City of Prescott

2017 Water and Wastewater Models

# TECHNICAL MEMORANDUM 2

# UTILITIES INFRASTRUCTURE IMPROVEMENT PLAN AND LAND USE ASSUMPTIONS

AMENDED FINAL | April 2019

carollo®

City of Prescott
2017 Water and Wastewater Models

# TECHNICAL MEMORANDUM 2

## Abbreviations

| | |
|---|---|
| % | percent |
| 2013 Study | 2013 Water and Wastewater Models Study |
| A.A.C. | Arizona Administrative Code |
| A.A.C.E. | Association for the Advancement of Cost Engineers |
| AADF | average annual daily wastewater flows |
| ADOA | Arizona Department of Administration |
| CIP | Capital Improvement Program |
| CVID | Chino Valley Irrigation District |
| d/D | depth over diameter |
| fps | feet per second |
| ft | feet |
| ft/ft | feet per foot |
| gpm | gallons per minute |
| IBC | International Building Code |
| IFC | International Fire Code |
| IIP | Infrastructure Improvement Plan |
| in | inch |
| MDWWF | maximum daily wet weather flows |
| MG | million gallons |
| mgd | million gallons per day |
| PCC | Prescott City Code |
| PRV | pressure reducing valve |
| psi | pounds per square inch |
| PVC | polyvinyl chloride |
| TM1 | Technical Memorandum 1 |
| WRF | water reclamation facility |
| WRP | water reclamation plant |
| WWTP | wastewater treatment plant |
| YPIT | Yavapai-Prescott Indian Tribe |
| YRMC | Yavapai Regional Medical Center |

Technical Memorandum 2

# FIVE YEAR IIP

This technical memorandum presents the proposed water and wastewater projects and recommendations associated with the City's Infrastructure Improvement Plan (IIP). The IIP includes projects that are required to serve growth areas.

Infrastructure recommendations are based on the hydraulic modeling results of the water and wastewater systems and related analyses.

This document includes the following:

**Section 1 – Planning Framework** – identifies the population growth assumptions, water demands, and wastewater flows that form the basis of the capacity analyses.

**Section 2 – Water System Evaluation** – describes the existing water system, summarizes the performance criteria or standards of measurement used in the infrastructure evaluations, and presents the results of the supply, storage, booster pumping, and distribution system analyses.

**Section 3 – Wastewater System Evaluation** – describes the existing wastewater system, summarizes the performance criteria or standards of measurement used in the infrastructure evaluations, and presents the results of the pipe and lift station capacity analyses.

**Section 4 – Costs and Project Timing** – presents the costing methodology, water and wastewater project summaries, and timing of projects.

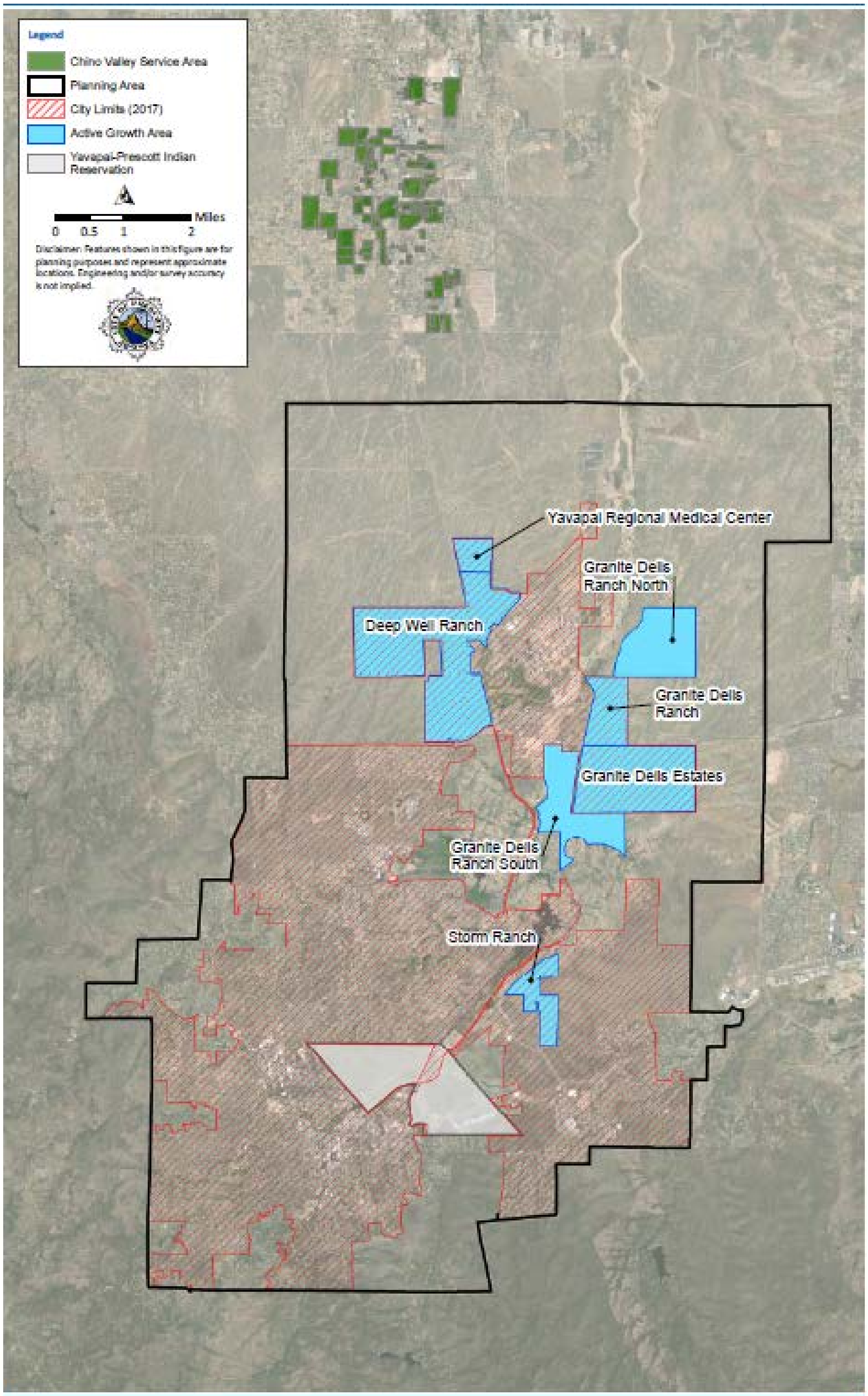
## Section 1 – Planning Framework

### 1.1 Study Area

The study area includes the City's Planning Area as illustrated in Figure 3.1, which includes the Chino Valley Irrigation District (CVID) and Yavapai-Prescott Indian Tribe (YPIT) reservation. There is growth projected in City's planning areas over the next fifteen years within the incorporated City limits and the unincorporated areas adjacent to the City. The City identified seven growth areas that are likely to develop in the next 5 to 15 years, including:

- Deep Well Ranch
- The Yavapai Regional Medical Center (YRMC), part of Deep Well Ranch
- Storm Ranch
- Granite Dells Estates
- Granite Dells Ranch – Commercial
- Granite Dells Ranch – North (outside 2017 incorporated limit)
- Granite Dells Ranch – South (outside 2017 incorporated limit)

Figure 3.1 Study Area



The timing of development in each of these growth areas was estimated using input from the City. All areas except the Granite Dells Ranch – North and South growth areas are expected to develop within the next 5 to 15 years. Table 3.1 summarizes the amount of growth expected in each planning year expressed as a percentage of the total development for the growth area.

Table 3.1 Anticipated Development Percentage in Growth Areas

| Growth Area | Planning Year | | | |
|---|---|---|---|---|
| | 2017 | 2022 | 2027 | 2032 |
| Deep Well Ranch | 0% | 15% | 30% | 50% |
| YRMC | 0% | 0% | 0% | 30% |
| Storm Ranch | 0% | 30% | 60% | 100% |
| Granite Dells Estates | 30% | 60% | 75% | 90% |
| Granite Dells Ranch – Commercial | 0% | 30% | 50% | 80% |
| Granite Dells Ranch – North | 0% | 0% | 0% | 10% |
| Granite Dells Ranch – South | 0% | 10% | 30% | 50% |

## 1.2 Population, Water Demand, and Wastewater Flow Projections

The City provided the following planning data to support the development of growth projections over the next 15 years:

- Historical population and housing trends
- Water production and billing records
- Wastewater flow records
- Land use and zoning classifications

This information was used to establish baseline (year 2017) water use and wastewater generation rates and to project water demands and wastewater flows for years 2022, 2027 and 2032. The methodology used to develop these projections is consistent with the approach taken in the City's 2013 Water and Wastewater Models Study (2013 Study).

### 1.2.1 Population Growth Summary

The growth projections from the 2013 Study for years 2017 through 2030 were compared to the Arizona Department of Administration (ADOA) Office of Employment and Population Statistics for years 2016 through 2050. The ADOA growth projections trended significantly lower than the 2013 Study with a projected year 2030 population of 42,300 compared to the 2013 Study projection of 52,500. To address this difference, Prescott's actual residential (single and multi-family) permit activity for years 2015 through 2017 was used to estimate that 300 new residential permits would be added each year through year 2037. Population estimates were then calculated assuming 1.6 people per home, which is the same figure used in the 2013 Study. Appendix A contains a graph that shows the population trends from the 2013 study, the ADOA 2016-2050 projections, and the projections for the 5- to 15-year IIP.

Table 3.2 summarizes the population and housing permit projections for the 5- to 15-year IIP.

Table 3.2 Prescott Population and Housing Permit Forecast

| Year | Population (1) | Change in Population (2) | Percent Population Change (%) | New Housing Units (No) |
|---|---|---|---|---|
| 2016 | 42,500 | 509 | 1.21 | 318 |
| 2017 | 42,999 | 499 | 1.17 | 312 |
| 2018 | 43,479 | 480 | 1.12 | 300 |
| 2019 | 43,959 | 480 | 1.10 | 300 |
| 2020 | 44,439 | 480 | 1.09 | 300 |
| 2021 | 44,919 | 480 | 1.08 | 300 |
| 2022 | 45,399 | 480 | 1.07 | 300 |
| 2023 | 45,879 | 480 | 1.06 | 300 |
| 2024 | 46,359 | 480 | 1.05 | 300 |
| 2025 | 46,839 | 480 | 1.04 | 300 |
| 2026 | 47,319 | 480 | 1.02 | 300 |
| 2027 | 47,799 | 480 | 1.01 | 300 |
| 2028 | 48,279 | 480 | 1.00 | 300 |
| 2029 | 48,759 | 480 | 0.99 | 300 |
| 2030 | 49,239 | 480 | 0.98 | 300 |
| 2031 | 49,719 | 480 | 0.97 | 300 |
| 2032 | 50,199 | 480 | 0.97 | 300 |

Notes:
(1) Year 2016 population provided by City.
(2) Year 2017 population growth calculated as: number of new housing units x 1.6 people per home (312 x 1.6 = 499). Years 2018 through 2032 population growth assumes 300 units added per year times 1.6 people per home.

### 1.2.2 Water Demand Summary

Average daily water demands were developed for each 5-year planning period over the next 15 years. Maximum day demands were estimated using a peaking factor of 1.8 (maximum day to average daily demand), which was developed using year 2012 through 2016 water production records. Peak hour demands were estimated using a peaking factor of 3.24 (peak hour to average daily demand), which is consistent with the value the City used in the 2013 Study.

A summary of the water demands for the 5- to 15-year IIP is presented in Table 3.3.

Table 3.3 Water Demand Projection Summary

| Planning Year | Water Demand (mgd) (1) | | |
|---|---|---|---|
| | Average Daily | Maximum Daily (2) | Peak Hour (3) |
| 2017 | 5.9 | 10.7 | 19.2 |
| 2022 | 6.7 | 12.0 | 21.6 |
| 2027 | 7.5 | 13.4 | 24.2 |
| 2032 | 8.4 | 15.0 | 27.1 |

Notes:
(1) Values include non-revenue water (8.5% of water production)
(2) Maximum daily to average daily demand peaking factor: 1.8
(3) Peak hour to average daily demand peaking factor: 3.24
Abbreviation:
mgd = million gallons per day

### 1.2.3 Wastewater Flow Summary

Average annual daily wastewater flows (AADF) were developed for each 5-year planning period over the next 15 years. Maximum daily wet weather flows (MDWWF) were estimated using the methodology established in the 2007 Wastewater Collection Model Study and carried forward to the 2013 Study. The City has two wastewater service areas. The Sundog service area sends flow to the Sundog Wastewater Treatment Plant (WWTP) and the Airport service area sends flow to the Airport Water Reclamation Facility (WRF). The City is consolidating these two service areas into one and will eventually send all wastewater flows to the Airport WRF.

A summary of the wastewater flows for the 5- to 15-year IIP by wastewater service area is presented in Table 3.4.

Table 3.4 Wastewater Flow Projection Summary

| Planning Year | Wastewater Flow (mgd) | | | | | |
|---|---|---|---|---|---|---|
| | Airport AADF | Airport Peak (1) | Sundog AADF | Sundog Peak (1) | Prescott AADF | Prescott Peak (1) |
| 2017 | 1.7 | 3.0 | 2.5 | 9.3 | 4.2 | 12.3 |
| 2022 | 1.9 | 3.2 | 2.8 | 9.6 | 4.7 | 12.8 |
| 2027 | 2.1 | 3.4 | 3.1 | 9.9 | 5.3 | 13.3 |
| 2032 | 2.4 | 3.7 | 3.4 | 10.2 | 5.8 | 13.9 |

Notes:

(1) Maximum day wet weather values included in Airport, Sundog and Prescott totals. For the Airport service area a wet weather flow of 1.3 mgd was added to the average annual daily flow, and for the Sundog service area a wet weather flow of 6.8 mgd was added to the average annual daily flow. These values were determined through inflow and infiltration analyses conducted as part of the 2007 Wastewater Collection Model Study.

# Section 2 – Water System Evaluation

## 2.1 Water System Description

The City of Prescott's water distribution system contains over 500 miles of distribution mains, approximately 37 miles of transmission mains that deliver water from five production wells in Chino Valley, two production wells in the Prescott Airport Area, 25 storage tanks, 38 booster pump stations, 72 pressure reducing valve (PRV) stations and is currently divided into 82 pressure zones.

The City's water distribution system hydraulic model was used in this analysis. This model was updated and validated as part of the 2017 Water and Wastewater Models Study (2017 Model Study).

## 2.2 Performance Criteria

### 2.2.1 Background

Performance criteria are the standards of measurement used to evaluate the adequacy of water system infrastructure including supply, storage, booster pumping, and distribution system capacity. Performance criteria are based on legal requirements and engineering best practices. The criteria in this document have been reviewed with City staff and represent the level of service the City strives to provide to its customers. The water system performance criteria have not changed since the 2013 Study.

According to the Arizona Administrative Code (A.A.C.), public water systems shall be designed using good engineering practice (A.A.C. R-18-5-502). The City's water system performance criteria includes standards from the A.A.C., Engineering Bulletin No. 10 (issued by the Arizona Department of Health Services, May 1978), water industry best practices, and criteria established in the 2004 Water Distribution Model Study and 2013 Study using data collected by the City. The City's water system performance criteria are considered good engineering practice and provide acceptable levels of water system performance and reliability.

### 2.2.2 Water System Components

The City's water system consists of the following components:

- Groundwater wells in the Chino Valley well field and Airport Area wells
- Power sources
- PRV stations
- Booster pump stations
- Storage tanks
- Transmission and distribution mains

The function of these water system components and their associated performance criteria is discussed in the following sections.

### 2.2.3 Basis of Criteria

The acceptable level of service expected from the water system is defined by the adequacy and reliability of the water supply delivered to the customer. A reasonable level of service usually includes the provision for adequate system pressure, fire protection, and supply reliability. Therefore, water system performance criteria address the following areas:

- **Water Supply Redundancy:** a level of service such that water supplies can be delivered into the distribution system from more than one source.
- **Water System Reliability:** a level of service such that the distribution system infrastructure can deliver water to as many areas as possible even when a key facility is not in service.
- **System Operational Requirements:** a level of service such that water can be delivered reliably under fire flow, maximum day demand, and peak hour demand conditions.

#### 2.2.3.1 Water Supply Redundancy

Water supply redundancy refers to the degree to which water can be supplied to the City's customers in the event that one or more of the water supply sources is unavailable. Decisions about the extent of redundancy are often policy decisions influenced by the price a utility is willing to pay for redundancy compared to the risk of having to implement water use restrictions or provide a lower level of service to the customer if a water supply source is unavailable. Under some conditions, it may be more economical for the City to implement water demand management or conservation measures rather than build infrastructure that will be used infrequently in response to water source availability.

Although no firm guidelines exist, many communities seek to provide redundant or backup water supplies for average day demand conditions because the average day demand provides sufficient water for public health and safety.

#### 2.2.3.2 Water System Reliability

The City's water system reliability is dependent on the reliability of all the components within the system and the reliability of the energy sources that supply the pump stations and wells. The level of reliability provided is usually based on historic operational experience and judgment, which results in confidence that the system can deliver water under a variety of normal and emergency conditions. Consequently, professional judgment must be used when specifying system components and the number and location of components needed to meet reliability criteria.

Reliability of the City's water system is provided by a combination of the following factors:

- Sufficient water sources to meet maximum day demand,
- Reserve system storage to meet emergency conditions, in addition to fire and normal operational needs,
- Transmission capacity to deliver water to the distribution system,
- Looped transmission and distribution system networks,
- Sufficient booster pumping capabilities with a pump station or the largest pump in a pump station out of service, and
- Backup power supply for critical facilities

#### 2.2.3.3 Water System Operational Requirements

System operational requirements provide for a defined level of service from the City to the customer. Levels of service include maximum and minimum pressures, maximum flow velocities, storage, redundancy, and provisions for emergency conditions. Adequate pressure is usually defined in terms of a minimum pressure under certain demand conditions, such as peak hour or fire flow. Adequate fire protection refers to providing adequate flow to meet firefighting demands. The water system is considered to be adequate when system demand conditions are satisfied while meeting system performance criteria, such as system pressure, velocity, and head loss.

### 2.2.4 Water Production Facilities

Production facilities for the water system should have sufficient capacity to meet the demands of the maximum day of the year.

State regulations regarding emergency operation plans require that municipalities be equipped to address emergency conditions, such as loss of a source of water supply. The City has provisions for emergency conditions codified in Prescott City Code (PCC) 3-10-11.

### 2.2.5 Fire Flow

Fire flow requirements are usually determined by the local fire department. The City has adopted the 2012 International Building Code (2012 IBC) and International Fire Code (2012 IFC), which specifies fire flow requirements for different types of building construction. The City may also consider establishing unique fire flow requirements of the wooded residential areas in parts of the City that are adjacent to the national forest (urban/wild land interface).

For one- and two-family dwellings, the 2012 IFC is specific for the minimum required fire flows as follows:

- < 3,600 square foot fire area: 1,000 gallons per minute (gpm) for 1 hour.
- ≥ 3,600 square foot fire area: refer to 2012 IFC Appendix B, Table B105.1.

Depending on the type of use, construction and fire area, the required fire flow and duration for fire areas greater than or equal to 3,600 square feet ranges from 1,500 gpm for 2 hours to 8,000 gpm for 4 hours.

Standard engineering practice is to assume that a major fire will not occur during the peak hour of the day, since the chance of this happening is minimal. It is more likely that a fire could occur under maximum day demand conditions. Consequently, this condition was used to evaluate water system infrastructure.

To assess the adequacy of the Prescott water system with respect to maximum day demand plus fire demand conditions, a land-use based approach was taken to assign fire flow requirements and durations as summarized in Table 3.5.

Table 3.5 Land Use Fire Flow Criteria

| Type of Development (Land Use) | Fire Flow (gpm) | Duration (hours) |
|---|---|---|
| Single Family Residential 1 (1) | 1,000 | 2 |
| Single Family Residential 2 (2) | 1,500 | 2 |
| Multi-Family Residential | 2,500 | 2 |
| Commercial – Low Risk | 2,500 | 3 |
| Commercial – High Risk | 4,000 | 4 |
| Industrial – Low Risk | 6,000 | 4 |
| Industrial – High Risk | 10,000 | 4 |

Notes:
(1) Residential area originally designed with fire flow standard less than 1,500 gpm.
(2) Residential area originally designed with fire flow standard of 1,500 gpm.

Some areas of the City were originally designed with a lower fire flow standard. In these parts of the City, the fire flow delivery capability is not raised to current standards arbitrarily, but may be raised on a case-by-case basis, driven by the economic viability of raising the standard.

### 2.2.6 Pump Stations

Pump stations are often the most critical components in a distribution system with respect to meeting reliability/redundancy criteria, because these facilities are subject to disruption by power outages, mechanical failures or line breaks.

Table 3.6 summarizes these conditions and the associated reliability criteria.

Table 3.6 Booster Pump Station Reliability Criteria

| Condition | Result | Criteria |
|---|---|---|
| Power Outage | Creates loss of pumping capacity at one or more pumping facilities. | Provide emergency backup power supply or dual power feed to critical facilities. |
| Mechanical Failure | Creates loss of pumping capacity due to pumps, electrical controls or other components being out of service. | Produce sufficient pumping capacity to each booster pump station to meet maximum day demands with the largest pump out of service ("firm" capacity). |
| Line Break | Occurrences at or near the booster station, creates a loss of all or a portion of the pumping capacity of the facility. | Mitigate short-term (less than 24 hours) disruption in supply caused by a line break by providing multiple pumping facilities, storage, looped transmission/distribution lines, PRV stations throughout the system. |

For line breaks affecting critical pumping facilities, reliability/redundancy criteria are established so that average day rather than maximum day demand conditions can be met in each pressure zone in the distribution system.

When pumping to a closed system with no other sources or elevated storage, a pump station should be sized for the larger of peak hour demand or maximum day demand plus fire flow demand conditions. Diurnal demands and fire demands will be supplied through the pumps. Pump stations should be designed based on the firm capacity that can be consistently provided with the largest pump out of service. In addition, pump stations that deliver water into higher pressure zones must be sized to meet the demands of both zones.

The booster pump station criteria are:

- When pumping to a closed system, the capacity equals the larger of peak hour demand or maximum day demand plus fire flow.
- The allowance for reliability and uncertainties in demand projections equals 10 percent.
- Booster pump stations should be sized to meet demands with the largest pump out of service (firm capacity) except when a single high capacity pump is required only for fire flow.
- When multiple booster pump stations supply a pressure zone, if the largest station is out of service, the remaining stations should be able to supply average day demands.

The firm capacity of booster pump stations that pump from tanks is often set so that half of the tank can be emptied in a six-hour period. These booster pump stations should also have a pumping capacity that exceeds the well capacity feeding the storage tank.

### 2.2.7 Transmission and Distribution Mains

Water system piping serve three basic purposes:

- To transfer water from the source of production to storage
- To provide a conduit for domestic water supply
- To provide a conduit for firefighting water

Water distribution mains should be looped and interconnected wherever possible so that in the event of a fire, a failure of a portion of the distribution system, or another emergency, there is more than one path for water to flow to supply customer demands and fire flows.

Transmission and distribution mains are sized for the greater of the following two demand conditions:

- Maximum day demand plus fire flow, or
- Peak hour demand.

The following pressure criteria are required for the distribution system:

- **Maximum Day Demand plus Fire Flow:** a minimum of 20 pounds per square inch (psi) at the point of maximum fire draft.
- **Peak Hour Demand:** a minimum service pressure of 40 psi.

The City's plumbing code (2012 International Plumbing Code) requires service line PRVs when distribution system pressures exceed 80 psi. Due to Prescott's topography and pressure zone elevation ranges, there are many areas which require service line PRVs.

The recommended pipeline maximum water velocity and head loss criteria are listed in Table 3.7 for maximum day, peak hour, and fire flow conditions.

Table 3.7 Water Main Velocity and Head Loss Criteria

| Condition | Pipe Diameter (in) | Maximum Velocity (fps) | Maximum Head Loss (ft head loss per 1,000 ft of pipe) |
|---|---|---|---|
| Maximum Day | < 36 | ≤ 5 | 2 – 7 |
| | ≥ 36 | ≤ 6 | 1 – 2.5 |
| Peak Hour | All | ≤ 8 | < 10 |
| Fire Flow | All | ≤ 10 | NA |

Abbreviations:
in = inch; fps = feet per second

### 2.2.8 Storage Facilities

Because production facilities are designed to operate at a steady rate over an extended period of time, storage tanks are used to accommodate fluctuating demands. Storage tanks should be designed and operated to meet daily demand fluctuations, fire demand, and emergency reserve storage, while achieving storage turnover to minimize water quality degradation.

#### 2.2.8.1 Storage for Diurnal Demands

The storage capacity required to meet diurnal demand fluctuation is the volume of water required to meet the peak hour demands exceeding the maximum day demand production rate (the difference between maximum day and peak hour). For storage volume planning, a conservative value of 20 percent of maximum day demand was used to evaluate storage capacity.

#### 2.2.8.2 Storage for Fire Demand

The fire flow duration for determining storage requirements is determined by the local fire department, but generally ranges from 2 to 4 hours for single fire flow occurrences within a pressure zone. For planning purposes, a land use and zoning approach was taken to develop fire flow and storage requirements. The required fire flow storage by development type is summarized in Table 3.8.

Table 3.8 Storage Required for Fire Flow

| Type of Development | Maximum Fire Flow | | Duration | Fire Storage |
|---|---|---|---|---|
| | (gpm) | (mgd) | (hours) | (MG) |
| **Fire Flow and Storage** (1) | | | | |
| Single Family Residential 1 (2) | 1,000 | 1.4 | 2 | 0.12 |
| Single Family Residential 2 (3) | 1,500 | 2.2 | 2 | 0.18 |
| Multi-Family Residential | 2,500 | 3.6 | 2 | 0.30 |
| Commercial – Low Risk | 2,500 | 3.6 | 3 | 0.45 |
| Commercial – High Risk | 4,000 | 5.8 | 4 | 0.96 |
| Industrial – Low Risk | 6,000 | 8.6 | 4 | 1.44 |
| Industrial – High Risk | 10,000 | 14.4 | 4 | 2.40 |

Notes:
(1) The City's Fire Marshall determines fire flow requirements for new construction. Reductions in maximum fire flow may be allowed under the 2012 IFC and upon approval from the Fire Marshall.
(2) Residential area originally designed with fire flow standard less than 1,500 gpm.
(3) Residential area originally designed with fire flow standard of 1,500 gpm.

There are some older developments in the City where the water infrastructure was designed using standards with lower required fire flows and storage requirements than are contained in the City's current performance criteria. In some of these areas, it may not be economically feasible to increase storage volumes to meet fire storage requirements because land may not be available for new or expanded tanks. In these cases, fire storage deficiencies may be met by utilizing available pumping capacity from lower pressure zones.

#### 2.2.8.3 Storage for Emergency Reserve

Emergency or reserve storage capacity is an additional volume of water that is held in the tank to meet various emergency conditions, such as facility outage. Emergency reserve storage is also available to provide reliability/redundancy to adjacent pressure zone through booster pump stations. The volume of emergency storage that a utility should plan for is largely based on professional judgment, and is influenced by a number of factors such as power outage history, line break frequency, and overall supply redundancy. For storage volume planning, a value of 10 percent of maximum day demand was used for emergency reserve storage.

### 2.2.9 Performance Criteria Summary

Table 3.9 summarizes the City's water system performance criteria. These criteria were used in the distribution system capacity evaluation to determine the adequacy of the water system, and for planning infrastructure improvements.

Table 3.9 Water System Performance Criteria Summary

| Description | Criteria | | | |
|---|---|---|---|---|
| **Water Production** | Maximum day demand for existing system evaluation; Maximum day demand + 10% reserve for future planning | | | |
| **Water Storage** | | | | |
| Equalizing (Diurnal) | 20% of maximum day demand | | | |
| Fire | Volume based on development type | | | |
| Emergency | 10% of maximum day demand | | | |
| **Booster Pumping** | | | | |
| Without Elevated Storage | Firm capacity equal to larger of peak hour or maximum day + fire flow + 10% | | | |
| Firm Capacity | Capacity with the largest pump out of service | | | |
| **Transmission/Distribution Pipes** (1) | | | | |
| Maximum day | | | | |
| Pipes < 36-in | ≤ 5 fps | | | |
| Pipes ≥ 36-in | ≤ 6 fps | | | |
| Peak hour | ≤ 8 fps | | | |
| Fire flow | ≤ 10 fps | | | |
| **System Pressure Criteria** (2) | | | | |
| Minimum | ≥ 40 psi | | | |
| Maximum | ≤ 120 psi | | | |
| Fire flow | ≥ 20 psi | | | |
| Type of Development | Maximum Fire Flow | | Duration | Fire Storage |
| | (gpm) | (mgd) | (hours) | (MG) |
| **Fire Flow and Storage** (3) | | | | |
| Single Family Residential 1 (4) | 1,000 | 1.4 | 2 | 0.12 |
| Single Family Residential 2 (5) | 1,500 | 2.2 | 2 | 0.18 |
| Multi-Family Residential | 2,500 | 3.6 | 2 | 0.30 |
| Commercial – Low Risk | 2,500 | 3.6 | 3 | 0.45 |
| Commercial – High Risk | 4,000 | 5.8 | 4 | 0.96 |
| Industrial – Low Risk | 6,000 | 8.6 | 4 | 1.44 |
| Industrial – High Risk | 10,000 | 14.4 | 4 | 2.40 |

Notes:
(1) Pipe head loss should be less than 10 ft per 1,000 linear feet of pipe.
(2) City's plumbing code requires service line PRVs in areas where static pressures are greater than 80 psi.
(3) The Adopted International Fire Code determines fire flow requirements for new construction. Reductions in maximum fire flow may be allowable under the 2012 IFC upon approval from the Fire Marshall.
(4) Residential area originally designed with fire flow standard less than 1,500 gpm.
(5) Residential area originally designed with fire flow standard of 1,500 gpm.
Abbreviations:
in = inch; fps = feet per second; MG = million gallons; ft = feet

## 2.3 Water Production

The City's maximum day water demand projections were used to evaluate the adequacy of the City's water supply. For year 2017, maximum day demands were compared to the firm production capacity (largest well out of service). For years 2022 through 2032, maximum day demands plus 10 percent were compared to the firm production capacity. New well sources were added as required in each planning year to increase supplies to meet demands.

## 2.4 Distribution System

This capacity of the distribution system piping was evaluated using the City's hydraulic model, the demands prepared for the model update as described in Section 1.2.2, and the City's water system performance criteria. There are several areas that require distribution system improvements so that the required fire flows can be delivered or the required peak hour pressures can be met. These recommended pipe improvements include areas identified in the City's 2013 Study as well as one additional area, including:

- Highway 69 from the New Zone 56/76 Booster Station to the new Zone 56 Tank
- Stony Creek Drive and Northridge Drive
- River Oaks Road & Shinnery Road and Valley Road & Tabosa Road
- From the Virginia Pump Station to Haisley Road & Valley Ranch Road
- Thumb Butte Road to the Thumb Butte Tank and to Upper Thumb Butte Tank
- Gail Gardner Drive from Fair Road to Linwood Road
- Pine Lakes Road
- Iron Springs Road
- Zone 61, 41, 40, 0 in various locations
- Zone 31
- Zone 51
- Buttermilk Drive
- Arrowhead Road from Iron Springs Road to Sidewinder Road
- White Cloud Road, Meadow Ridge Road and Estrella Road
- Highland Avenue from Evergreen Road to Copper Basin Road (2017 Model Study)

## 2.5 Infrastructure for Growth Areas

The Intermediate Storage Tanks and Booster Pump Station (planned for Year 2019) are needed to serve growth in the Deep Well Ranch Area. Future connections in Deep Well Ranch and adjacent areas were assumed to be supported by the Intermediate Storage Tank and a separate pump station (CIP Project 156 W) to serve future Pressure Zone 110.

The Intermediate Storage Tanks and Booster Pump Station project will include a fill valve to allow Airport (Pressure Zone 12) supplies to enter the Intermediate Storage Tanks where they would then be available to serve existing customers as well as future growth in Pressure Zone 110.

Table 3.10 summarizes the pipes required for each growth area.

Table 3.10 Growth Area Water Pipes

| Growth Area and Pipe Size (inch) | Length (ft) | Total Length (ft) |
|---|---|---|
| **Deep Well Ranch** | | **78,581** |
| 12 | 65,877 | |
| 16 | 12,705 | |
| **Granite Dells Estates** | | **66,421** |
| 8 | 53,735 | |
| 12 | 6,257 | |
| 16 | 6,428 | |
| **Granite Dells Ranch** | | **7,219** |
| 8 | 1,888 | |
| 12 | 3,250 | |
| 16 | 2,081 | |
| **Granite Dells Ranch North** | | **17,991** |
| 8 | 6,529 | |
| 12 | 4,869 | |
| 16 | 6,593 | |
| **Granite Dells Ranch South** | | **30,649** |
| 8 | 19,364 | |
| 12 | 11,285 | |
| **Storm Ranch** | | **16,375** |
| 12 | 16,375 | |
| **Yavapai Medical** | | **12,439** |
| 12 | 10,887 | |
| 16 | 1,553 | |

## 2.6 Water Capital Improvement Project Summary

Figure C.1 in Appendix B shows the locations of the recommended water system capital improvement projects and the projects required for growth areas. The project numbers correspond to the numbering convention adopted in the 2013 Study. A tabulation of all water projects is included in Section 4 – Costs and Project Timing.

The IIP Service Areas developed in the 2013 Study were modified to include the growth areas identified in the 2017 Model Study. Additionally, the IIP Service Areas have been reduced to 2 Service Areas. Area A includes the entire water service area, Area B begins just north of the Airport and continues south to the City limits. All projects identified in the 2017 Model Study were assigned a Service Area.

# Section 3 – Wastewater System Evaluation

## 3.1 Wastewater System Description

The City's wastewater system contains over 355 miles of gravity mains, 18 miles of force mains, 61 sewage lift stations and three wastewater treatment facilities. Some parts of the City are not sewered, so residents in those locations have individual septic systems.

The City is in the process of building the infrastructure that will allow all wastewater to be treated at the Airport WRF. Once this infrastructure is in place, the City will decommission the Sundog WWTP and utilize the site as a flow equalization basin with a lift station that will pump wastewater to the Airport WRF. The Hassayampa Water Reclamation Plant (WRP) will continue to operate as a flow scalping facility to provide reclaimed water for golf course turf irrigation.

The City's wastewater collection system hydraulic model was used in this analysis. This model was updated and validated as part of the 2017 Model Study.

## 3.2 Performance Criteria

### 3.2.1 Background

Performance criteria are the standards of measurement used to evaluate the adequacy of wastewater collection system infrastructure including pipe (gravity and force main) and lift station capacity. Performance criteria are based on legal requirements and engineering best practices. The criteria in this document have been reviewed with City staff and represent the level of service the City strives to provide to its customers. The wastewater system performance criteria have not changed since the 2013 Study.

### 3.2.2 Pipe Capacities

Sewer capacities are dependent on many factors. These include roughness of pipe, maximum allowable depth of flow, limiting velocity, and pipe slope. The Continuity Equation and Manning's Equation are used to calculate sewer capacity under steady-flow hydraulic conditions. The Manning's coefficient 'n' is a friction coefficient that varies with respect to pipe material, size of pipe, depth of flow, smoothness of joints, root intrusion, and other factors.

For gravity sewers, the Manning's coefficient ranges between 0.011 and 0.017. For planning purposes, an 'n' value of 0.013 is used for this project, except where modified during calibration of the wastewater system model to actual performance data, or where the pipe material is known. It should be noted that the A.A.C. requires the use of 0.013 for the design of new sewers (A.A.C. R18-9-E301(D)(2)(e))

### 3.2.3 Flow Depth (d/D)

When designing sewers, it is common practice to adopt variable flow depth capacity criteria for various pipe sizes. This criterion is expressed as a ratio of maximum depth of flow (d) to pipe diameter (D). Design d/D ratios typically range from 0.5 to 0.75, with the lower values typically used for smaller pipes that may experience flow peaks greater than planned or may experience blockages from debris.

The Arizona Administrative Code requires that the d/D ratio for new sewers shall not exceed 0.75 for peak dry weather flow conditions (A.A.C. R-18-9-E301(D)(2)(e)). The flow depth criterion used in this study for new sewers is 0.5 for diameters less than 12 inches, and 0.75 for diameters 12 inches and greater. However, existing sewers were evaluated based on a flow depth criteria of 0.9 at peak flows because there are fewer unknowns, especially in established, built out areas, and because there is no need to replace or provide relief for an existing sewer until flows are at the design capacity of the pipe (A.A.C. R-18-E301(D)(b)(i)). The hydraulic criteria used for sizing proposed future gravity sewers will have a greater factor of safety than the criteria used to evaluate the capacity of the existing system due to the uncertainties in making projections of future flows.

### 3.2.4 Velocity

In order to minimize the settlement of solids in the flow and promote scouring, it is standard design practice to specify that a minimum velocity of 2 fps be maintained when the pipe is flowing half full. At this velocity, the sewer flow will typically provide adequate scouring to clean the pipe. Due to the hydraulics of a circular pipe, the velocity for half-full pipe flow approaches the velocity of nearly full pipe flow. The Arizona Administrative Code requires new sewers to be designed with minimum slopes calculated from Manning's Equation using a roughness coefficient of 0.013 and a velocity of 2 fps when flowing full (A.A.C. R-18-9-E301(D)(2)(e)).

Table 3.11 lists the minimum slopes for maintaining self-cleaning velocities at full flow with d/D = 1.0, which provides the most conservative minimum slope. The minimum slope listed in the table is 0.0008 feet per foot (ft/ft), which is the minimum practical slope for gravity sewer construction.

Table 3.11 Recommended Minimum Slopes for Circular Gravity Sewers

| Pipe Diameter (in) | Minimum Slope (ft/ft) (1)(2) | Pipe Capacity (3) | |
|---|---|---|---|
| | | (gpm) | (mgd) |
| 8 | 0.0034 (4) | 310 | 0.45 |
| 10 | 0.0025 | 485 | 0.70 |
| 12 | 0.0020 | 700 | 1.02 |
| 14 | 0.0016 | 960 | 1.38 |
| 15 | 0.0015 | 1,100 | 1.59 |
| 16 | 0.0014 | 1,250 | 1.80 |
| 18 | 0.0012 | 1,580 | 2.28 |
| 20 | 0.001 | 1,960 | 2.82 |
| 21 | 0.001 | 2,160 | 3.11 |
| 24 | 0.0008 | 2,820 | 4.06 |
| >24 | 0.0008 | -- | -- |

Notes:
(1) Slopes are calculated using Manning's Equation for pipes flowing full with a minimum velocity of 2 fps.
(2) Sewers larger than 24-inches should have a slope ≥ 0.0008.
(3) Based on pipe flowing full (d/D = 1.0)
(4) Prescott prefers a slope of 0.005 (0.5 percent), for 8-inch diameter pipes where possible.

Greater slopes are desirable if they are compatible with existing topography and infrastructure, provided that the wastewater velocity does not exceed the maximum velocity criteria of 10 fps established in A.A.C. R-18-9-E301(D)(2)(f) unless scour resistant material is used. Velocities greater than 10 fps may also result in turbulent flow conditions that contribute to odor problems.

The slopes listed in Table 3.11 are target criteria for existing and proposed sewer pipes. However, it should be noted that some of the existing sewer pipes in the City's collection system have flat slopes, and the minimum slope criteria may not be met in all locations with the wastewater system.

### 3.2.5 Manhole Spacing

Manholes are typically installed at grade changes, changes in sewer pipe sizes, alignment changes, and intersections with other sewer pipes. In addition, manholes should be located to facilitate sewer cleaning. The recommended maximum manhole spacing for different diameters of sewer pipe are listed in Table 3.2. The recommended maximum spacing is in accordance with A.A.C. R18-9-E301(D)(3)(a).

Table 3.12 Recommended Maximum Manhole Spacing

| Sewer Pipe Diameter (in) | Maximum Manhole Spacing (ft) (1) |
|---|---|
| Less than 8-in | 400 |
| 8-in to less than 18-in | 500 |
| 18-in to less than 36-in | 600 |
| 36-in to less than 60-in | 800 |

Notes:
(1) A.A.C. R18-9-E301(D)(3)(a)

### 3.2.6 Changes in Pipe Size

When a smaller sewer joins a larger sewer, the invert of the larger sewer will be lowered sufficiently to maintain the same energy gradient across the manhole. The GIS data for the City's wastewater system was used for the sewer inverts. For master planning purposes, proposed sewer crowns were matched at manholes when a smaller sewer joins a larger one.

### 3.2.7 Lift Stations

Lift stations should be sized for a "firm" capacity greater than the peak daily flow. The lift station should be able to provide a firm pumping capacity with the largest pump out of service. This same concept applies to package lift stations with equally sized, duplex pumps where one pump acts as the duty pump and the other as the standby pump. In these cases, the required pumping capacity should be provided by the duty pump.

#### 3.2.7.1 Normal Operation

Lift station wet well sizing takes into consideration the fill time at average flow conditions and the minimum pump cycle time. The minimum wet well volume shall be per the City of Prescott General Engineering Standards. When selecting the minimum cycle time, the pump manufacturer's duty cycle recommendations will be utilized. Starting and stopping more than seven times an hour for any one pump is not recommended.

#### 3.2.7.2 Emergency Operation

The objective of emergency operation is to protect public health by preventing sewer back-ups and subsequent discharge into streets and other public or private property. The most common emergency would be a power outage. The City has permanent back-up generators with automatic transfer switches at their regional lift stations and portable generators that can be used at all other stations. The City requires emergency generators to be installed at all new lift stations.

### 3.2.8 Force Mains

The Arizona Administrative Code requires that new force mains be designed to maintain a minimum flow velocity of 3 fps and a maximum flow velocity of 7 fps (A.A.C. R-18-9-E301(D)(4)(a)). These velocity criteria promote scouring so that the solids deposited in the force main while the pumps are off will be transported downstream when the pumps are operating. Wastewater retention time in the pipeline should also be considered in sizing force mains to avoid excessive hydrogen sulfide generation.

### 3.2.9 Gravity Sewer Planning Guidelines

Gravity sewers should be designed and constructed to have:

- A minimum of 4 feet of cover or sufficient depth to serve the ultimate drainage area
- A target depth of 7 feet of cover for new sewer main installation

Gravity sewers and force mains must have a minimum separation of 6 feet from potable water mains and reclaimed water mains unless they are provided with increased protection in accordance with A.A.C. R-18-5, Article 4 and A.A.C. R-18-9, Article 6.

Manholes with sewers intersecting at greater than or equal to 90-degree angles should provide 0.1 foot of invert drop across the manhole. Other manholes should provide a minimum 0.1 foot of invert drop.

### 3.2.10 Peaking Factors

Peaking factors for the City's wastewater system were calculated based on field data from the flow monitoring conducted in the spring of 2017. The peak hour to average daily flow ratio during the flow monitoring period was determined for each of the flow monitoring locations.

Table 3.13 summarizes the peak flow multipliers for each flow monitoring basin. The values range from 1.6 to 2.5, and are dependent on the size of the drainage area and type of development contributing to the flow monitoring point.

Table 3.13 Peak Hour Wastewater Flow Factors

| Flow Monitoring Basin | Peak Flow Multiplier |
|---|---|
| North Force Main | 1.66 |
| Pinion Oaks | 2.41 |
| City Lights | 2.14 |
| Robinson | 2.29 |
| Prescott Lakes Parkway | 1.69 |
| Forest Trails | 2.27 |
| Hassayampa | 2.52 |
| Prescott Heights | 1.63 |
| Banning Creek | 1.64 |
| Copper Basin | 1.62 |
| Gurley | 1.96 |

### 3.2.11 Storm Inflows

Storm inflows are based on a review of the historical flow data to the Sundog and Airport Plants and the wet weather flow analysis performed in the winter of 2004. The storm flow for the Sundog Basin was 6.8 mgd and the storm flow for the Airport Basin was 1.3 mgd.

Future inflow is modeled as an additional 30 percent of future dry weather flow in the Sundog Basin and 15 percent of future dry weather flow in the Airport Basin.

### 3.2.12 Performance Criteria Summary

Table 3.14 summarizes the City's wastewater collection system performance criteria.

Table 3.14 Wastewater System Performance Criteria Summary

| Pipe Diameter (in) | Minimum Slope (ft/ft) (1)(2) | Pipe Capacity (3) (gpm) | (mgd) |
|---|---|---|---|
| **Gravity Sewer Minimum Slope and Capacity** | | | |
| 8 | 0.0034 (4) | 310 | 0.45 |
| 10 | 0.0025 | 485 | 0.70 |
| 12 | 0.0020 | 700 | 1.02 |
| 14 | 0.0016 | 960 | 1.38 |
| 15 | 0.0015 | 1,100 | 1.59 |
| 16 | 0.0014 | 1,250 | 1.80 |
| 18 | 0.0012 | 1,580 | 2.28 |
| 20 | 0.001 | 1,960 | 2.82 |
| 21 | 0.001 | 2,160 | 3.11 |
| 24 | 0.0008 | 2,820 | 4.06 |
| >24 | 0.0008 | -- | -- |

| Description | Criteria |
|---|---|
| **Maximum Velocity** | ≤ 10 fps (polyvinyl chloride [PVC] pipe)<br>> 10 fps (scour resistant pipe) |
| **Flow Depth, d/D (dry weather peak)** | |
| d/D for evaluating existing mains | 0.9 |
| d/D for planning new pipes < 12-in diameter | 0.5 |
| d/D for planning new pipes ≥ 12-in diameter | 0.75 |
| **Head Loss in Existing Pipes** | |
| Gravity main | Manning's "n" = 0.013 |
| Pressure pipes | Hazen Williams "C" = 120 |
| **Changes in Pipe Size** | |
| When a smaller sewer joins a larger sewer | Sewer crowns will be matched at a minimum or an internal drop at the transition manhole will be provided |
| **Head Loss at Manholes** | |
| Manholes with pipes intersecting at angles greater than 90 degrees | Provide 0.1-ft invert drop |
| Manholes with pipes intersecting at angles 90 degrees or less | Provide 0.1-ft invert drop |
| **Collection System Peaking Factors** | |
| Peak flow to average daily flow (5) | 1.6 – 2.5 |
| **Inflow and Infiltration** | |
| Sundog service area | 6.8 mgd existing + 30% of future dry weather flow |
| Airport service area | 1.3 mgd existing + 15% of future dry weather flow |

Notes:
(1) Slopes are calculated using Manning's Equation for pipes flowing full with a minimum velocity of 2 fps.
(2) Sewers larger than 24-inches should have a slope ≥ 0.0008.
(3) Based on pipe flowing full (d/D = 1.0) at the minimum pipe slope.
(4) Prescott prefers a slope of 0.005 (0.5 percent), for 8-inch diameter pipes where possible.
(5) Values measured during flow monitoring study conducted for 2017 Water and Wastewater Models Study.
Abbreviation:
d/D = depth over diameter

## 3.3 Collection System

Most of the pipes in the City's collection system have sufficient capacity to convey the existing and projected future flows. There are several areas where the estimated depth over diameter (d/D) exceeded 0.9 for 2017 flow conditions. These areas were previously identified in the 2013 Study and include:

- Sewers north of Virginia Street near Mount Vernon Street
- Sewers on Willow Creek Road, Rosser Street, and Demerse Avenue
- Sewers on Fifth Street, Sixth Street, and Hillside Avenue from the Sundog Trunk Main to Fifth Street
- Sewers on Granite Street from north of Aubrey Street to Sheldon Street
- Sewer on Sun Drive east of Scott Drive
- Sundog Trunk Main Phase I – Sundog WWTP to Highway 89
- Sundog Trunk Main Phase II – Sundog WWTP to Miller Valley Road
- Sewer on Josephine/Osburn Road from Plaza Drive to Miller Valley Road
- Sewer on Thumb Butte Road from Meadowbrook Road to Country Club Drive
- Sewer on Meadowbrook Road from Butte Canyon Drive to 200 feet east

The sewer on Prescott Lakes Parkway north of SR69 was identified in the model with a d/D of greater than 0/9 for 2017 conditions. However, the model predicted surcharging is limited and the City's field review of this area do not indicate a need for a capital improvement project at this time. It is recommended that the City continue to monitor this line, particularly if additional connections immediately upstream of this location are made or if the Ranch 1 lift station is ever modified with higher capacity pumps.

## 3.4 Infrastructure for Growth Areas

There are three lift stations shown on Figure C.2 in Appendix B, considered for future growth areas:

- Granite Dells Ranch (commercial) and Granite Dells Estates – New lift station on the east side of Granite Creek
- Deep Well Ranch – New lift station north west of the Prescott Airport
- Yavapai Regional Medical Center – one lift station on site, which will also convey flow to the new lift station north west of the Prescott Airport

Table 3.12 summarizes the pipes required for each growth area.

Table 3.12 Growth Area Wastewater Pipes

| Growth Area and Pipe Size (inch) | Length (ft) | Total Length (ft) |
|---|---|---|
| **Deep Well Ranch** | | **70,596** |
| 8 | 27,567 | |
| 10 | 9,309 | |
| 12 | 17,470 | |
| 15 | 5,787 | |
| 18 | 10,463 | |
| **Granite Dells Estates** | | **42,207** |
| 8 | 39,204 | |
| 10 | 1,197 | |
| 12 | 1,807 | |
| **Granite Dells Ranch** | | **12,788** |
| 8 | 6,839 | |
| 10 | 2,661 | |
| 12 | 3,288 | |
| **Granite Dells Ranch North** | | **5,933** |
| 10 | 5,651 | |
| 24 | 282 | |
| **Granite Dells Ranch South** | | **3,026** |
| 8 | 975 | |
| 10 | 148 | |
| 12 | 1,903 | |
| **Storm Ranch** | | **17,226** |
| 8 | 13,059 | |
| 10 | 3,139 | |
| 12 | 752 | |
| 18 | 276 | |
| **Yavapai Medical** | | **6,571** |
| 8 | 6,571 | |

### 3.5 Wastewater Infrastructure Improvement Plan Summary

Figure C.2 in Appendix B shows the locations of the recommended wastewater system projects required for growth areas. The project numbers correspond to the numbering convention adopted in the 2013 Study. A tabulation of all wastewater projects is included in Section 4 – Costs and Project Timing.

The IIP Service Areas developed in the 2013 Study were modified to include the growth areas identified in the 2017 Model Study. Additionally, the IIP Service Areas have been reduced to a single Service Area with centralization of wastewater treatment. All projects identified in the 2017 Model Study were assigned the single Service Area.

## Section 4– Costs and Project Timing

### 4.1 Cost Development Methodology

Cost estimates have been developed for the water and wastewater capital improvement projects identified in the previous sections. These estimates were prepared in accordance with the guidelines of the Association for the Advancement of Cost Engineers (A.A.C.E) International for a Class 4 estimate unless otherwise noted. Table 3.16 summarized the A.A.C.E International cost estimating classifications, the level of project definition (percent of design), uses, appropriate cost estimating methodologies, and the expected accuracy of each class. Design work would need to be undertaken to obtain more precise cost estimates.

Table 3.16 A.A.C.E International Cost Estimating Classification Summary

| Estimate Class | Maturity Level of Project Definition Deliverables - (Level of Engineering Design) | End Use | Typical Cost Estimating Methodology Used | Expected Accuracy Range (Low/High) |
|---|---|---|---|---|
| Class 5 | 0% to 2% | Conceptual screening | Capacity factored, parametric models, judgment or analogy | L: -20% to -50%<br>H: +30% to +100% |
| Class 4 | 1% to 15% | Study or feasibility | Equipment factored or parametric models | L: -15% to -30%<br>H: +20% to +50% |
| Class 3 | 10% to 40% | Budget authorization or control | Semi-detailed unit costs with assembly level line items | L: -10% to -20%<br>H: +10% to +30% |
| Class 2 | 30% to 75% | Control or bid/tender | Detailed unit cost with forced detailed take-off | L: -5% to -10%<br>H: +5% to +20% |
| Class 1 | 65% to 100% | Check estimate or bid/tender | Detailed unit cost with detailed take-off | L: -3% to -10%<br>H: +3% to +15% |

### 4.2 Unit Costs

Unit costs were developed for the recommended water and wastewater projects using R.S. Means and other unit cost sources, unless otherwise noted. Multipliers for general conditions (15%), construction overhead and profit (16%), sales tax (65% of applicable costs at 9.1%), contingencies (30%), and general conditions (15%) were then added to prepare unit construction costs. When multiplied by the capacity, quantity, or size of infrastructure, the unit construction cost represents what the City should expect to pay a contractor to construct the project. The City will have other expenses to complete the project including design, inspection, project management, and contingencies. A multiplier of 1.4 was used to represent these additional costs and applied to the unit construction cost to obtain a project cost for each project.

Table 3.17 summarizes the water infrastructure unit costs for the 5- to 15-year IIP. The water infrastructure unit cost detail is included in Appendix C.

Table 3.17 Water Infrastructure Unit Costs

| Infrastructure | Unit Construction Cost (1) |
|---|---|
| **Water Pipelines** | **($/LF)** |
| 8-in | $177 |
| 8-in (with hydrant) | $206 |
| 12-in | $195 |
| 12-in (with hydrant) | $230 |
| 16-in | $227 |
| 18-in | $263 |
| 20-in | $298 |
| 24-in | $336 |
| 30-in | $544 |
| 36-in | $635 |
| **Wells** | **($M)** |
| 750 gpm (1.1 mgd) | $2.2 |
| 1,400 gpm (2.0 mgd) | $3.1 |
| **Booster Pump Stations** | **($M)** |
| 1.5 mgd | $1.6 |
| 2.0 mgd | $1.9 |
| 3.0 mgd | $2.2 |
| 4.0 mgd | $2.4 |
| 6.0 mgd | $2.8 |
| 8.0 mgd | $3.7 |
| 10.0 mgd | $4.3 |
| 12.0 mgd | $5.0 |
| 18.0 mgd | $5.7 |
| **Storage Tanks (2)** | **($M)** |
| 0.225 MG (3) | $0.6 |
| 0.5 MG | $1.8 |
| 1.0 MG | $2.3 |
| 1.5 MG | $2.6 |
| 2.0 MG | $3.0 |
| 2.5 MG | $3.6 |
| 3.0 MG | $4.1 |
| 5.0 MG | $6.1 |

Notes:
(1) ENR CCI = 10870 (20 Cities Index, November 2017)

(2) Site-specific conditions impact the bid-based unit costs for tanks. The projects these costs area applied to in the 2017 CIP/IIP are expected to have similar site-specific conditions.
(3) Storage tank unit construction costs are based on current bid pricing provided by the City.

Table 3.18 summarizes the wastewater infrastructure unit costs for the 5- to 15-year IIP. The wastewater infrastructure unit cost detail is included in Appendix C.

Table 3.18 Wastewater Infrastructure Unit Costs

| Infrastructure | Unit Construction Cost (1) |
|---|---|
| **Force Mains** | **($/LF)** |
| 6-in | $163 |
| 8-in | $169 |
| 12-in | $192 |
| 18-in | $374 |
| 20-in | $391 |
| **Gravity Sewers** | **($/LF)** |
| 8-in | $193 |
| 10-in | $205 |
| 12-in | $209 |
| 15-in | $213 |
| 18-in | $220 |
| 24-in | $258 |
| 30-in | $333 |
| 36-in | $441 |
| 39-in | $480 |
| 42-in | $505 |
| 48-in | $628 |
| **Lift Stations** | **($M)** |
| 0.2 mgd | $0.6 |
| 0.5 mgd | $0.7 |
| 0.8 mgd | $0.8 |
| 3.0 mgd | $2.2 |
| 6.0 mgd | $3.3 |
| 9.0 mgd | $4.3 |
| 12.0 mgd | $5.0 |
| 15.0 mgd | $5.6 |

Notes:
(1) ENR CCI = 10870 (20 Cities Index, November 2017)

## 4.3 Water System Project Summary

Table 3.19 summarizes the water system projects needed to serve future growth. The City has two IIP service areas for the water system:

- Area A – includes the entire system from the Chino Valley wells, tanks and booster station.
- Area B – includes all portions of the system except the Chino Valley wells, tanks and booster station.

The number of EDUs that are expected to be added to the system between year 2017 and year 2032 was determined to allocate the portion of new infrastructure cost for new development (fees) with existing customers (rates). For the growth areas shown in Figure 3.1, the ultimate number of EDUs was used as a basis for the new growth cost allocation because the initial phases of development will be laying the back bone infrastructure for these areas and the infrastructure should be sized for ultimate needs of the area that it serves.

The water IIP service areas are shown in Figure C.1 in Appendix B. Table 3. identifies which projects are part of the water IIP, the service area the project is in, and an estimate of the percent cost allocation associated with impact fees and rates based on projected growth within each service area.

Table 3.19 summarizes the water system project costs and construction costs.

## 4.4 Wastewater System Project Summary

Table 3.20 summarizes the wastewater system projects needed to serve future growth. There is one IIP service area for the wastewater system because the City has adopted a centralized wastewater collection and treatment approach. Therefore cost allocations between new development and existing customers will be shared for projects required to serve the entire system.

The wastewater IIP service areas are shown in Figure C.2 of Appendix B. Table 3.20 identifies which projects are associated with the wastewater IIP, the service area the project is in, and estimate of the percent cost allocation associated with impact fees and rates based on projected growth within each service area.

Table 3.20 summarizes the wastewater system project costs and construction costs.

Table 3.19 2018-2032 Water System Infrastructure Improvement Plan

| Project No. | Description | Diameter (in) | Length (ft) | Capacity (gpm or MG) | Unit Cost ($) | Construction Cost ($) | Project Cost ($) | IIP (Y/N) | Service Area | % Fees | % Rates | Planning Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1W | Impact Fee Ordinance Implementation and User Rates Project | | | | | | 450,000 | Y | A | 50 | 50 | 2019 |
| 6W | Water Model Update | | | | | | 120,450 | Y | A | 50 | 50 | 2019 |
| 44 W | Zone 56 Tank and Pipeline and Zone 7 Pump Station | 16 | 4,560 | 1.5 MG | -- | 4,500,000 | 5,300,000 | Y | B | 25 | 75 | 2020 |
| 52 W | New Water Main from Centerpointe/Side Rd. to Heckthorn Rd. | 12 | 2,105 | -- | 230 | 484,000 | 678,000 | Y | B | 100 | 0 | 2025 |
| 56W | Zone 56/76 Booster Pump Station | | | | | 900,000 | 1,010,000 | Y | B | 60 | 40 | 2019 |
| 60 W | Future Airport Well No. 5 - Location not yet determined | -- | -- | 950 gpm | -- | 1,750,000 | 2,248,000 | Y | B | 100 | 0 | 2024 |
| 64 W | Upsize water main along Hwy 69 from new Zone 56/76 booster pump station | 16 | 7,225 | -- | 227 | 1,639,500 | 2,295,000 | Y | B | 35 | 65 | 2024 |
| 66W | Zone 16 Virginia Pump Station, Haisley Tank and Pipelines & Haisley Rd Reconstruction | | | | | 5,237,000 | 5,737,000 | Y | B | 40 | 60 | 2019 |
| 68W | Upsize Water Main from Zone 27 to Zone 24 Tank | | | | | 2,850,000 | 3,170,000 | Y | B | 30 | 70 | 2019 |
| 70 W | Upper Rancho Vista Booster Pump upsize | -- | -- | 1,000 gpm | Prev. Proj. Specific | 600,000 | 755,000 | Y | B | 35 | 65 | 2022 |
| 76 W | Sundog Ranch Rd. Connector Water line between Yavpe Connector and Prescott Lakes Parkway | 12 | 8,826 | -- | 230 | 600,000 | 700,000 | Y | B | 35 | 65 | 2019 |
| 80W | New Zone 61 Water Mains | | | | | 2,000,000 | 2,243,000 | Y | B | 25 | 75 | 2023 |
| 82W | New Zone 41 Water Mains | | | | | 1,300,000 | 1,450,000 | Y | B | 25 | 75 | 2024 |
| 92 W | Chino Valley/Intermediate Pump Station and Tanks | 16 | 5050 | 10 MG | -- | 20,000,000 | 22,600,000 | Y | A,B | 50 | 50 | 2019 |
| 106W | Future Airport Well No. 6 | | | | | 1,750,000 | 2,500,000 | Y | B | 100 | 0 | 2020 |
| 108W | North Airport Distribution Loop | | | | | 850,000 | 1,071,000 | Y | B | 75 | 25 | 2025 |
| 110W | East Airport Distribution Loop | | | | | 650,000 | 834,000 | Y | B | 90 | 10 | 2025 |
| 112W | New Water Main from Hwy 89A to Larry Caldwell Dr | | | | | 1,100,000 | 1,301,000 | Y | B | 90 | 10 | 2023 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 114W | Water Main to Connect Zone 51 to Northwest Regional Tank | 1,250,000 | 1,477,000 | Y | B | 35 | 65 | 2021 |
| | Sundog Trunk Main, Phase C | 250,000 | 300,000 | Y | B | 35 | 65 | 2021 |
| | | | | | | | | |

Table 3.20 2018-2032 Wastewater System Infrastructure Improvement Plan

| Project No. | Description | Diameter (in) | Length (ft) | Capacity (gpm or MG) | Unit Cost ($) | Construction Cost ($) | Project Cost ($) | IIP (Y/N) | Service Area | % Fees | % Rates | Planning Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1WW | Impact Fee Ordinance Implementation and User Rates Project | | | | | | 434,848 | Y | A | 50 | 50 | 2019 |
| 3WW | Wastewater Model Update | | | | | | 210,450 | Y | A | 50 | 50 | 2019 |
| 26 WW | Sundog Trunk Main Phase C from Miller Valley Rd. to Veterans Administration (VA) | Various | 10,990 | -- | | 6,000,000 | 7,150,000 | Y | A | 35 | 65 | 2019 |
| 32 WW.1 | Granite Dells Development (DA) Wastewater Requirements – Airport East Regional Lift Station | -- | -- | 1,000 | | 1,800,000 | 2,340,000 | Y | A | 100 | 0 | 2025 |
| 32 WW.2 | Granite Dells Development (DA) Wastewater Requirements – Airport East Pipelines | 10 | 2,430 | -- | 192 | 466,000 | 652,000 | Y | A | 100 | 0 | 2025 |
| 32 WW.3 | Granite Dells Development (DA) Wastewater Requirements – Airport East Pipelines | 12 | 1,348 | -- | 209 | 282,000 | 395,000 | Y | A | 100 | 0 | 2025 |
| 34 WW.1 | Centralization – Airport Trunk Main | 48 | 10,640 | -- | 628 | 6,678,000 | 9,850,000 | Y | A | 65 | 35 | 2020 |
| 34 WW.2 | Centralization – Airport Trunk Main | 36 | 6,410 | -- | 441 | 1,828,000 | 2,900,000 | Y | A | 35 | 65 | 2020 |
| 36 WW | Montezuma Trunk Main Upsizing – Sewers on Granite St. from north of Aubrey to Sheldon St. | 18 | 3,043 | -- | 220 | 670,000 | 938,000 | Y | A | 25 | 75 | 2022 |
| 38 WW.1 | Hassayampa Sewer Trunk Main Upsizing - Josephine/Osburn from Plaza to Miller Valley Rd. | 8 | 1,131 | -- | 193 | 218,000 | 305,000 | Y | A | 25 | 75 | 2022 |
| 38 WW.2 | Hassayampa Sewer Trunk Main Upsizing - Josephine/Osburn from Plaza to Miller Valley Rd. | 10 | 860 | -- | 205 | 176,000 | 246,000 | Y | A | 25 | 75 | 2022 |
| 38 WW.3 | Hassayampa Sewer Trunk Main Upsizing - Josephine/Osburn from Plaza to Miller Valley Rd. | 12 | 2,862 | -- | 209 | 598,000 | 837,000 | Y | A | 25 | 75 | 2022 |
| 38 WW.4 | Hassayampa Sewer Trunk Main Upsizing - Josephine/Osburn from Plaza Dr. to Miller Valley Rd. | 15 | 5,600 | -- | 213 | 1,192,000 | 1,669,000 | Y | A | 25 | 75 | 2022 |
| 54 WW.1 | Sundog Equalization Basin | NA | NA | -- | Proj. Specific | 1,180,000 | 1,650,000 | Y | A | 35 | 65 | 2023 |
| 54 WW.2 | Plant Decommissioning | NA | NA | -- | | -- | 175,000 | Y | A | 35 | 65 | 2023 |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 56 WW.1 | Upsize Willow Creek Trunk Main from Willow Lake Regional Lift Station west to Cottonwood Ln. | 12 | 4,967 | -- | 209 | 1,038,000 | 1,453,000 | Y | A | 25 | 75 | 2021 |
| 56 WW.2 | Upsize Willow Creek Trunk Main from Willow Lake Regional Lift Station west to Cottonwood Ln. | 15 | 3,926 | -- | 213 | 836,000 | 1,170,000 | Y | A | 25 | 75 | 2021 |
| 56 WW.3 | Upsize Willow Creek Trunk Main from Willow Lake Regional Lift Station west to Cottonwood Ln. | 18 | 1,795 | -- | 220 | 395,000 | 553,000 | Y | A | 25 | 75 | 2021 |
| 58 WW | Airport WRF Phase II Expansion | -- | -- | 7.5 | Proj. Specific | 13,393,000 | 18,750,000 | Y | A | 100 | 0 | 2023 |
| 62 WW.1 | Willow Creek Gravity Sewer from Willow Lake Regional Lift Station to Prescott Lakes Regional Lift Station | 8 | 527 | -- | 193 | 102,000 | 143,000 | Y | A | 35 | 65 | 2020 |
| 62 WW.2 | Willow Creek Gravity Sewer from Willow Lake Regional Lift Station to Prescott Lakes Regional Lift Station | 21 | 6,488 | -- | 224 | 1,453,000 | 2,034,000 | Y | A | 35 | 65 | 2020 |
| | Centralization – Sundog Trunk Main, Phase B | | | | | 2,200,000 | 2,500,000 | Y | A | 35 | 65 | 2019 |