# 4. SYSTEM ANALYSIS

## 4.1 WATER RIGHTS ANALYSIS

The Utility has one water right for surface water diversion from the Cowlitz River for municipal use. Maximum instantaneous flow is 0.50 cubic feet per second (cfs) or about 224 gallons per minute (gpm). A maximum annual volume is not specified.

The water right is Surface Water Certificate No. 9616. The original priority date is November 9, 1961; but the point of withdrawal along the Cowlitz River was slightly changed in 1972 and the priority date changed to November 21, 1972. Water rights issued in the 1960s and 1970s commonly did not include an annual withdrawal amount. Copies of the water right documents are in Appendix C.

Table 4.1 compares the existing water rights with the existing capacity of the system. The source capacity considered the limiting factors of the river pump (200 gpm), raw water pump station (two pumps at 100 gpm), integrity of the raw water transmission line, and water treatment plant (two 100 gpm treatment units). The source capacity was determined to be 200 gpm. Table 4.1 shows that the system is operating within their water right.

**TABLE 4.1 – WATER RIGHTS SUMMARY**

| Source | Certificate | Instantaneous Withdrawal | | |
|---|---|---|---|---|
| | | Water Right (gpm) | Existing Capacity (gpm) | Surplus (+) or Deficit (-) (gpm) |
| Cowlitz River | 9616 | 224 | 200 | 24 |

Tables 4.2 through 4.4 show the status of existing, forecasted 6-year and forecasted 20-year water rights. The tables show that the water right is not the limiting factor for system operation and growth.

Washington State Department of Health
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# TABLE 4.2 - Project Report Form
## Water Rights Self Assessment – Existing Water Right Status

| Permit Certificate or Claim # | Name of rightholder or claimant | Priority Date | Source Name/Number | Primary or supplemental | Existing System Capacity - based on water right limits | | Projects Production/withdrawal with New Project On-line | | Projected System Capacity Status (excess or deficiency of water rights) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Maximum Instantaneous Flow rate (Qi) | Maximum Annual Volume (Qi) | Maximum Instantaneous Flow Rate (Qi) | Maximum Annual Volume (Qa) | Maximum Instantaneous Flow Rate (Qi) | Maximum Annual Volume (Qa) |
| Certificate No. 9616 | Town of Vader | 11/21/72 | Cowlitz River | Primary | 224 gpm | N/A | 200 gpm | N/A | 24 gpm | N/A |
| | | | | | | | | | | |
| | | | | | | | | | | |
| | | | | | | | | | | |
| Claims 1. | | | | | | | | | | |
| 2. | | | | | | | | | | |
| 3. | | | | | | | | | | |
| 4. | | | | | | | | | | |
| Total | | | | | | | | | | |

| Intertie Name/Identifier | Name of Purveyor Providing Water | Existing Limits on Intertie Water Use | | Projected Production/Withdrawal with New Project On-line | | Current Intertie Supply Status (Excess/Deficiency) | |
|---|---|---|---|---|---|---|---|
| | | Maximum Instantaneous Flow rate (Qi) | Maximum Annual Volume (Qa) | Maximum Instantaneous Flow rate (Qi) | Maximum Annual Volume (Qa) | Maximum Instantaneous Flow Rate (qi) | Maximum Annual Volume (Qa) |
| 1. | | | | | | | |
| 2. | | | | | | | |
| 3. | | | | | | | |
| 4. | | | | | | | |
| TOTAL *********** | ****** *********** ************* | | | | | | |

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Office of Drinking Water

# TABLE 4.3 - Project Report Form
## Water Rights Self Assessment – Forecasted 6-Year Water Right Status

| Permit Certificate or Claim # | Name of rightholder or claimant | Priority Date | Source Name/Number | Primary or supplemental | Existing System Capacity - based on water right limits | | Projects Production/withdrawal with New Project On-line | | Projected System Capacity Status (excess or deficiency of water rights) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Maximum Instantaneous Flow rate (Qi) | Maximum Annual Volume (Qi) | Maximum Instantaneous Flow Rate (Qi) | Maximum Annual Volume (Qa) | Maximum Instantaneous Flow Rate (Qi) | Maximum Annual Volume (Qa) |
| Certificate No. 9616 | Town of Vader | 11/21/72 | Cowlitz River | Primary | 224 gpm | N/A | 200 gpm | N/A | 24 gpm | N/A |
| | | | | | | | | | | |
| | | | | | | | | | | |
| | | | | | | | | | | |
| Claims 1. | | | | | | | | | | |
| 2. | | | | | | | | | | |
| 3. | | | | | | | | | | |
| 4. | | | | | | | | | | |
| Total | | | | | | | | | | |

| Intertie Name/Identifier | Name of Purveyor Providing Water | Existing Limits on Intertie Water Use | | Projected Production/Withdrawal with New Project On-line | | Current Intertie Supply Status (Excess/Deficiency) | |
|---|---|---|---|---|---|---|---|
| | | Maximum Instantaneous Flow rate (Qi) | Maximum Annual Volume (Qa) | Maximum Instantaneous Flow rate (Qi) | Maximum Annual Volume (Qa) | Maximum Instantaneous Flow Rate (qi) | Maximum Annual Volume (Qa) |
| 1. | | | | | | | |
| 2. | | | | | | | |
| 3. | | | | | | | |
| 4. | | | | | | | |
| TOTAL *********** | ****** *********** ************* | | | | | | |

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Office of Drinking Water

# TABLE 4.4 - Project Report Form
## Water Rights Self Assessment – Forecasted 20-Year Water Right Status

| Permit Certificate or Claim # | Name of rightholder or claimant | Priority Date | Source Name/Number | Primary or supplemental | Existing System Capacity - based on water right limits | | Projects Production/withdrawal with New Project On-line | | Projected System Capacity Status (excess or deficiency of water rights) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Maximum Instantaneous Flow rate (Qi) | Maximum Annual Volume (Qi) | Maximum Instantaneous Flow Rate (Qi) | Maximum Annual Volume (Qa) | Maximum Instantaneous Flow Rate (Qi) | Maximum Annual Volume (Qa) |
| Certificate No. 9616 | Town of Vader | 11/21/72 | Cowlitz River | Primary | 224 gpm | N/A | 220 gpm | N/A | 24 gpm | N/A |
| | | | | | | | | | | |
| | | | | | | | | | | |
| | | | | | | | | | | |
| Claims 1. | | | | | | | | | | |
| 2. | | | | | | | | | | |
| 3. | | | | | | | | | | |
| 4. | | | | | | | | | | |
| Total | | | | | | | | | | |

| Intertie Name/Identifier | Name of Purveyor Providing Water | Existing Limits on Intertie Water Use | | Projected Production/Withdrawal with New Project On-line | | Current Intertie Supply Status (Excess/Deficiency) | |
|---|---|---|---|---|---|---|---|
| | | Maximum Instantaneous Flow rate (Qi) | Maximum Annual Volume (Qa) | Maximum Instantaneous Flow rate (Qi) | Maximum Annual Volume (Qa) | Maximum Instantaneous Flow Rate (qi) | Maximum Annual Volume (Qa) |
| 1. | | | | | | | |
| 2. | | | | | | | |
| 3. | | | | | | | |
| 4. | | | | | | | |
| TOTAL *********** | ****** *********** ************* | | | | | | |

## 4.2 SOURCE CAPACITY

The water system receives water from a single surface water source, the Cowlitz River. DOH requires source production capacity to be equal to meet Maximum Day Demands (MDD). And the water rights must be sufficient to meet MDD and Average Day Demands (ADD). Since the source capacity of 200 gpm is less than the water right of 224 gpm, the source analysis will make comparisons to the source capacity at an 18-hour production day. Table 4.5 compares current and projected MDD and ADD values with the source capacity. The source capacity can adequately handle demands in the two planning horizons.

| TABLE 4.5 – 18 HOUR SOURCE PRODUCTION CAPACITY ANALYSIS | | | |
|---|---|---|---|
| CATEGORY | BASE (2014) | 6-YEAR (2020) | 20-YEAR (2034) |
| Without Conservation (Table 3.14) | | | |
| Projected ERU and Demands | | | |
| ERU | 468 | 501 | 590 |
| ADD (gpd) | 54,232 | 58,106 | 68,417 |
| MDD (gpd) | 75,925 | 81,348 | 95,784 |
| | | | |
| Existing Source Capacity (gpd) | 216,000 | 216,000 | 216,000 |
| | | | |
| **Source Surplus(+)/Deficiency (-) (gpd)** | **+140,075** | **+134,652** | **+120,216** |

## 4.3 STORAGE CAPACITY

### Existing Effective Storage

The effective storage capacity in the reservoir is the volume available of being withdrawn at the rates and pressures required for water storage purposes. Generally, the effective storage is equal to the total storage minus operational and dead storage.

Table 4.6 summarizes the characteristics of the steel reservoir.

| TABLE 4.6 – RESERVOIR CHARACTERISTICS | |
|---|---|
| Characteristic | Value |
| Nominal Capacity | 250,000 gallons |
| Diameter | 55 ft |
| Unit Volume | 17,772 gal/ft |
| Overflow Elevation | 328 ft |
| Source Call Elevation | 326.5 ft |
| Minimum Operating Elevation | 316 ft |
| Outlet Elevation | 315 ft |
| Base Elevation | 315 ft |
| Effective Storage Depth | 10.5 ft |
| Effective Storage Volume | 186,510≈186,500 |

The Total Storage (TS) is the volume between the base and overflow elevations. This volume is about 230,910 gallons.

$$TS = \pi \times 55^2/4 \times (328\text{ ft} - 315\text{ ft}) \times 7.48\text{ gal/cf} = 230{,}908\text{ gal} \approx 230{,}910\text{ gallons}$$

The Operational Storage (OS) is the volume between the low and high water storage elevations set to control system pumps. This volume is about 26,640 gallons.

$$OS = \pi \times 55^2/4 \times (328\text{ ft} - 326.5\text{ ft}) \times 7.48\text{ gal/cf} = 26{,}643\text{ gal} \approx 26{,}640\text{ gallons}$$

Dead Storage (DS) is the last foot of water in the reservoir because the reservoir should not be drawn down within a foot of the outlet pipe elevation. This volume is about 17,760 gallons.

$$DS = \pi \times 55^2/4 \times (1\text{ ft}) \times 7.48\text{ gal/cf} = 17{,}762\text{ gal} \approx 17{,}760\text{ gallons}$$

The Effective Storage is (ES) Total Storage minus Operational Storage and Dead Storage. This volume is about 185,790 gallons.

$$ES = TS - OS - DS = 230{,}910 - 26{,}640 - 17{,}760 = 186{,}510\text{ gallons.}$$

The system has an additional 24,220 gallons available from the clearwell. The clearwell is a 61,650 gallon (29 ft x 20.33 ft x 14 ft) concrete reservoir beneath the treatment plant that is used as a clearwell to provide adequate contact time. The remaining 37,433 gallons is needed to provide adequate chlorine contact time. If needed, about 24,220 gallons of additional storage is available in the clearwell. This brings the total available storage to 210,730 gallons (=186,510 + 24,220 gallons).

## Equalizing Storage

Equalizing storage is typically used to meet diurnal demands that exceed the average daily and peak day demands. The volume of equalizing storage required depends on peak system demands, the magnitude of diurnal water system demand variations, the source production rate, and the mode of system operation. Sufficient equalizing storage must be provided in combination with available water sources and pumping facilities such that peak system demands can be satisfied.

Equalizing storage is calculated using the following equation from Table 9-1 of the DOH Water System Design Manual:

$$VES = (PHD - QS) \times 150\text{ minutes}$$

Where VES = Equalizing Storage component (gallons)
PHD = Peak Hourly Demand (gpm)
QS = Total Source of Supply Capacity, excluding emergency sources (gpm) = 200 gpm.

Equalizing storage is zero because the peak hour demand is less than the source capacity of 200 gpm.

### Standby Storage

Standby storage is provided to meet demands in case of a system failure such as a power outage, an interruption of supply or a break in the major transmission line. The amount of emergency storage should be based on the reliability of supply and pumping equipment, standby power sources, and the anticipated out of service length of time.

Standby storage is calculated using the following equation from Table 9-1 of the DOH Water System Design Manual:

VBS = 2 days x ADD X N

Where VBS = Total standby storage component (gallons)
ADD = Average daily demand per ERU (gpd/ERU)
N = Number of ERUs.

Table 4.7 lists the standby storage volumes for existing and the two projected planning horizons.

### Fire Suppression Storage

Fire suppression storage is provided to ensure that the volume of water required for firefighting is available. Fire suppression storage also reduces the impact of firefighting on distribution water system. The amount of water required for firefighting purposes is specified in terms of rate of flow in gpm and an associated duration. Fire flows must be provided at a residual water system pressure of at least 20 psi.

Fire suppression storage is calculated using the following equation 9-4 of the DOH Water System Design Manual:

FSS = FF x T
= 750 gpm x 30 minutes = 22,500 gallons

Where FSS = Fire suppression storage
FF = Required fire flow rate (gpm) as specified by local fire protection authority or under WAC 246-293-640 whichever is greater.
T = Duration (minutes)

### Storage Capacity Analysis

Table 4.7 lists the equalizing and standby storage volumes for existing and the two projected planning horizons (6-year, 20-year). The projected demands and ERU values are from Table 3.14. The values with no conservation are used.

| TABLE 4.7 – PROJECTED STORAGE CAPACITY REQUIREMENTS | | | |
|---|---|---|---|
| CAPACITIES | 2014 (gallons) | 2020 (gallons) | 2034 (gallons) |
| EQUALIZING STORAGE | 0 | 0 | 0 |
| STANDBY STORAGE | 108,460 | 116,210 | 136,830 |
| FIRE SUPPRESSION | 22,500 | 22,500 | 22,500 |
| **TOTAL** | **130,960** | **138,710** | **159,330** |
| EFFECTIVE STORAGE | 186,500 | 186,500 | 186,500 |
| **AVAILABILITY/DEFICIT** | **+55,540** | **+47,790** | **+27,170** |

The storage capacity can meet the projected 6-year and 20-year planning horizons. The projected water demands used a loss value of 19.2% as derived from the water balance analysis using 2014 data outlined in Table 3.10. If non-revenue losses can be further reduced, the storage capacity can have increased availability.

## 4.4 DISTRIBUTION SYSTEM ANALYSIS

### 4.4.1 Hydraulic Modeling

As required by DOH, the water system was analyzed using a computer hydraulic model. The distribution system was analyzed and deficiencies were identified for two conditions: peak hour demands (PHD) and maximum day demands (MDD) plus fire flow. All modeling calculations were performed using EPANET.

Hydraulic models require a configuration of the system and assignment of specific system elements such as pipes, nodes and reservoirs. The system was modeled as 65 nodes, 85 pipes and 1 tank. The system has no operable PRV. A schematic map of the system is in Appendix D.

The layout of the water system was recreated in the computer model using an updated system map. This system map was developed by the Utility in 2010 using as-built plans, field investigations, operator lore, and the 2008 WSP. The system map was updated in 2013 to include the water system improvements made by the Utility in 2012.

Chapter 3 presents information on water demands for the existing system and for two planning horizons (2020, 2034). For the model, the demand forecast shown in Table 3.14 under the "without conservation" was used to determine the demand in the service area.

### 4.4.2 Demand Allocation

Demand allocation was determined by the number and type of services at a specific node. The number of customers and type of service customer were assigned at either the nearest or downstream node of the particular water main segment. The spatial distribution of demand was allocated across every node with the exception of nodes that were located on a transmission main, and near the tank. The total number of customers were then totaled and compared to the number of active and inactive service connections. There are about 402 available water service connections as confirmed by the presence of existing service meter boxes.

After the existing demand allocation was conducted, it was used as the basis for the allocation of the two planning horizons: 6-year (2020); and 20-year (2034). Future non-residential demands at specific nodes in the non-residential land use zones were adjusted. Future residential demands were adjusted using a multiplier of 1.1 (=370/341) and 1.3 (=437/341) for 2020 and 2034, respectively. The derivation of the multiplier is based on the projected number of residential ERU with 341, the existing number of residential ERU.

The water demand values shown in Table 3.14 were then used to compute the demand at each junction node. The demand used is the total demand which is the sum of authorized consumption and non-revenue water loss.

### 4.4.3 Model Calibration

The calibration of a hydraulic model provides a measure of assurance that the model is accurate and representative of the actual system. The model was calibrated using field data from fire hydrant tests obtained at various locations in the system. Readings of static pressures, fire flows and residual pressures were taken on June 5, 2014. The system conditions at the time of each test were recorded. The tank water level was full at the time of hydrant testing. Table 4.8 summarizes the test locations and associated node numbers.

| TABLE 4.8 - HYDRANT TEST READINGS | | |
|---|---|---|
| **TEST #** | **NODE #** | **LOCATION** |
| 1A | 59 | 9th/E St |
| 1B | 86 | 9th/G St |
| 2A | 76 | 10th/A St |
| 2B | 75 | 10th/B St |
| 3A | 29 | 6th/Annonen |
| 3B | 25 | 6th/Main |
| 4 | 85 | 8th/I St |
| 5A | 111 | EVD/Spring Ct |
| 5B | 109 | EVCC |

Using the system conditions for each hydrant test, the hydraulic model was used to generate static and residual pressures at the measured hydrant flow rates. The total system demand at the time of the hydrant tests was assumed to be the average day demand for 2013 with a full reservoir. Static pressure readings were compared to model output from this simulation. Residual pressure readings were compared to model output from placing an added demand at the test hydrant locale equal to the field measured hydrant flow rate.

The field results were then compared to the model simulations described above. System pressures and water flow rates are dependent upon the friction loss characteristics for each pipe. These characteristics in the model are set by model parameters such as pipe type, roughness coefficients, pipe lengths and elevations. These parameters were adjusted through an iterative process until the model output approximated the field measured data. The model output was compared with the field measurements for static pressure and residual pressure. The comparison is summarized in Table 4.9.

| | TABLE 4.9 – MODEL CALIBRATION RESULTS | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| TEST # | NODE # | FLOW (gpm) | STATIC PRESSURE (gpm) | | | RESIDUAL PRESSURE (gpm) | | |
| | | | FIELD | MODEL | DIFFERENCE | FIELD | MODEL | DIFFERENCE |
| 1A | 59 | | 54 | 50 | 4 | - | - | - |
| 1B | 86 | | 52 | 49 | 3 | - | - | - |
| 1B, Fire | 86 | 1000 | - | - | - | 40 | 45 | 5 |
| 2A | 76 | | 82 | 78 | 4 | - | - | - |
| 2B | 75 | | 70 | 71 | 1 | - | - | - |
| 2B, Fire | 75 | 1250 | - | - | - | 62 | 66 | 4 |
| 3A | 29 | | 84 | 81 | 3 | | | |
| 3B | 25 | | 70 | 72 | 2 | | | |
| 3B, Fire | 25 | 1200 | - | - | - | 62 | 67 | 5 |
| 4 | 85 | - | 44 | 42 | 2 | - | - | - |
| 5A | 111 | | 50 | 47 | 3 | | | |
| 5B | 109 | | 50 | 46 | 4 | | | |

Hydraulic models are required to be within 5 psi of measured pressure readings for long range planning according to the DOH Design Manual, Table 8-1. Calibration of the model produced results within 4 psi of the field data for static pressure, and within 5 psi of the field data for residual pressure. Detailed analyses of the model input and calibration simulations are in Appendix D.

### 4.4.4 Model Scenarios

After calibration of the model, hydraulic analyses were made for six scenarios. The scenarios are listed in Table 4.10.

| TABLE 4.10 – MODELING SCENARIOS | | |
|---|---|---|
| DESCRIPTION | DEMAND | PURPOSE |
| Existing, Peak Hour | 2014 PHD | Evaluate system |
| Existing, Fire Flow | 2014 MDD plus fire flow | Evaluate system |
| Plan Year 6 (2020), Peak Hour | 2020 PHD | Evaluate system performance and develop CIP for Plan Year 6 peak hour conditions |
| Plan Year 6 (2020), Fire Flow | 2020 MDD plus fire flow | Evaluate system performance and develop CIP for Plan Year 6 fire flow conditions |
| Plan Year 20 (2034), Peak Hour | 2034 PHD | Evaluate system performance and develop CIP for Plan Year 20 peak hour conditions |
| Plan Year 20 (2034), Fire Flow | 2034 MDD plus fire flow | Evaluate system performance and develop CIP for Plan Year 20 fire flow conditions |

### 4.4.5 Peak Hour Demand Results

In accordance with WAC 246-290-230, a minimum pressure of 30 psi must be maintained at all customer connections under PHD conditions. The system was modeled under existing, 2020 and

2034 peak hour demand conditions. The pressures from these scenarios are in Appendix D. The system is capable of meeting the minimum pressure requirements.

### 4.4.6 Fire Flow Analysis Results

A minimum of 20 psi must be maintained for fire flows under MDD conditions. Minimum fire flows were obtained from WAC 246-293-640. Although the existing customer base is primarily residential, the City of Vader has some lands designated as commercial so fire flows of 750 gpm were used. Table 4.11 shows fire flows at all of the hydrant locations in the system. The system is able to meet fire flows for the 6-year and 20-year planning horizons.

To meet higher fire flows in the EVCC area, the small 2-inch and 4-inch mains must be replaced with larger piping.

**TABLE 4.11 – AVAILABLE FIRE FLOW**

| NODE # | HYDRANT LOCATION | FIRE FLOW GOAL (gpm) | AVAILABLE FIRE FLOW (gpm) | | |
|---|---|---|---|---|---|
| | | | 2014 | 2020 | 2034 |
| 4 | 6th/G St | 750 | 2600 | 2600 | 2600 |
| 12 | 6th/D St | 750 | 2200 | 2200 | 2200 |
| 19 | 6th/B St | 750 | 2200 | 2200 | 2200 |
| 22 | 5th/A St | 750 | 2200 | 2200 | 2200 |
| 25 | 6th/Main | 750 | 2200 | 2200 | 2200 |
| 29 | 6th/Annonen | 750 | 2200 | 2200 | 2200 |
| 30 | 7th/Annonen | 750 | 2200 | 2200 | 2200 |
| 32 | SR506/Winlock Vader Rd | 750 | 1000 | 1000 | 1000 |
| 35 | 7th/A St | 750 | 2200 | 2200 | 2200 |
| 37 | 7th/B St | 750 | 2200 | 2200 | 2200 |
| 39 | 7th/E St | 750 | 2200 | 2200 | 2200 |
| 43 | 8th/E Alley | 750 | 2100 | 2100 | 2100 |
| 45 | 8th/C St | 750 | 2100 | 2100 | 2100 |
| 50 | 9th/A Alley | 750 | 2100 | 2100 | 2100 |
| 54 | 9th/B St | 750 | 2100 | 2100 | 2100 |
| 59 | 9th/E St | 750 | 2100 | 2100 | 2100 |
| 68 | 9th/G St | 750 | 2000 | 2000 | 2000 |
| 69 | 10th/F St | 750 | 2100 | 2100 | 2100 |
| 70 | 10th/E St | 750 | 2100 | 2100 | 2100 |
| 72 | 10th/D St | 750 | 2100 | 2100 | 2100 |
| 75 | 10th/B St | 750 | 2100 | 2100 | 2100 |
| 76 | 10th/A St | 750 | 2100 | 2100 | 2100 |
| 85 | 8th/I St | 750 | 1800 | 1800 | 1800 |
| 86 | 9th/G St | 750 | 1900 | 1900 | 1900 |
| 109 | Enchanted Valley Country Club (EVCC) | 750 | 750 | 750 | 750 |
| 111 | EVD N/Spring Ct | 750 | 1000 | 1000 | 1000 |

## 4.5 SUMMARY OF SYSTEM CAPACITIES

The capacity of the system is defined by the limiting capacities of several system elements. These elements are summarized in the Table 4.12.

| TABLE 4.12 – WATER FACILITY CAPACITIES | |
|---|---|
| FACILITY | CAPACITY |
| Source @ 18 hour Pump Rate | 216,000 gpd |
| Source @ 24 hour Pump Rate | 288,000 gpd |
| Water Rights, Qi | 224 gpm |
| Intake Pumping Capacity | 200 gpm |
| Treatment Plant | 200 gpm |
| Storage from Tank and Clearwell | 210,730 gallons (=186,510 tank + 24,220 clearwell) |

A comparison was made between the facility capacities and the forecasted water demands provided in Table 3.14. For brevity, the forecasted water demands with no water conservation are provided in Table 4.13 since conservation measures would reduce projected water demands.

| TABLE 4.13 – WATER DEMAND WITH NO CONSERVATION | | | |
|---|---|---|---|
| SCENARIO | WATER DEMAND WITH NO CONSERVATION | | |
| | ADD | MDD | PHD |
| BASE (2014) | 54,232 gpd | 75,925 gpd | 127 gpm |
| 6-YEAR (2020) | 58,106 gpd | 81,348 gpd | 134 gpm |
| 20-YEAR (2034) | 68,417 gpd | 95,784 gpd | 150 gpm |

The analysis shows that the system has the facility capacity to meet projected demands. This adequacy is based on the following assumptions:

- Continuation of the water loss rate as outlined in Table 3.11 and used in forecasted demands in Table 3.14.
- No significant change in the number and usage habits of residential customers
- No expanded change in water usage from primarily residential to industrial and commercial.
- No change in the ERU factor of 116 gpd/ERU.

Although the system can adequately meet future demands, we consider the high water loss rate as unacceptable and unsustainable for the utility. A water use efficiency program is outlined in Chapter 5 to further reduce non-revenue water loss.

Water main improvement projects are outlined in Chapter 9 and were analyzed with the hydraulic model. A discussion of the priority assessment and of the utility's philosophy of individual capital improvement projects is provided in Chapter 9.