DETAILED PROJECT DESCRIPTION:

Project will replace ductile iron pipe (DIP) water lines in an area west of OU bounded by Brooks to the north, Flood to the west, Lindsey to the south, and Elm to the east. DIP is rupturing causing extensive damage to driveways, streets, and yards. The DIP is not compatible with the corrosive clay soils. Project will replace approximately 9,700 feet of 6 " and 8 " DIP with C900 PVC pipe.




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This project would add a new 1.0 million gallon elevated storage tank in the Main Distribution System in the southeast corner of the City. The elevated storage tank would be connected to the mith a new 24 " pipeline approximately 800 ft long



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Appendix J - Recommended Changes to Upper Pressure Zone Boundary






[^0]:    ${ }^{1}$ Clarion, Norman 2025 Land Use and Transportation Plan, Adoption Draft, October 15, 2004, as amended through date of data collection.
    ${ }^{2}$ Carollo Engineers in association with Tetratech, 2060 Strategic Water Supply Plan, prepared for Norman Utilities Authority, August 2014.

[^1]:    ${ }^{3}$ Current Urban Service Area = The City provides or plans to provide potable water in this area. This area is also sewered by gravity flow, sewerable by gravity flow, served by existing lift stations, or designed to be served by existing lift stations.
    ${ }^{4}$ Future Urban Service Area = area outside existing water and/or sanitary sewer service areas. The City provides or plans to provide potable water in this area.
    ${ }^{5}$ Suburban Residential Area = area suitable for development from an environmental standpoint but not planned for sanitary sewer service. In general, potable water in this area will come from private wells.
    ${ }^{6}$ Country Residential Area = area predominantly located over the Garber-Wellington Aquifer primary recharge area and/or within the flood plains of the Little River and South Canadian River. In general, potable water in this area will come from private wells.

[^2]:    ${ }^{7}$ Carollo Engineers in association with Tetratech, 2060 Strategic Water Supply Plan, prepared for Norman Utilities Authority, August 2014.
    ${ }^{8}$ A real loss is water lost physically from the distribution system, like through pipe leaks or storage tank overflows. This water's financial loss is calculated using the production cost of water.
    ${ }^{9}$ An apparent loss is water not paid for because it is "lost" due to metering inaccuracies, unauthorized consumption or water theft, or billing data errors. This water's financial loss is calculated using the retail cost of water.

[^3]:    ${ }^{10}$ Clarion, Norman 2025 Land Use and Transportation Plan, Adoption Draft, October 15, 2004, as amended through date of data collection.

[^4]:    ${ }^{11}$ The "High SA" scenario is described in the notes for Figure 3-3 (page 18).

[^5]:    Key

    | 1 | $\begin{array}{l}\text { Pressure difference is final "Modeled Pressure" after model improvements minus } \\ \text { "Observed Pressure." }\end{array}$ |
    | :---: | :--- |
    |  | Pressure difference is greater than 5 psi between modeled pressure and SCADA data. |
    |  | Hydrant test is a good match to SCADA data and within an acceptable range. |

[^6]:    ${ }^{12}$ DEQ 252:626-19-1
    ${ }_{14}^{13}$ DEQ 252:626-3-6(c)(6)
    ${ }^{14}$ Distribution System Requirements for Fire Protection, AWWA, $4^{\text {th }}$ Edition, pp 13.

[^7]:    ${ }^{1}$ City of Norman, Oklahoma: Water Treatment Plant Expansion Study, Carollo Engineers, May 2007.

[^8]:    ${ }^{1}$ Other uses may include water used for fire-fighting, street cleaning, water main and sewer flushing, fire flow tests, and other unmetered uses.

[^9]:    ${ }^{1}$ P. Boulos, K. Lansey, and B. Karney (2006). Comprehensive Water Distribution Systems Analysis Handbook for Engineers and Planners, MWH Soft. Pasadena, CA. Section 7.3.3.

[^10]:    ${ }^{2}$ A sufficient pressure drop is indicated by at least a 10 percent drop in pressure (static to residual condition) or up to a 10 psi drop in pressure. The maximum pressure drop will occur when the two flow hydrants are open simultaneously. (Source: AWWA Manual: Installation, Field Testing, and Maintenance of Fire Hydrants [2006])

