# Wastewater Collection System Master Plan Update

## FINAL

Prepared for:

**City of Klamath Falls**

**SHN Consulting Engineers & Geologists, Inc.**

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Klamath Falls, OR 97601
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December 2014
613011.500

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Reference: 613011.500

December 19, 2014

Mr. Mark Willrett, PE, Public Works Director
City of Klamath Falls
226 S. 5th Street
Klamath Falls, OR 97601

**Subject: Wastewater Collection System Master Plan Update (Final)**

Dear Mr. Willrett:

Please find enclosed the final version of the Wastewater Collection System Master Plan. You are receiving four hard copies and one electronic PDF version on CD. Let us know if you have any questions. Additional supporting material is available upon request.

Sincerely,

**SHN Consulting Engineers & Geologists, Inc.**

Anders Rasmussen

Anders H. Rasmussen, PE
Senior Civil Engineer

AHR: dkl

Enclosures: Final Wastewater Collection System Master Plan Update

Reference: 613011.500

# Wastewater Collection System Master Plan Update

## FINAL

Prepared for:

**City of Klamath Falls**
Klamath Falls, OR

REGISTERED PROFESSIONAL ENGINEER
62601
Anders H. Rasmussen
OREGON
SEPT. 14, 1999
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EXPIRATION DATE: 12/31/14

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December 2014

QA/QC: RFS

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# Abbreviations and Acronyms

| | |
|---|---|
| ADWF | Average dry weather flow |
| AWWF | Average wet weather flow |
| CII | Commercial/industrial/institutional |
| CIPP | Cured-In-Place Pipe |
| GPM | Gallons per minute |
| MGD | Million gallons per day |
| MH | Manhole |
| PWWF | Peak wet weather flow |
| SSSTP | Spring Street Sewage Treatment Plant |
| WWCSMP | Wastewater collection system master plan |
| I/I | Inflow and infiltration |
| RDII | Rainfall derived inflow and infiltration |
| WWTP | Wastewater treatment plant |

# Executive Summary

SHN Consulting Engineers & Geologists, Inc. ("SHN") completed an updated Wastewater Collection System Master Plan with an emphasis on updating the existing wastewater collection system model. Condition assessment was not included as part of the scope.

SHN performed a review of the existing model created for the previous Master Plan completed in 2006. A number of deficiencies were found with appropriate changes made to the model, including pipes with adverse slopes, high peaking factors, flow patterns, and inaccurate pump station performance data. Results of the updated model were found to compare generally well to observed conditions in key locations for dry weather conditions, although the model results may be slightly conservative.

Based on the modeling results, in conjunction with field observations and discussion with City of Klamath Falls ("City") staff, the following recommendations are made:

1. Verify inverts of various pipes that have inverts suspected of being incorrect.
2. TV the pipe segments along the Stewart/Lenox Interceptor with high Mannings *n* value to verify condition.
3. Monitor high water elevations in key areas to verify whether surcharging is occurring during dry weather flows.
    a. Highest priority location is along Veterans Park and Klamath Avenue.
    b. Other locations as described in Chapter 5 with equal priority for other segments.
4. Install flow meters on the discharge of the Hanks and Pearl pump stations to verify pumping capacities.
5. Upsize 640 LF of pipe along Division Street between MHs 03-013 and 03-023 to 12 inches. Estimated construction cost: $90,000 ($10/in/ft plus 15% for design and inspection). Verify flows and whether surcharging is occurring during dry weather conditions.
6. Develop a flow monitoring plan to gather flow data at key locations throughout the collection system to refine the inflow values.

Chapter 1

# Introduction

The City provides wastewater collection and treatment to a service area generally contiguous with current city limits. It also receives wastewater from two residential communities located outside city limits. The previous Wastewater Collection System Master Plan was completed in 2006 by Brown & Caldwell.

The City contracted with SHN to prepare an update to the 2006 Wastewater Collection System Master Plan with a focus on updating the existing computer model of the collection system.

Chapter 2

# Basis of Planning

## Service Area Description

Klamath Falls is located in a semi-arid region of south-central Oregon at an elevation of approximately 4100 ft above sea level. The wastewater collection system in the City of Klamath Falls serves a population of approximately 20,000 along with commercial, industrial, and institutional users generally located within the city limits. Some limited areas within City limits are served by the South Suburban Sanitary District. The City also receives wastewater from the Running Y Resort and the Falcon Heights community, both of which are located outside city limits, and are served through mutual agreements.

The wastewater collection system consists of approximately 150 miles of varying sized sewer pipe and 12 pump stations. The service area is shown in Figure 1.

## Current Planning Effort

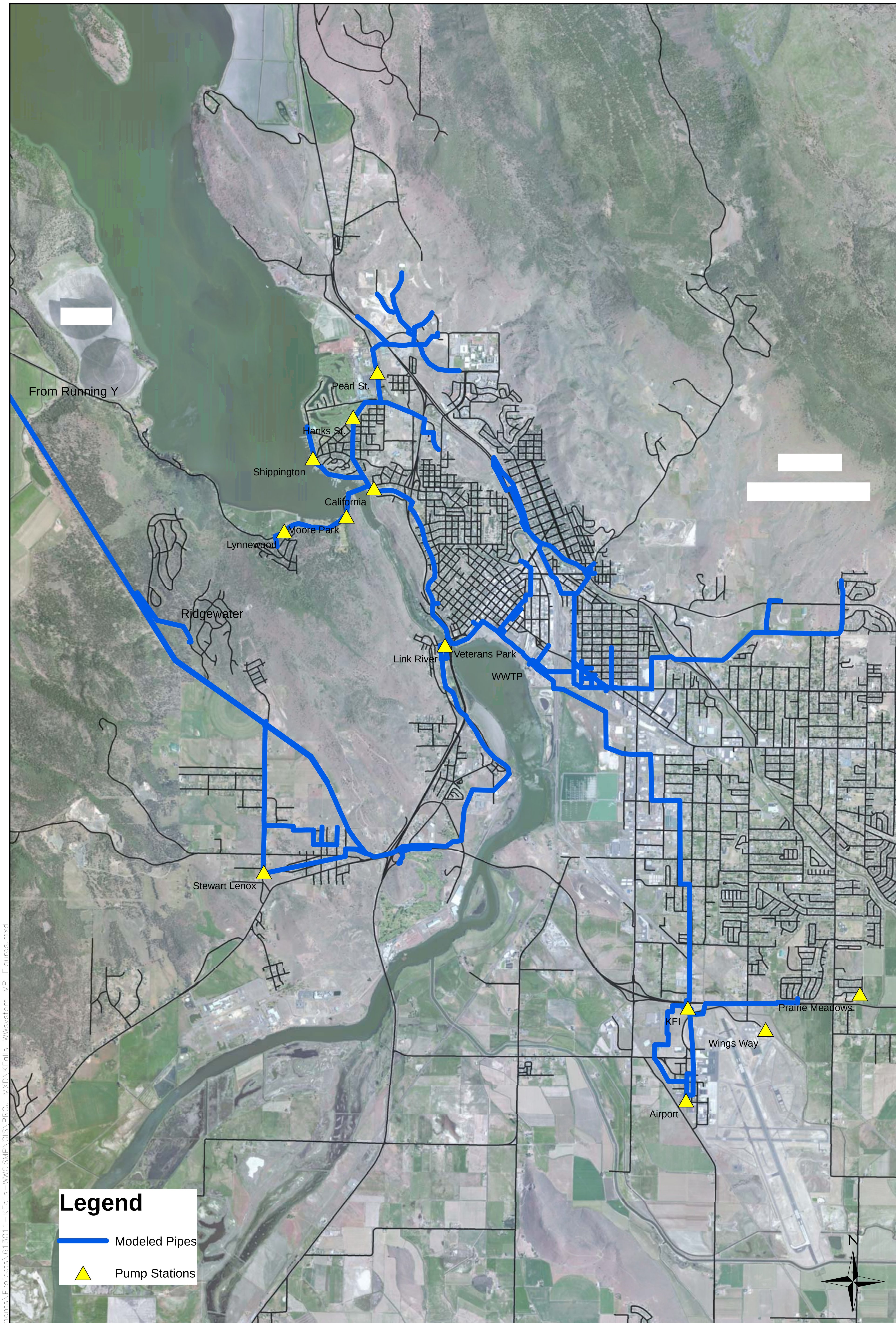
This Wastewater Collection System Master Plan (WWCSMP) builds on the efforts of the previous WWCSMP. The City has maintained the model developed for the 2006 WWCSMP, and this model was used as the basis for the current effort.

The previous WWCSMP appeared to show some deficiencies in the collection system that did not match with observed conditions. For this reason, the City wanted to focus on reviewing and updating the model, as described in the following chapters.

Due to the economic downturn experienced in recent years, growth has been rather flat in the service area and region at large. As such, it is currently difficult to define a future condition within the near-term planning horizon (5-10 years) that is significantly different than current conditions. In discussion with City staff, it was decided that future conditions would be limited to certain areas of the City with growth potential and limited to estimating the available additional capacity of the existing system serving these growth-potential areas.

## Scope of Work

The scope of work for this project focused on improving the modeling. Pipe condition assessment was not part of this project.




SHN Consulting Engineers & Geologists, Inc.

City of Klamath Falls
Wastewater Collection System
Klamath Falls, Oregon

Future Growth Potential Areas
SHN 613011

November 2014 | KFalls_WWsystem_MP_Figures | Figure 1

# Review of Existing Model

## Overview

SHN reviewed the existing model, which uses the H2OMap Sewer software by Innovyze. In discussion with City staff regarding the existing model, the main concerns were errors with the pipe network and assumptions for peak flow, especially peaking factors. This section discusses findings with the existing model. Updates to the model are described in Chapter 4.

## Updates Since the 2006 Master Plan

After the initial development of the model for the 2006 Master Plan, an update to flows was completed in 2008 by Brown and Caldwell to reflect a significantly increased development picture within the service area. The model was subsequently maintained by Harrison Engineering and the following updates were made to the model:

- Ridgewater and Running Y lift stations with associated force mains were added to the model. Previously, flows from these two areas were added as a gravity input downstream of the force mains.
- The pipe networks in the Stewart-Lenox, airport, Lakeport Boulevard and OIT areas were extended. In general, flows are still input downstream of these extended network areas.
- The Cogeneration Power Plant lift station and force main that send non-blowdown process water and sanitary flows to the collection system were added.

## Pipe Network

The pipe network was reviewed, and eight pipe segments were found to have adverse slopes. In general, the pipe inverts matched survey data, indicating that there may have been some errors with the survey. In addition, a few pipes were found to have incorrect diameters.

In general, the model appeared to include all of the major interceptors and trunk lines (i.e. 12 inches and greater). The modeled system represents approximately 46 miles of pipes, which is 31% of the entire collection system (150 miles).

## Pump Stations

Pump stations in the model were reviewed. Pump capacities in the model were 80% of the rated capacities. No flow testing had been performed to verify the flows. For the City-owned pump stations, all physical pumps, including redundant ones, were included in the model. Pump start and stop settings were reviewed against current settings, and some minor discrepancies were found, possibly due to changes in operation since model development.

## Dry Weather Flows

Dry weather flows used in the model were determined based on limited flow monitoring conducted in 2005. A cursory review of the inflows revealed the values and inflow locations to be generally reasonable throughout the model network with only a small number of exceptions. Further, the average modeled flow entering the Spring Street Sewage Treatment Plant was found to be about 4 MGD. While this is higher than current average dry weather flow, which is about 2.75 MGD, the 4 MGD figure may be a somewhat conservative but reasonable estimate for full occupancy of existing facilities (housing, commercial, industrial, and institutional facilities, etc.).

## Peaking Factors

The capacity of a wastewater collection system is based on its ability to handle peak flows. Oregon regulations prohibit sanitary sewer overflows except during a storm event greater than the one-in-five-year, 24-hour duration storm in winter (November 1 through May 21) or the one-in-ten-year, 24-hour duration storm in summer (May 22 through October 31), whichever is greater (OAR 340-41-0009). In Klamath Falls, the 5-year, 24-hour rainfall is 2.0 inches while the 10-year, 24-hour rainfall is 2.1 inches (NOAA Isopluvial Precipitation Maps). It was determined in the 2006 Master Plan that the 5-year event during winter produces the greater flow.

The peaking factor is comprised of several components, including diurnal pattern, the ratio of average wet weather flow (AWWF) to average dry weather flow (ADWF), and rainfall-derived infiltration/inflow (RDII). These are generally developed based on flow monitoring. Since it is often not feasible to measure flows in all input nodes, these factors are developed at the flow monitoring locations and then applied at similar inflow nodes within the model.

In the 2006 Master Plan, the stated overall peaking factor for peak wet weather flows (PWWF) during a 5-year, 24-hour storm event was six times the average dry weather flow. City Staff expressed concern that this peaking factor was higher than is actually experienced within the collection system. The value of six is based on a factor of two for the AWWF/ADWF ratio and a factor of three for the RDII factor (2x3=6). Refer to the following sections for additional explanation.

Upon further review of the modeling details, it was discovered that the peaking factor of six did not include the peaks associated with the diurnal pattern. The effective peaking factor at each input node varied from 6.0 to 7.5 times the ADWF, depending on which diurnal pattern was applied at that node. The average effective peaking factor in the model was 6.46 times the ADWF. By contrast, the City's standard for design calls for a peaking factor of 3.0 for residential and 3.5 for commercial/industrial/institutional flows.

### Diurnal Flow Patterns

In the model, an hourly flow pattern is superimposed onto the average daily flow at each input node. The patterns in the previous model were determined based on observed patterns from the flow monitoring. The shape of the patterns generally appeared reasonable, but the peak within the diurnal pattern varied from 1.155 to 1.75 times the average input flow at individual nodes. Figure

2a shows the pattern developed from the North Hills area (MH 15-002), which is a typical pattern for a residential area, showing peak flows in early morning and early evening. Figure 2b shows the pattern developed for the area around Sky Lakes Medical Center (MH 6-64), which is typical for a commercial/industrial/institutional area, showing a mid- or late morning peak with sustained higher flows throughout the day.

Some flow patterns applied at inflow nodes were based on monitoring data from manholes downstream of pump stations. The pump station influence adversely affects the type of flow pattern that would be observed above the pump station within the gravity portion of the system. Figure 2c shows a pattern developed from flow monitoring in a manhole along Riverside Drive (MH 11-77), which conveys flows predominantly from pump stations (Stewart-Lenox, Running Y, Ridgewater, and Cogen). This pump-station-influenced pattern was applied to some of the inflow nodes upstream of these pump stations, where one would expect a more typical residential pattern as shown in Figure 2a.

## AWWF/ADWF Ratio

SHN reviewed the ratio of average wet weather flow (AWWF) to average dry weather flow (ADWF). This ratio is important to determining the overall peaking factor. It is generally understood that the difference between the AWWF and ADWF is caused by inflow and infiltration (I/I) due to wintertime factors such as high ground water and storm water infiltration through leaky joints. The 2006 Master Plan stated that treatment plant inflow from 2004-2005, a dry winter, indicated a ratio of average wet weather to average dry weather of 1.2 and further stated that a ratio of two was conservatively assumed for the modeling.

## RDII Peaking Factor

During a rainfall event and subsequent runoff period, there is usually a noticeable corresponding increase in flow in the collection system. This is termed rainfall-derived inflow and infiltration (RDII). The increase in flow is due to inflow and infiltration directly related to runoff from the particular storm event. The design event used when estimating peaking factors is the five-year, 24-hour event, which is determined from the NOAA Isopluvial Precipitation maps as being 2.0 inches.

Determining the short term flow response within the collection system due to RDII requires flow monitoring data from locations with only gravity flows (i.e. no upstream pump stations) and correlated with precipitation data on an hourly basis. Available flow data have been very limited, and there have not been any precipitation events in the City that exceed the two-year, 24-hour event within the last six years. Further, most storms produce rather scattered rainfall patterns within the City.

For the 2006 Master Plan, a peaking factor of three was assumed for the flow response within the system. This is based on a factor of two for the peak flow during a 5-year, 24-hour storm event experienced at the treatment plant with an additional 1.5 factor within the collection system to remove the attenuation that would be observed at the WWTP (2x1.5=3).

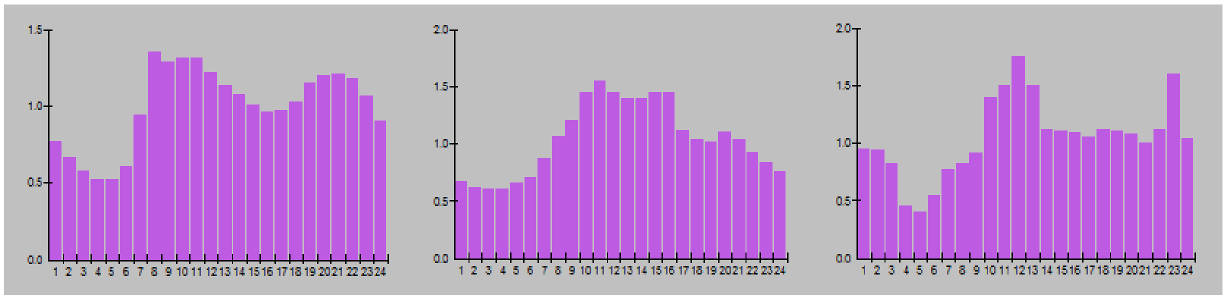



***Figure 2. Patterns used in the previous modeling effort, showing (a) a typical residential pattern, (b) a typical commercial/industrial/institutional pattern, and (c) a pattern highly influenced by upstream pumped flows.***

# Model Update

## Overview

In general, SHN focused the study update on the main concerns expressed by City staff related to the existing model and on other issued we identified that can influence the model results. Staff concerns were related errors with the pipe network and the flow peaking factor. The other issues include input flows and pump station settings. The software used for the modeling was H2OMap Sewer Pro Suite version 10.5 by Innovyze, and allows for multiple scenarios and multiple data sets. Extraneous model scenarios and data sets were deleted from the updated model.

## Pipe Network

For the eight pipes with adverse slope in the model, the inverts were modified to represent positive drainage slope. Since the modifications were small, there was no significant impact observed with these changes. However, the inverts should be field verified.

In addition, we fixed the pipes with incorrect diameters in the model. The correct diameters were taken from the GIS database provided by the City. The recent upgrade in the force main from the California Pump Station was added by changing the diameter of the force main in the model from 10 to 16 inches. While the 10-inch force main remains in operation it is not part of the model as it is used only during nighttime low flows.

## Pump Stations

Pump capacities used in the model should reflect actual conditions. The previous model had assumed the pumps operate at 80% of the rated design capacity, but there had been no verification of the actual pumping rates occurring at existing pump stations. An analysis was performed to estimate actual pumping rates using information from the SCADA data for the City's pump stations and data collected from observed drawdown tests. Table 1 shows the results of the analysis. Pump rates for the California and KFI pump stations were the maximum reading from the flow meters during a pumping cycle. Additional details of the analysis are provided in the Appendix.

Some of the measured pumping rates vary significantly from what was previously used in the model. In particular, there is significant discrepancy with the Pearl and Hanks pump stations. Normally, a downstream pump station (Hanks) should have a higher capacity than the upstream pump station (Pearl), especially when the upstream pump station serves only a subset of the service area of downstream pump station; in this case, the field data show otherwise. Note the recommendation for flow metering later in this chapter.

Pump on/off set points were revised as needed to reflect current set points in the system. In some cases, set points may have been revised because of operational changes since the last model update.

For two of the non-City pump stations in the model, Running Y and Ridgewater, the pump capacities were left at 80% of the design capacity. For the Cogen pump station, measured flow data by minute were provided by Iberdrola Renewables, the owner and operator of the Cogeneration Power Plant (Cogen). Based on this data, it was decided to set the Cogen inflows as direct inputs into the Stewart Lenox Interceptor rather than route it through the Cogen pump station. This is discussed further in the following section.

**Table 1**
**Pump Capacities At Pump Stations As Used In The Model**

| Pump Station[1] | Pumping rate used in previous model[2] (gpm) | Revised pumping rate used for updated model[3] (gpm) | Comments |
|---|---|---|---|
| California | 2240 | 2270 | The third low flow pump is not included in the model |
| Hanks | 760 | 479 | |
| KFI | 1000 | 1956 | |
| Link River | 840 | 1157 | |
| Lynnewood | 180 | 332 | |
| Moore Park | 320 | 563 | |
| Pearl Street | 280 | 585 | |
| Ridgewater[4] | 150 | 150 | Not a City pump station |
| Running Y[4] | 465 | 465 | Not a City pump station |
| Shippington | 280 | 374 | |
| Stewart/Lenox | 456 | 610 | |

1. Smaller City-owned pump stations not listed here are not currently included in the model.
2. The number represents 80% of the rated capacity of each pump, except as otherwise noted.
3. These values are based on SCADA or physical drawdown tests except for the non-City pumps; California and KFI based on maximum instantaneous flow from flow meter readings.
4. The Ridgewater and Running Y pump stations were added to the model after the completion of the previous master plan. No drawdown tests were performed. The values shown represent the full capacity of each pump at each pump station.

# Dry Weather Flows

The inflow values appeared generally reasonable; therefore, no changes were made except for the following:

- MH 11-77, along Riverside Drive, on the Stewart/Lenox Interceptor. The inflow value was changed to reflect existing conditions of inflows along Greensprings Drive, which are very minor. The residential areas using septic systems outside the current City limits in the Greensprings area will eventually connect to the interceptor; however, it is not likely to occur within a reasonable planning horizon with any degree of certainty. Therefore, only the existing connections (approximately 122 connections) were assumed to contribute flow, and this flow (0.05 cfs ADWF) was placed further upstream at MH 11-136.
- MH 10-012, located on Conger Drive near the intersection with Main Street, on the line from the California Pump Station. The previous model had a significant inflow that was much

higher than the actual few direct connections to the gravity portion of the line along California Avenue south of Doty Street. Inflows from the area east of California Avenue are already accounted for in a connector pipe at MH 10-028 that connects to the California line at N 1st Street. The model was modified so that no inflow is entering the system at MH 10-012.

- MHs 751, 17-006, and 7-30, all upstream of the Stewart/Lenox Pump Station. The inflow numbers in the previous model are much higher than would account for the current number of ERUs for the Stewart/Lenox and Southview areas. The revised inflow values are 0.039 cfs for MH 751 (Southview, 94 ERUs), 0.031 cfs for MH 17-006 (along Orindale Road, 76 ERUs), and 0.090 cfs for MH 7 -30 (just upstream of the pump station, receiving flow from the southern portion of Stewart/Lenox, 220 ERUs).
- Non-blowdown process wastewater flows, including a small amount of sanitary flows, from the Klamath Cogeneration Power Plant ("Cogen"). Based on data provided by Cogen staff, it was found that the flows from Cogen could occur at any time of the day and could last up to approximately 20 minutes. For the purposes of the modeling, it was assumed that the Cogen flow coincides with other peak flows as a worst case scenario. Instead of routing the flows through the Cogen pump station, it was determined that a better reflection of the potential worst case would be to input the Cogen flows directly into the Stewart/Lenox interceptor at MH 11-139 for a duration long enough to ensure it would coincide with other peak flows from the Stewart/Lenox area.

# Peaking Factors

As aforementioned, the overall peaking factor applied to average dry weather flows is comprised of several components, including the following:

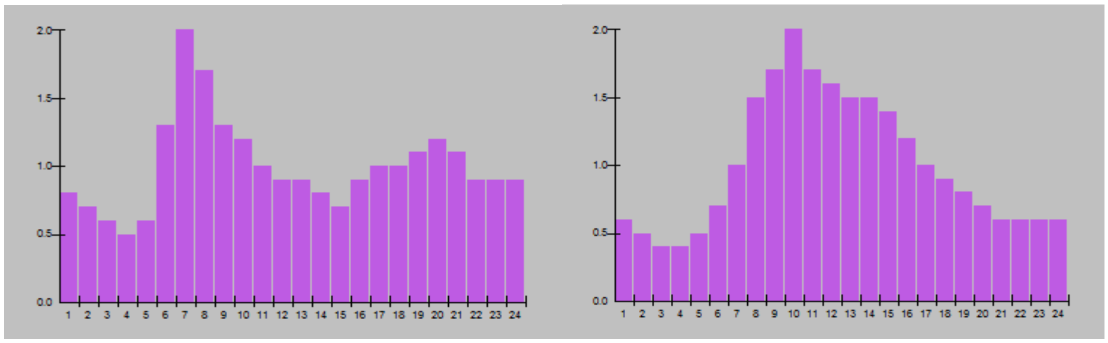
- Diurnal pattern
- Ratio of average wet weather flow to average dry weather flow
- Rainfall-derived infiltration/inflow (RDII).

The following sections describe how the overall peaking factor was developed for this update.

## Diurnal Flow Patterns

Available flow data in locations not influenced by significant pump station flows were reviewed to determine a typical diurnal peak. It was determined that, on an hourly basis, a peaking factor of two would be reasonable for both residential and commercial/industrial/institutional (CII) areas to account for the peak daily variation in flow under average dry weather conditions.

Generic diurnal flow patterns were created for both residential and CII areas. These are shown in Figure 3. These patterns were applied to the input nodes with the residential pattern applied to primarily residential areas and the CII pattern to primarily CII areas.




(a) (b)

***Figure 3. Generalized patterns used in the current model, showing (a) a typical residential pattern and (b) a typical commercial/industrial/institutional pattern.***

## AWWF/ADWF Ratio

Analysis of daily treatment plant inflow data from 2007-2012 indicated that the AWWF/ADWF ratio is as high as 1.40, which occurred during a wet year (winter of 2010-11). Table 2 summarizes the results of the analysis. Two approaches to calculating the ratio were used: (1) By calendar year (Jan-Dec) and (2) by water year (Oct-Sept).

It should be noted that the City is aware of at least some minor illicit flows from private geothermal heating systems into the collection system. At this point, these are assumed to be insignificant flows.

## RDII Peaking Factor

A typical methodology to determine peak flow in response to rain events is outlined in the Oregon DEQ document *Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon*.

The first step is to determine how flows are affected by precipitation on a monthly basis during the months of January through May. The monthly inflow to the WWTP was plotted against the monthly cumulative rainfall for the years 2008-2012 (Figure 4). Little, if any, correlation was found between inflows at the treatment plant and precipitation events. While much of the winter precipitation falls as snow, temperatures are usually high enough to melt most of the snow within a short period (often within one week). This would mean that, even during the coldest month of winter, if the collection system has significant direct response to precipitation, there should be a correlation at least on a monthly basis. However, as stated earlier, little correlation was found between monthly precipitation and inflow at the WWTP.

| Table 2<br>Summary Of Treatment Plant Inflows And Determination Of AWWF/ADWF Ratios | | | | | | |
|---|---|---|---|---|---|---|
| | **Average Monthly WWTP Inflow by Year, mgd** | | | | | |
| **Month** | **2007** | **2008** | **2009** | **2010** | **2011** | **2012** |
| Jan | | 3.55 | 3.73 | 3.15 | 4.30 | 3.03 |
| Feb | | 4.84 | 3.18 | 3.15 | 3.64 | 3.05 |
| Mar | | 5.01 | 3.75 | 2.77 | 4.87 | 3.41 |
| Apr | | 3.47 | 3.20 | 3.17 | 3.99 | 3.49 |
| May | | 3.52 | 3.38 | 3.07 | 3.47 | 3.10 |
| Jun | | 3.46 | 3.40 | 3.08 | 3.17 | 3.07 |
| Jul | | 3.16 | 2.82 | 2.76 | 3.05 | 2.65 |
| Aug | | 3.11 | 2.86 | 2.83 | 2.82 | 2.50 |
| Sept | | 3.05 | 2.92 | 2.83 | 2.71 | 2.58 |
| Oct | 3.26 | 3.12 | 3.02 | 3.01 | 2.27 | 2.63 |
| Nov | 3.10 | 3.16 | 2.84 | 3.19 | 2.29 | 2.75 |
| Dec | 3.19 | 3.02 | 2.80 | 4.53 | 2.28 | 3.14 |
| **Method 1: By calendar year (Jan-Dec)** | | | | | | |
| ADWF | | 3.24 | 3.07 | 2.93 | 2.92 | 2.75 |
| AWWF | | 3.84 | 3.25 | 3.33 | 3.56 | 3.14 |
| **AWWF/ADWF ratio** | | **1.19** | **1.06** | **1.14** | **1.22** | **1.14** |
| **Method 2: By water year (Oct-Sept) (Note that ADWF is the same for both methods)** | | | | | | |
| AWWF | | 3.86 | 3.34 | 2.98 | 4.09 | 2.92 |
| **AWWF/ADWF ratio** | | **1.19** | **1.09** | **1.02** | **1.40** | **1.06** |
| Note: Dry weather is May-October. Wet weather is January-April and November-December. | | | | | | |

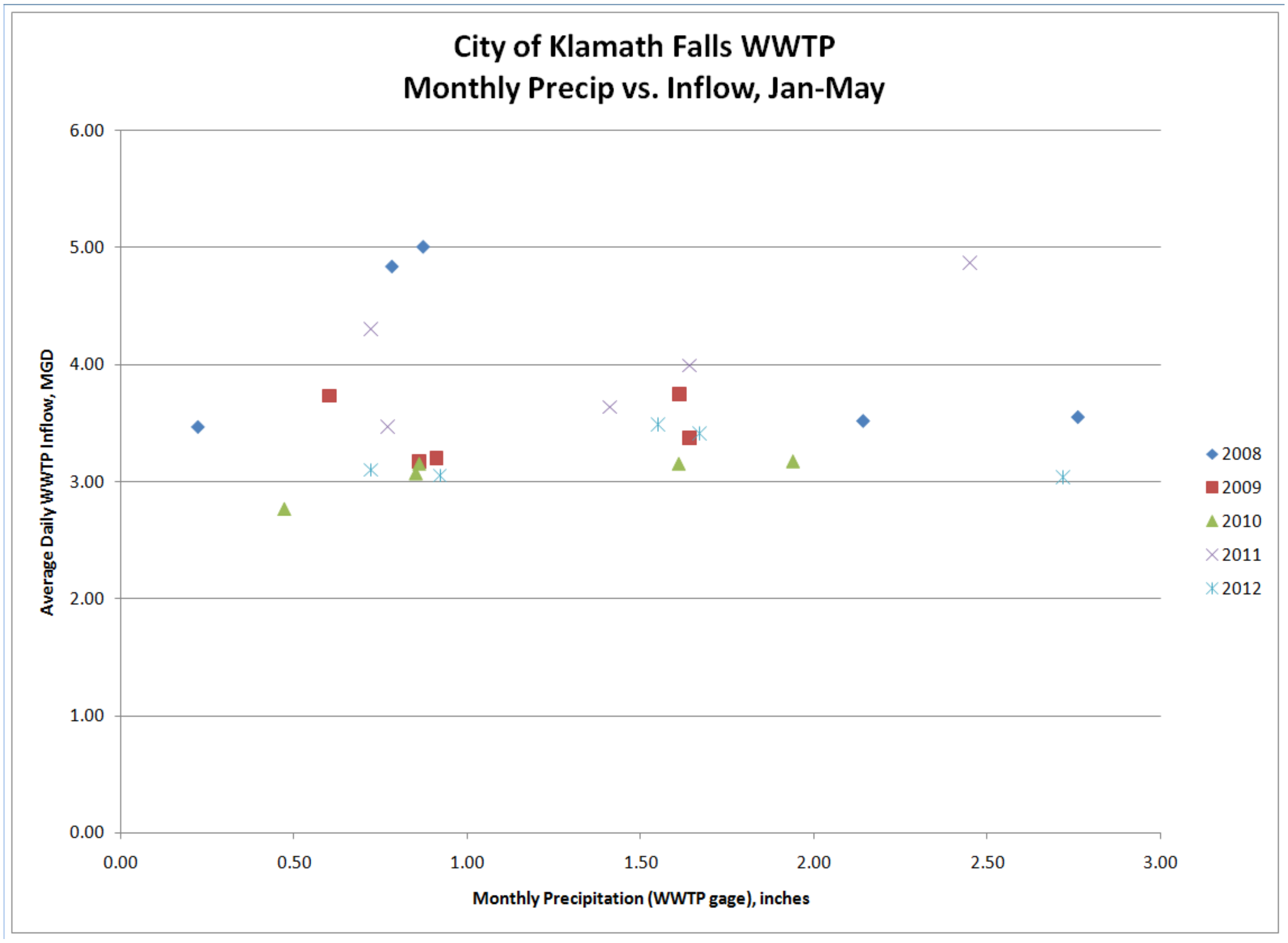



*Figure 4. Plot of total monthly precipitation for the months of January through May for the years 2008-2012 versus average daily inflow at the WWTP for the same months.*

This corroborates what was found for the AWWF/ADWF ratio discussed in the previous section. This may not indicate that the City's system is relatively tight with little I/I since much of the collection system is old. This may, however, indicate that groundwater levels are low and that flows from rainfall and snowmelt effectively have other flow pathways than into the wastewater collection system.

Available flow data were reviewed to see if there were any observable flow increases after significant rainfalls greater than 0.5 inches in a 24-hour period. Any increases in flows in response to the rainfall were very minor and generally difficult to observe in the flow data.

Based on this analysis, it was concluded that RDII is not significant in the collection system. However, it should not be neglected in the modeling in order to have an adequate factor of safety. The following section discusses the overall peaking factor used in the model.

## Overall Peaking Factor

Based on the analysis described above, the following conclusions were reached:

- The AWWF/ADWF is at most 1.40, indicating that overall I/I (RDII plus groundwater) is moderate at most.
- The magnitude of response to flow after a rainfall event was difficult to determine from the flow data and appeared to be small.

Nevertheless, a reasonable allocation for wet weather flows and RDII needs to be added in the model as a factor of safety. A factor of 1.0xADWF to account for wet weather flows and RDII peaks was assumed for the model. This means that the overall peaking factor for the peak wet weather flow (PWWF) scenario used in the model, accounting for diurnal peaks, wet weather, and RDII, is 2+1=3.0xADWF. This compares well with the City's design standard, which calls for using an overall peaking factor of 3.0 for residential flows and 3.5 for CII flows.

It is important to note that the PWWF scenario assumes that the peak RDII flows occur at the same time as the peak of the diurnal flows. Precipitation can occur at any time of the day, which means that any RDII flows could be in the collection system during any time of the day, including during low flows. For the purposes of modeling, a conservative approach is assumed in that the RDII and diurnal flow peaks occur simultaneously.

# Recommendations

## Pumping Rates

We recommend the City place flow meters on the discharge of the Hanks and Pearl pump stations to verify pumping rates, given that these pump stations serve areas of potential growth. Based on the results of the flow metering data, it may be necessary to consider replacement of the pumps at these pump stations.

## Peaking Factor

In order to improve the modeling and more accurately reflect peaking factors and flow patterns, additional flow data should be acquired in strategic locations throughout the collection system prior to the next update of the Wastewater Collection System Master Plan. Based on limited field observations, it appears that modeling results (see Chapter 5) using the peaking factors discussed above are somewhat conservative, but not overly conservative.

# Hydraulic Analysis

## Existing Conditions

The updated H20Map Sewer model of the Klamath Falls wastewater collection system was run for two existing conditions: (1) average dry weather flow (ADWF) and (2) peak wet weather flow (PWWF).

### Average Dry Weather Flow Scenario

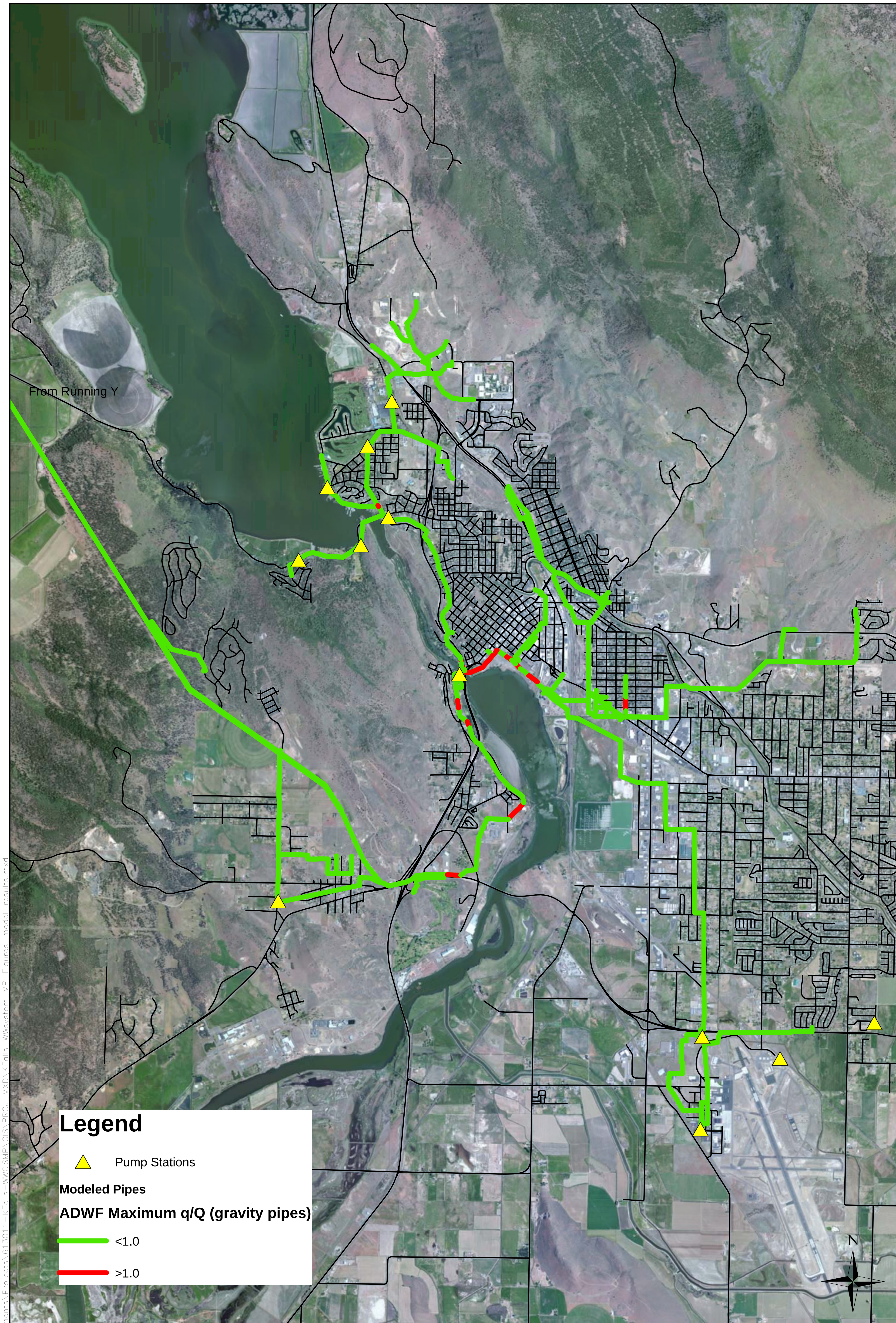
In general, the model results indicate that the existing system has sufficient capacity for dry weather conditions with minor surcharge in some areas under certain conditions, such as along the Stewart/Lenox interceptor when there is coincident flow from all upstream pump stations or along Veterans Park with coincident flow from the California and Link River pump stations. In all cases, the hydraulic grade line remained greater than three feet below manhole rim elevations. Field observations of some key areas indicate that the model results may be more conservative than actual conditions as described in the following paragraphs. As such, there are no recommended actions to modify pipe sizes due to existing dry weather conditions, with the exception of two pipes along Division Street (see following paragraphs) unless subsequent field measurements indicate otherwise. It is recommended that a level indicator be used in various manholes to determine the peak water elevations during typical dry weather flows. Table 3 lists the pipes with a maximum ratio of flow to capacity (q/Q) of greater than 1.0. Figure 5 shows the locations of these pipes.

#### Stewart/Lenox Interceptor

The Stewart/Lenox Interceptor runs from the intersection of Hwy 140 and Hwy 66 along South Side Bypass then through the east side of the Greensprings area to Riverside Drive, discharging to the Link River Pump Station located on the west bank of Link River at Main Street. The main contributors to flow are from four pump stations: Stewart/Lenox, Running Y, Ridgewater, and Cogen. The model results show that there is minor surcharge in a few sections along this interceptor when there are simultaneous flows from all four pump stations. Based flow data, the likelihood of all four pump stations contributing flow at the same time is very small, and field observations did not reveal signs of surcharging. Further, a number of the pipes were modeled with a Mannings *n* (friction factor) value of 0.017 (carried over from the previous model used for the 2006 Master Plan). A typical *n* value for concrete pipe is 0.013. Some pipes along Riverside Drive have a very flat slope in the model. We recommend the following:

- Verify inverts along the flat pipes;
- TV the pipes noted with high friction factors to verify condition and, thus, the Mannings *n* value used in the model;
- Observe high water levels during typical flows using level sensors.

If the condition of the existing pipes warrants the higher friction factor used in the model and surcharge conditions are observed during ADWF conditions, we would recommend lining the




SHN Consulting Engineers & Geologists, Inc.

City of Klamath Falls
Wastewater Collection System
Klamath Falls, Oregon

Existing Conditions
ADWF Scenario Results
SHN 613011

November 2014 | KFalls_WWsystem_MP_Figures_model_results | Figure 5

Path: C:\My Documents\Projects\613011-KFalls-WWCSMP\GIS\PROJ_MXD\KFalls_WWsystem_MP_Figures_model_results.mxd existing pipes with cured in place pipe (CIPP) to reduce the friction factor and thereby increase capacity.

### Veterans Park to WWTP

This segment is from the confluence of flows from the Link River and California pump stations at Veterans Park and runs along Klamath Avenue and through Timberline Shores to the wastewater treatment plant. Many of the pipes have a very flat grade, which limits their capacity, and the model results (Figure 5) show frequent, though minor, surcharging in Veterans Park, along Klamath Avenue, and in a couple of pipe segments through Timberline Shores. Field observations in Veterans Park showed that the pipes do not surcharge unless there are simultaneous flows from both the California Pump Station and two pumps from the Link River Pump Station. Under current conditions, only one pump at the Link River Pump Station operates at a time.

In 2000, the City installed a 36-inch-diameter relief sewer parallel to the existing 21-inch pipe from near MH 746 in George Nurse Way just downstream of where the flow from the Link River pump station enters to near MH 01-072A at the intersection of Klamath Avenue and 3rd Street. This relief sewer has not been connected and is not in use currently.

Some of the pipe invert data appear to be incorrect. In particular, the pipe between MH 747 and MH 746 has a zero slope in the model. Further, two of the pipes along this segment had reverse grades in the previous model and were revised using estimated inverts. It is recommended that the City resurvey the manhole inverts along this segment in addition to using level sensors to observe high water levels.

If surcharging is found under ADWF conditions, then use of the relief sewer should be evaluated, since frequent surcharging, even if minor, can reduce the life of the existing sewer.

### California Pump Station to Veterans Park

This segment includes the gravity portion where the force main from the California Pump Station discharged into a manhole at California Avenue and Doty Street to Veterans Park at Main Street. Only two pipes show slight overcapacity and slight surcharge from the model results. These pipes have a flatter slope than the adjacent pipes, and the inverts in the model may be incorrect. It is recommended that the inverts in the respective manholes be verified.

### Division Street

Two 10-inch-diameter pipes between MH 03-013 and MH 03-023 appear to be under capacity for dry weather flows. However, surcharging is minor, and no evidence of surcharging was seen during field observations. As discussed later in this chapter, these two pipes appear to be significantly under capacity during peak wet weather flows. We recommend upsizing these two pipes to 12-inch diameter. Prior to conducting this upsizing, the City could install flow meters and level sensors to verify the flows used in the model and the model results.

### Other Locations

Two other pipes, located upstream of the California Pump Station, show a minor amount of surcharge. For one of the pipes, the main contributor to flow is the Hanks Pump Station, while the other pipe is just downstream of the discharge of the Shippington Pump Station. It is recommended to observe the flows when the pump stations are running to verify whether any surcharging is occurring.

**Table 3**
**Model Results For Existing ADWF Conditions[1]**

| Pipe ID | Location | Extg ADWF Max q/Q | Comments (see text for additional comments and recommended actions) |
|---|---|---|---|
| **Stewart/Lenox to Link River PS** | | | |
| 701TO11-137 | South Side Bypass | 1.124 | Minor surcharge when all upstream pumps running, moderately flat slope (0.38%). |
| 11-137TO11-136 | South Side Bypass | 1.201 | Minor surcharge when all upstream pumps running, moderately flat slope (0.33%). |
| 11-157TO11-110 | Greensprings | 1.058 | High friction factor used in model ($n$=0.017), flat slope (0.13%). |
| 11-110TO11-109 | Greensprings | 1.117 | High friction factor used in model ($n$=0.017), flat slope (0.12%). |
| 11-109TO11-108 | Greensprings | 1.386 | High friction factor used in model ($n$=0.017), flat slope (0.077%). |
| 11-144TO11-143 | Riverside Drive | 2.547 | Flat slope (0.0092%). |
| 612TO611 | Riverside Drive | 1.673 | Flat slope (0.021%). |
| **Veterans Park to WWTP** | | | |
| 747TO746 | Veterans Park | 15.382 | Zero slope in model. |
| 746TO745 | Veterans Park | 1.348 | Flat slope (0.13%), most surcharging due to tailwater from 10-017TO01-071. |
| 10-068TO10-070 | Veterans Park | 1.472 | Flat slope (0.10%), most surcharging due to tailwater from 10-017TO01-071. |
| 10-071TO01-071 | Klamath Avenue | 2.103 | Under capacity, flat slope (0.045%). |
| 01-071TO01-072A | Klamath Avenue | 2.626 | Flat slope (0.019%) based on estimated inverts after fixing reverse grade in previous model. |
| 01-148TO01-149 | Timberline Shores | 1.068 | Flat slope (0.031%). |
| **California PS to Veterans Park** | | | |
| 10-041TO608 | Conger Avenue | 1.168 | Flat slope (0.17%). |
| 10-012TO10-011 | Conger Avenue | 1.166 | Flat slope (0.16%). |

| Table 3, Continued | | | |
|---|---|---|---|
| **Pipe ID** | **Location** | **Extg ADWF Max q/Q** | **Comments (see text for additional comments and recommended actions)** |
| **Division Street** | | | |
| 03-013TO03-014 | Division Street | 1.541 | Under capacity. |
| 03-014TO03-023 | Division Street | 1.341 | Under capacity. |
| 03-024TO03-080 | Shasta Avenue @ Division Street | 1.353 | Flat pipe (0.053%). |
| **Other Locations** | | | |
| 12-1TO12-002 | Front Street downstream of Shippington PS | 1.132 | Flat pipe (0.053%). |
| 12-342TO12-18 | California Avenue @ Front Street | 1.01 | Minor surcharging. |
| 1. Only pipes with maximum q/Q greater than or equal to 1.0 are listed. | | | |

## Peak Wet Weather Flow Scenario

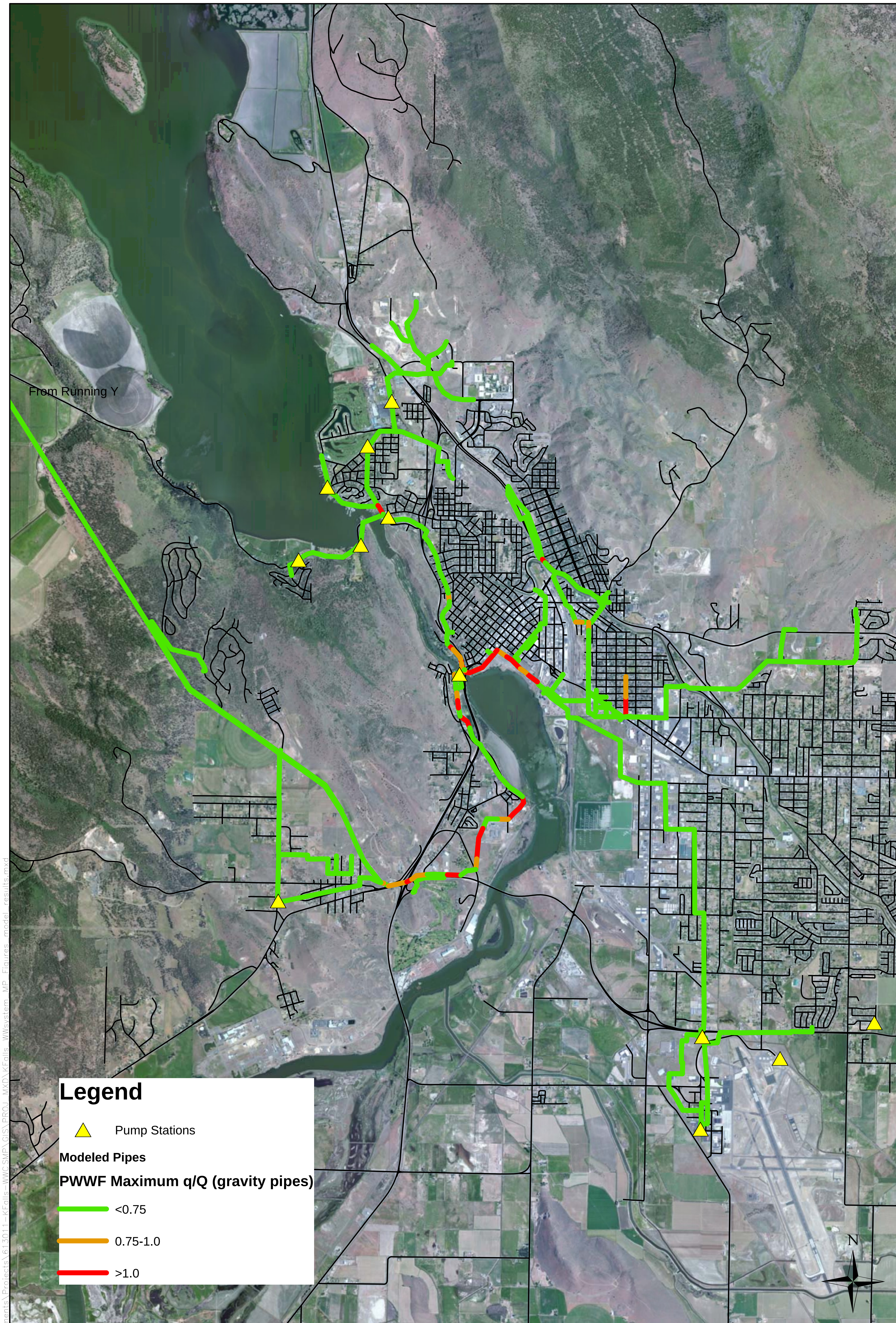
As discussed in Chapter 4, the PWWF scenario was defined by using an effective peaking factor of three. In general, the model results indicate that the existing system has sufficient capacity for peak wet weather conditions with minor surcharge in some areas under certain conditions. In all cases, the hydraulic grade line remained greater than three feet below manhole rim elevations. In discussion with City staff, minor amounts of surcharging during peak wet weather conditions is acceptable as long as the hydraulic grade line remains well below (> 3 ft) manhole rim elevations.

Some upsizing along Division Street is recommended as described below. Field observations of some key areas indicate that the model results may be more conservative than actual conditions as described in the following paragraphs. As such, most pipes are recommended for field measurement of high water elevations and/or flow rates to verify model results. For the few pipes where an upsizing is recommended, we still suggest using level indicators to verify actual conditions prior to pipe upsizing. Table 4 lists the pipes with a maximum ratio of flow to capacity (q/Q) of greater than 0.75. Those pipes with q/Q between 0.75 and 1.0 under the PWWF scenario require no action currently but should be noted, especially in areas of potential future growth. Figure 6 shows the locations of these pipes. The future conditions scenario is discussed later in this chapter.

Unless noted otherwise, the recommendations for those pipes also listed for ADWF conditions remain the same.

### Stewart/Lenox Interceptor

Under the PWWF scenario, surcharging is generally minor along this segment when there are simultaneous flows from all four upstream pump stations. The same issues as described under the ADWF scenario also apply here, namely that a number of pipes are flat and the friction factor used for some pipes seems to be too high. Since it is a relatively rare scenario that all four pump stations




| | | |
|---|---|---|
| SHN Consulting Engineers & Geologists, Inc. | City of Klamath Falls<br>Wastewater Collection System<br>Klamath Falls, Oregon | Existing Conditions<br>PWWF Scenario Results<br>SHN 613011 |
| | November 2014 / KFalls_WWsystem_MP_Figures_model_results | Figure 6 |

Path: C:\My Documents\Projects\613011-KFalls-WWCSMP\GIS\PROJ_MXD\KFalls_WWsystem_MP_Figures_model_results.mxd are contributing flow simultaneously, the model results may be more conservative than typical actual conditions. For example, the Stewart/Lenox pumps currently are set to run for about one minute during each cycle, which means that the likelihood of that flow occurring simultaneously to Running Y and Cogen is very rare. The same recommendations as listed under the ADWF scenario section apply here.

**Veterans Park to WWTP**

Under the PWWF conditions, increased flows from the Stewart/Lenox Interceptor may cause a second pump at the Link River Pump Station to run. When two pumps at the Link River Pump Station are running and contributing flow simultaneously to flows from the California Pump Station, minor surcharging does occur along this segment, as was observed in MH 745.

As mentioned earlier for the ADWF scenario, many of the pipes are flat and some inverts used for this study appear to be incorrect and should be field checked. Monitoring of this segment for surcharging should be undertaken and, if frequent surcharging is occurring, then the City should evaluate the use of the existing unused 36-inch-diameter relief sewer.

**California Pump Station to Veterans Park**

Model results for the PWWF scenario indicate that minor surcharging is occurring. Surcharing appears to be caused by either (1) apparent limited capacity in the pipe or (2) tailwater from Veterans Park. Some of the inverts in the model appear to be incorrect and should be field verified.

**Division Street**

As with the ADWF scenario, the model results indicate that the existing 10-inch-diameter pipes between MH 03-013 and MH 03-023 are under capacity and should be upsized to 12-inch pipes. Surcharging at other pipes is minor.

**Other Locations**

These locations include one pipe along Crater Lake Parkway and several pipes upstream of the California Pump Station. Surcharging is minor and acceptable for PWWF conditions.

**Table 4**
**Model Results For Existing PWWF Conditions**

| Pipe ID | Location | Extg PWWF Max q/Q | Comments (see text for additional comments and recommended actions) |
|---|---|---|---|
| **Stewart/Lenox to Link River PS** | | | |
| 700TO11-118 | South Side Bypass @ US 97 | 0.864 | No action. |
| 11-119TO11-120 | South Side Bypass @ US 97 | 0.888 | No action. |
| 11-120TO11-121 | South Side Bypass @ US 97 | 0.957 | No action. |

| Table 4, Continued | | | |
|---|---|---|---|
| **Pipe ID** | **Location** | **Extg PWWF Max q/Q** | **Comments (see text for additional comments and recommended actions)** |
| **Stewart/Lenox to Link River PS** | | | |
| 11-121TO11-122 | South Side Bypass @ Greensprings Drive | 1.187 | Minor surcharge. |
| 11-122TO11-123 | South Side Bypass | 0.839 | No action. |
| 11-123TO11-141A | South Side Bypass | 0.881 | No action. |
| 701TO11-137 | South Side Bypass | 1.311 | Minor surcharge. |
| 11-137TO11-136 | South Side Bypass | 1.405 | Minor surcharge. |
| 11-133TO11-132 | Greensprings (Memorial Drive) | 0.857 | No action. |
| 11-132TO11-128 | Greensprings (Memorial Drive) | 1.075 | Minor surcharge. |
| 11-128TO11-127 | Greensprings | 1.062 | Minor surcharge. |
| 11-127TO11-126 | Greensprings | 1.029 | Minor surcharge. |
| 11-159TO11-158 | Greensprings | 0.764 | No action. |
| 11-158TO11-157 | Greensprings | 0.840 | No action. |
| 11-157TO11-110 | Greensprings | 1.258 | High friction factor. |
| 11-110TO11-109 | Greensprings | 1.328 | High friction factor. |
| 11-109TO11-108 | Greensprings | 1.650 | High friction factor, flat slope |
| 11-108TO11-107 | Greensprings | 1.018 | High friction factor |
| 11-144TO11-143 | Riverside Drive | 3.135 | Flat slope. |
| 11-143TO11-77 | Riverside Drive | 1.017 | Minor surcharge. |
| 612TO611 | Riverside Drive | 2.082 | Flat slope. |
| 611TO610 | Riverside Drive | 0.962 | No action. |
| 11-014TO11-013 | Riverside Drive | 0.756 | No action. |
| **Veterans Park to WWTP** | | | |
| 748TO747 | Veterans Park | 0.821 | No action. |
| 747TO746 | Veterans Park | 22.837 | Zero slope. |
| 746TO745 | Veterans Park | 2.020 | Tailwater from downstream. |
| 745TO10-068 | Veterans Park | 1.485 | Tailwater from downstream. |
| 10-068TO10-070 | Veterans Park | 2.220 | Tailwater from downstream. |
| 10-070TO10-071 | Veterans Park | 1.409 | Tailwater from downstream. |
| 10-071TO01-071 | Klamath Avenue | 3.186 | Undercapacity, flat slope. |
| 01-071TO01-072A | Klamath Avenue | 3.948 | Flat slope, estimated inverts |
| 01-072ATO01-075 | Timberline Shores | 0.765 | No action. |
| 01-075TO01-165 | Timberline Shores | 1.380 | Minor surcharging. |
| 01-165TO01-142 | Timberline Shores | 1.260 | Minor surcharging. |
| 01-142TO01-143 | Timberline Shores | 1.325 | Minor surcharging. |

| Table 4, Continued | | | |
|---|---|---|---|
| **Pipe ID** | **Location** | **Extg PWWF Max q/Q** | **Comments (see text for additional comments and recommended actions)** |
| **Veterans Park to WWTP** | | | |
| 01-143TO01-144 | Timberline Shores | 0.916 | No action. |
| 01-144TO01-148 | Timberline Shores | 0.849 | No action. |
| 01-148TO01-149 | Timberline Shores | 1.584 | Flat slope. Downstream IE appears to be in error. |
| 01-149TO01-150 | Timberline Shores | 0.782 | No action. |
| **California PS to Veterans Park** | | | |
| 10-165TO10-166 | California Avenue | 0.977 | No action. |
| 606TO607 | Conger Avenue | 1.001 | Minor surcharge. |
| 607TO10-042 | Conger Avenue | 0.808 | No action. |
| 10-042TO10-041 | Conger Avenue | 0.867 | No action. |
| 10-041TO608 | Conger Avenue | 1.322 | Minor surcharge, tailwater from Veterans Park. |
| 608TO10-012 | Conger Avenue | 0.997 | Tailwater from Veterans Park. |
| 10-012TO10-011 | Conger Avenue | 1.336 | Minor surcharge, tailwater from Veterans Park. |
| 10-011TO749 | Conger Avenue | 0.800 | Tailwater from Veterans Park. |
| **Division Street** | | | |
| 03A-001TO03-004 | Division Street | 0.894 | No action. |
| 03-004TO677 | Division Street | 0.859 | No action. |
| 677TO03-013 | Division Street | 0.957 | No action. |
| 03-013TO03-014 | Division Street | 2.315 | Under capacity. |
| 03-014TO03-023 | Division Street | 2.014 | Under capacity. |
| 03-023TO03-024 | Division Street | 1.434 | Minor surcharge. |
| 03-024TO03-080 | Shasta Avenue @ Division Street | 2.033 | Flat slope, minor surcharge. |
| **Other Locations** | | | |
| 02-068TO02-069 | Adams Street | 0.817 | No action. |
| 02-106TO02-066 | Oak Avenue | 0.826 | No action. |
| 650TO06-008 | Crater Lake Parkway | 1.067 | Minor surcharge. |
| 06-008TO05-128 | Crater Lake Parkway | 0.799 | No action. |
| 12-1TO12-002 | Front Street downstream of Shippington PS | 1.132 | Minor surcharge. |
| 12-342TO12-18 | California Avenue @ Front Street | 1.112 | Minor surcharge. |

| Table 4, Continued | | | |
|---|---|---|---|
| **Pipe ID** | **Location** | **Extg PWWF Max q/Q** | **Comments (see text for additional comments and recommended actions)** |
| **Other Locations** | | | |
| 12-18TO12-21 | California Avenue upstream of Calif. PS | 1.073 | Minor surcharge. |
| 12-330TO12-332 | Influent pipe to Calif. PS | 1.174 | Minor surcharge. |
| 1. Only pipes with maximum q/Q greater than or equal to 1.0 are listed. | | | |

## Future Scenario

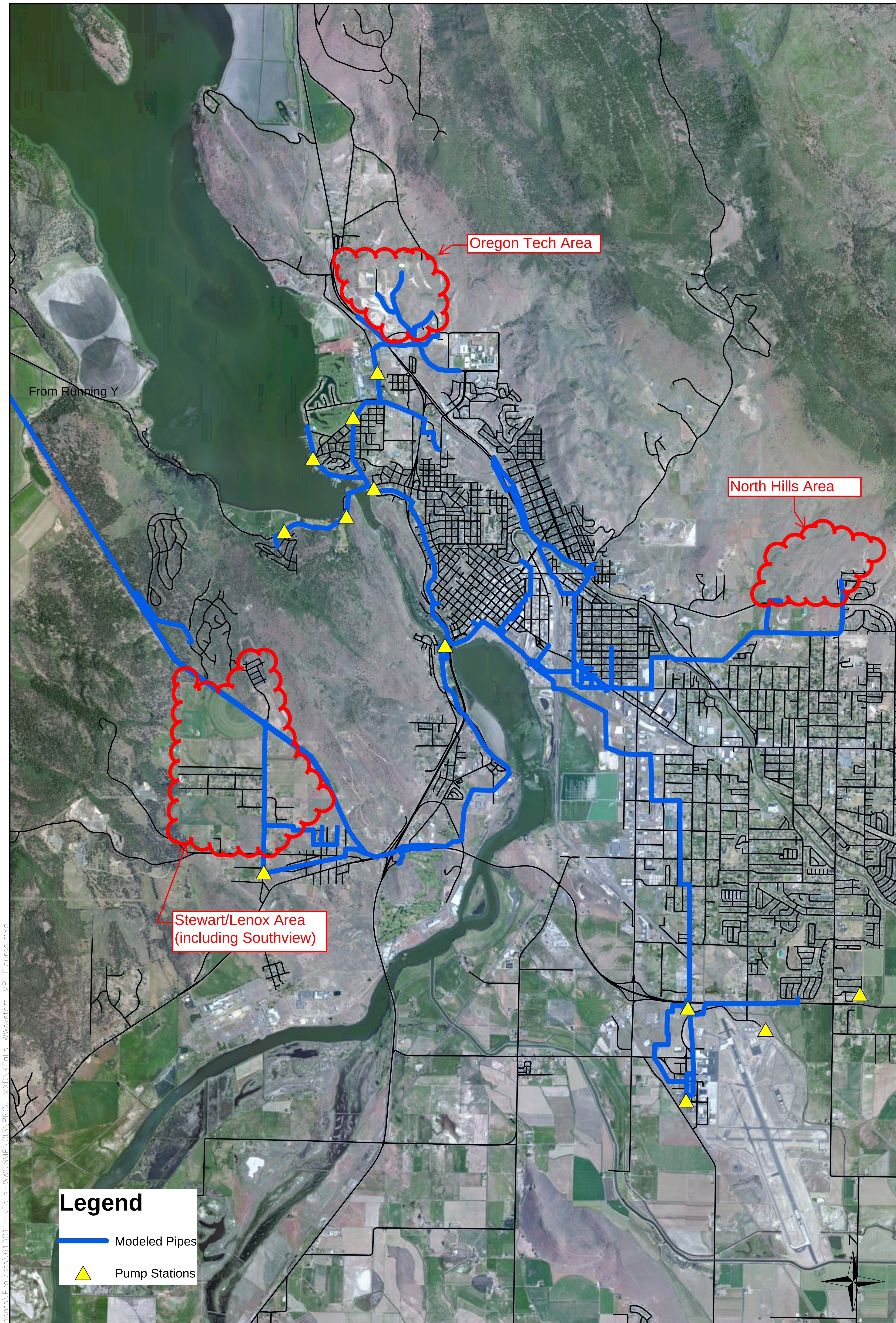
As mentioned earlier in this document, growth has been very flat in Klamath Falls since the economic downturn that started in 2008. As such, it is difficult to define growth scenarios within the near-term planning horizon (five to ten years). Since the existing system generally has capacity for growth in certain areas, it was decided that, for the purposes of this Master Plan update, growth areas would be defined and then the existing system would be evaluated. The model results would determine the amount of additional flow, which equates to a number of equivalent residential units (ERUs), that the existing collection system could safely accommodate.

The potential growth areas evaluated were (1) the North Hills area, (2) the area immediately north of the Oregon Institute of Technology, and (3) the Stewart/Lenox area (Figure 7).

Additional dry weather flows were added at various input nodes and the peak wet weather peaking factor of three (see Chapter 4) was applied to these flows. The modeling was an iterative process to determine the maximum additional flow that could be accommodated within the existing collection system. The results are shown in Table 5 and indicate there is sufficient capacity in the existing system.

| Table 5<br>Capacity Of The Existing Collection System In Key Areas With Significant Growth Potential | | | | |
|---|---|---|---|---|
| **Potential Growth Area** | **Additional ADWF Capacity (cfs)** | **Equivalent ERU Capacity (@265 gpd/ERU)** | **Limiting Element** | **Comments** |
| North Hills | 0.240 | 585 | Pipe 03-028TO03-081 (Shasta Way at South 6th Street); flat slope of 0.015% | The available capacity can be divided between an expansion of the North Hills subdivision and the undeveloped subdivision immediately north of Steens Park. |

| Table 5, Continued | | | | |
|---|---|---|---|---|
| **Potential Growth Area** | **Additional ADWF Capacity (cfs)** | **Equivalent ERU Capacity (@265 gpd/ERU)** | **Limiting Element** | **Comments** |
| Oregon Tech Area | 0.136 | 332 | Pearl Pump Station | Pearl is a duplex pump station, so it was assumed only one pump (@585 gpm) handles PWWF. As growth occurs in this area, intermediate modeling should be performed to verify capacity. |
| Stewart/Lenox | 0.293 | 715 | Stewart/Lenox Pump Station | This area is upstream of the Stewart/Lenox Pump Station. One pump (@610 gpm) for PWWF. The ERUs include Southview, since its flows go to the Stewart/Lenox Pump Station. |

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Wastewater Collection System
Klamath Falls, Oregon

Future Growth Potential Areas
SHN 613011

November 2014

KFalls_WWsystem_MP_Figures

Figure 7

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Chapter 6

# Conclusions and Recommendations

Based on the modeling results, in conjunction with field observations and discussion with City staff, the following recommendations are made:

1. Verify inverts of various pipes that have inverts suspected of being incorrect.
2. TV the pipe segments along the Stewart/Lenox Interceptor with high Mannings *n* value to verify condition.
3. Monitor high water elevations in key areas to verify whether surcharging is occurring during dry weather flows.
   a. Highest priority location is along Veterans Park and Klamath Avenue.
   b. Other locations as described in Chapter 5 with equal priority for other segments.
4. Install flow meters on the discharge of the Hanks and Pearl pump stations to verify pumping capacities.
5. Upsize 640 LF of pipe along Division Street between MHs 03-013 and 03-023 to 12 inches. Estimated construction cost: $90,000 ($10/in/ft plus 15% for design and inspection). Verify flows and whether surcharging is occurring during dry weather conditions.
6. Develop a flow monitoring plan to gather flow data at key locations throughout the collection system to refine the inflow values.

Appendix

# Supporting Documentation

CONSULTING ENGINEERS & GEOLOGISTS, INC.
PO Box 460
Klamath Falls, OR 97601
Tel. 541/827-7855

| | | | |
|---|---|---|---|
| JOB | 613011 | | |
| SHEET NO. | 1 | OF | |
| CALC'ED BY | AHR | DATE | 6/12/2014 |
| CHECKED BY | | DATE | |

**Pump Drawdown Test Analysis**

**Pump Station:** Shippington
**SCADA data dates:** 5/28/14 12:00 PM - 5/29/14 11:59 AM

Wet well description and dimensions: 4 ft x 8 ft
Wet well plan area: 32 sf
Notes: 1. $Q_d = Q_{drawdown} = Q_{pump} - Q_{fill}$
2. $Q_{pump} = Q_d + Q_{fill}$

| | **Drawdown** | | | **Fill** | | | **Pump** |
|---|---|---|---|---|---|---|---|
| Pump # | ΔH (ft) | Δt (sec) | $Q_d$ (gpm) | ΔH (ft) | Δt (sec) | $Q_{fill}$ (gpm) | $Q_{pump}$ (gpm) |
| 1 | 1.63 | 60 | 390 | 0.17 | 60 | 41 | 431 |
| 2 | 1.94 | 60 | 464 | 0.05 | 60 | 12 | 476 |
| 2 | 2.51 | 60 | 601 | 0.07 | 60 | 17 | 618 |
| 1 | 1.24 | 60 | 297 | 0.14 | 60 | 34 | 330 |
| 2 | 1.59 | 60 | 381 | 0.10 | 60 | 24 | 405 |
| 1 | 2.11 | 60 | 505 | 0.06 | 60 | 14 | 519 |
| 1 | 0.92 | 60 | 220 | 0.12 | 60 | 29 | 249 |
| 2 | 0.85 | 60 | 203 | 0.00 | 60 | 0 | 203 |
| 2 | 0.43 | 60 | 103 | 0.10 | 60 | 24 | 127 |
| 1 | 1.88 | 61 | 443 | 0.14 | 60 | 34 | 476 |
| 1 | 1.61 | 60 | 385 | 0.19 | 60 | 45 | 431 |
| 2 | 1.12 | 60 | 268 | 0.11 | 60 | 26 | 294 |
| 2 | 1.23 | 60 | 294 | 0.19 | 60 | 45 | 340 |
| 1 | 0.48 | 60 | 115 | 0.28 | 60 | 67 | 182 |
| 2 | 1.36 | 60 | 326 | 0.09 | 60 | 22 | 347 |
| 2 | 0.74 | 60 | 177 | 0.15 | 60 | 36 | 213 |
| 2 | 1.31 | 60 | 314 | 0.18 | 60 | 43 | 357 |
| 1 | 1.44 | 60 | 345 | 0.07 | 60 | 17 | 361 |
| 2 | 2.21 | 60 | 529 | 0.20 | 60 | 48 | 577 |
| 1 | 1.48 | 60 | 354 | 0.05 | 60 | 12 | 366 |
| 1 | 1.06 | 60 | 254 | 0.15 | 60 | 36 | 290 |
| 1 | 2.47 | 60 | 591 | 0.13 | 60 | 31 | 622 |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

**Summary:**

| Pump # | min $Q_{pump}$ | max $Q_{pump}$ | avg $Q_{pump}$ | Count |
|---|---|---|---|---|
| 1 | 182 | 622 | 387 | 11 |
| 2 | 127 | 618 | 360 | 11 |
| 3 | 0 | 0 | | 0 |

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## Pump Drawdown Test Analysis

**Pump Station:** Stewart Lennox
**SCADA data dates:** 5/28/14 12:00 PM - 5/29/14 11:59 AM

Wet well description and dimensions: 9 ft x 9 ft with beveled corners
Wet well plan area: 79 sf
Notes: 1. $Q_d = Q_{drawdown} = Q_{pump} - Q_{fill}$
2. $Q_{pump} = Q_d + Q_{fill}$

| | Drawdown | | | Fill | | | Pump |
|---|---|---|---|---|---|---|---|
| Pump # | ΔH (ft) | Δt (sec) | $Q_d$ (gpm) | ΔH (ft) | Δt (sec) | $Q_{fill}$ (gpm) | $Q_{pump}$ (gpm) |
| 2 | 1.30 | 60 | 768 | 0.10 | 60 | 59 | 827 |
| 1 | 1.10 | 60 | 650 | 0.10 | 120 | 30 | 680 |
| 2 | 0.60 | 60 | 355 | 0.10 | 60 | 59 | 414 |
| 2 | 0.60 | 60 | 355 | 0.10 | 60 | 59 | 414 |
| 1 | 1.10 | 60 | 650 | 0.10 | 60 | 59 | 709 |
| 2 | 1.00 | 60 | 591 | 0.10 | 120 | 30 | 620 |
| 1 | 1.10 | 60 | 650 | 0.10 | 60 | 59 | 709 |
| 2 | 1.20 | 60 | 709 | 0.10 | 60 | 59 | 768 |
| 1 | 1.00 | 60 | 591 | 0.10 | 60 | 59 | 650 |
| 2 | 0.60 | 60 | 355 | 0.10 | 60 | 59 | 414 |
| 2 | 0.90 | 60 | 532 | 0.10 | 60 | 59 | 591 |
| 1 | 0.90 | 60 | 532 | 0.10 | 60 | 59 | 591 |
| 2 | 1.10 | 60 | 650 | 0.10 | 60 | 59 | 709 |
| 2 | 0.70 | 60 | 414 | 0.10 | 120 | 30 | 443 |
| 1 | 1.00 | 60 | 591 | 0.10 | 120 | 30 | 620 |
| 2 | 0.90 | 60 | 532 | 0.10 | 120 | 30 | 561 |
| 1 | 0.90 | 60 | 532 | 0.00 | 60 | 0 | 532 |
| 1 | 1.00 | 60 | 591 | 0.10 | 60 | 59 | 650 |
| 1 | 1.20 | 60 | 709 | 0.10 | 60 | 59 | 768 |
| 2 | 1.10 | 78 | 500 | 0.10 | 60 | 59 | 559 |
| 1 | 0.90 | 60 | 532 | 0.10 | 120 | 30 | 561 |
| 2 | 1.10 | 60 | 650 | 0.20 | 59 | 120 | 770 |
| 1 | 1.10 | 60 | 650 | 0.00 | 60 | 0 | 650 |
| 2 | 0.70 | 60 | 414 | 0.10 | 48 | 74 | 488 |
| 1 | 0.50 | 60 | 295 | 0.10 | 120 | 30 | 325 |
| 2 | 1.10 | 60 | 650 | 0.10 | 60 | 59 | 709 |
| 1 | 1.10 | 60 | 650 | 0.10 | 120 | 30 | 680 |
| 2 | 1.10 | 60 | 650 | 0.00 | 60 | 0 | 650 |
| | | | | | | | |

**Summary:**

| Pump # | min $Q_{pump}$ | max $Q_{pump}$ | avg $Q_{pump}$ | Count |
|---|---|---|---|---|
| 1 | 325 | 768 | 625 | 13 |
| 2 | 414 | 827 | 596 | 15 |
| 3 | 0 | 0 | | 0 |

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| CHECKED BY | | DATE | |

**Pump Drawdown Test Analysis**

**Pump Station:** Pearl
**SCADA data range:** 5/28/14 12:00 PM - 5/29/14 11:59 AM

Wet well description and dimensions: 8 ft diameter
Wet well plan area: 50.3 sf
Notes: 1. $Q_d = Q_{drawdown} = Q_{pump} - Q_{fill}$
2. $Q_{pump} = Q_d + Q_{fill}$

| | Drawdown | | | Fill | | | Pump |
|---|---|---|---|---|---|---|---|
| Pump # | ΔH (ft) | Δt (sec) | $Q_d$ (gpm) | ΔH (ft) | Δt (sec) | $Q_{fill}$ (gpm) | $Q_{pump}$ (gpm) |
| 2 | 1.68 | 60 | 632 | 0.04 | 60 | 15 | 647 |
| 1 | 0.65 | 60 | 245 | 0.27 | 60 | 102 | 346 |
| 2 | 1.71 | 62 | 623 | 0.16 | 60 | 60 | 683 |
| 3 | 2.80 | 120 | 527 | 0.29 | 60 | 109 | 636 |
| 1 | 2.04 | 120 | 384 | 0.51 | 60 | 192 | 576 |
| 3 | 2.75 | 120 | 517 | 0.39 | 64 | 138 | 655 |
| 1 | 2.14 | 120 | 403 | 0.35 | 60 | 132 | 534 |
| 2 | 1.64 | 60 | 617 | 0.01 | 60 | 4 | 621 |
| 3 | 1.93 | 77 | 566 | 0.25 | 60 | 94 | 660 |
| 3 | 0.87 | 60 | 327 | 0.41 | 61 | 152 | 479 |
| 3 | 1.54 | 60 | 579 | 0.03 | 60 | 11 | 591 |
| 1 | 2.58 | 120 | 485 | 0.19 | 60 | 71 | 557 |
| 3 | 1.88 | 60 | 707 | 0.13 | 60 | 49 | 756 |
| 1 | 2.15 | 120 | 404 | 0.17 | 76 | 50 | 455 |
| 1 | 1.77 | 60 | 666 | 0.25 | 60 | 94 | 760 |
| 2 | 0.99 | 59 | 379 | 0.22 | 60 | 83 | 462 |
| 3 | 1.65 | 60 | 621 | 0.37 | 61 | 137 | 758 |
| 1 | 2.00 | 120 | 376 | 0.23 | 60 | 87 | 463 |
| 3 | 2.65 | 120 | 499 | 0.30 | 60 | 113 | 611 |
| 1 | 2.05 | 120 | 386 | 0.29 | 60 | 109 | 495 |
| 2 | 2.21 | 120 | 416 | 0.40 | 60 | 150 | 566 |
| 1 | 2.17 | 120 | 408 | 0.14 | 60 | 53 | 461 |
| 2 | 1.42 | 60 | 534 | 0.04 | 60 | 15 | 549 |
| 3 | 2.56 | 120 | 482 | 0.55 | 86 | 144 | 626 |
| 1 | 2.94 | 180 | 369 | 0.30 | 60 | 113 | 482 |
| 2 | 2.40 | 120 | 451 | 0.14 | 60 | 53 | 504 |
| 3 | 2.80 | 120 | 527 | 0.30 | 60 | 113 | 640 |
| 1 | 2.91 | 180 | 365 | 0.45 | 60 | 169 | 534 |
| 2 | 1.69 | 60 | 636 | 0.30 | 60 | 113 | 749 |

**Summary:**

| Pump # | min $Q_{pump}$ | max $Q_{pump}$ | avg $Q_{pump}$ | Count |
|---|---|---|---|---|
| 1 | 346 | 760 | 515 | 11 |
| 2 | 462 | 749 | 598 | 8 |
| 3 | 479 | 758 | 641 | 10 |

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**Pump Drawdown Test Analysis**

**Pump Station:** Hanks
**SCADA data dates:** 5/28/14 12:00 PM - 5/29/14 11:59 AM

Wet well description and dimensions: 8 ft diameter
Wet well plan area: 50.3 sf
Notes: 1. $Q_d = Q_{drawdown} = Q_{pump} - Q_{fill}$
2. $Q_{pump} = Q_d + Q_{fill}$

| | Drawdown | | | Fill | | | Pump |
|---|---|---|---|---|---|---|---|
| Pump # | ΔH (ft) | Δt (sec) | $Q_d$ (gpm) | ΔH (ft) | Δt (sec) | $Q_{fill}$ (gpm) | $Q_{pump}$ (gpm) |
| 3 | 2.10 | 152 | 312 | 0.33 | 60 | 124 | 436 |
| 1 | 1.90 | 120 | 357 | 0.08 | 60 | 30 | 388 |
| 2 | 2.60 | 120 | 489 | 0.17 | 60 | 64 | 553 |
| 2 | 1.80 | 121 | 336 | 0.25 | 60 | 94 | 430 |
| 3 | 1.30 | 60 | 489 | 0.17 | 60 | 64 | 553 |
| 3 | 2.30 | 120 | 433 | 0.17 | 60 | 64 | 497 |
| 1 | 2.80 | 120 | 527 | 0.17 | 60 | 64 | 591 |
| 1 | 2.00 | 120 | 376 | 0.17 | 83 | 46 | 422 |
| 2 | 1.80 | 120 | 339 | 0.08 | 60 | 30 | 369 |
| 3 | 2.00 | 120 | 376 | 0.33 | 60 | 124 | 500 |
| 2 | 1.60 | 120 | 301 | 0.25 | 60 | 94 | 395 |
| 1 | 2.40 | 120 | 451 | 0.33 | 60 | 124 | 576 |
| 2 | 2.10 | 120 | 395 | 0.17 | 60 | 64 | 459 |
| 3 | 1.20 | 60 | 451 | 0.25 | 60 | 94 | 546 |
| 1 | 2.20 | 120 | 414 | 0.25 | 60 | 94 | 508 |
| 2 | 1.40 | 60 | 527 | 0.17 | 60 | 64 | 591 |
| 3 | 2.30 | 120 | 433 | 0.33 | 60 | 124 | 557 |
| 2 | 2.30 | 120 | 433 | 0.17 | 60 | 64 | 497 |
| 2 | 2.10 | 120 | 395 | 0.17 | 60 | 64 | 459 |
| 3 | 2.10 | 120 | 395 | 0.17 | 60 | 64 | 459 |
| 2 | 1.10 | 60 | 414 | 0.08 | 60 | 30 | 444 |
| 1 | 2.30 | 125 | 415 | 0.08 | 60 | 30 | 445 |
| 2 | 1.30 | 60 | 489 | 0.08 | 60 | 30 | 519 |
| 3 | 2.10 | 120 | 395 | 0.08 | 60 | 30 | 425 |
| 1 | 1.80 | 120 | 339 | 0.17 | 66 | 58 | 397 |
| 2 | 1.50 | 95 | 356 | 0.25 | 60 | 94 | 451 |
| 3 | 1.70 | 120 | 320 | 0.08 | 60 | 30 | 350 |
| 1 | 2.80 | 133 | 475 | 0.25 | 60 | 94 | 569 |
| 2 | 2.20 | 120 | 414 | 0.17 | 71 | 54 | 468 |

**Summary:**

| Pump # | min $Q_{pump}$ | max $Q_{pump}$ | avg $Q_{pump}$ | Count |
|---|---|---|---|---|
| 1 | 388 | 591 | 487 | 8 |
| 2 | 369 | 591 | 469 | 12 |
| 3 | 350 | 557 | 480 | 9 |

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**Pump Drawdown Test Analysis**

**Pump Station:** Lynnewood
**SCADA data dates:** 5/28/14 12:00 PM - 5/29/14 11:59 AM

Wet well description and dimensions: 7.5 ft x 7.5 ft
Wet well plan area: 56.25 sf
Notes: 1. $Q_d = Q_{drawdown} = Q_{pump} - Q_{fill}$
2. $Q_{pump} = Q_d + Q_{fill}$

| | Drawdown | | | Fill | | | Pump |
|---|---|---|---|---|---|---|---|
| Pump # | ΔH (ft) | Δt (sec) | $Q_d$ (gpm) | ΔH (ft) | Δt (sec) | $Q_{fill}$ (gpm) | $Q_{pump}$ (gpm) |
| 2 | 0.61 | 60 | 257 | 0.02 | 60 | 8 | 265 |
| 1 | 0.90 | 61 | 372 | 0.10 | 60 | 42 | 415 |
| 2 | 0.73 | 60 | 307 | 0.04 | 60 | 17 | 324 |
| 1 | 0.71 | 60 | 299 | 0.08 | 60 | 34 | 332 |
| 1 | 0.93 | 60 | 391 | 0.06 | 60 | 25 | 417 |
| 2 | 0.58 | 60 | 244 | 0.08 | 60 | 34 | 278 |
| 1 | 0.72 | 60 | 303 | 0.08 | 60 | 34 | 337 |

**Summary:**

| Pump # | min $Q_{pump}$ | max $Q_{pump}$ | avg $Q_{pump}$ | Count |
|---|---|---|---|---|
| 1 | 332 | 417 | 375 | 4 |
| 2 | 265 | 324 | 289 | 3 |
| 3 | 0 | 0 | | 0 |

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**Pump Drawdown Test Analysis**

**Pump Station:** Moore Park
**SCADA data dates:** 5/28/14 12:00 PM - 5/29/14 11:59 AM

Wet well description and dimensions: 8 ft x 10 ft
Wet well plan area: 80 sf
Notes: 1. $Q_d = Q_{drawdown} = Q_{pump} - Q_{fill}$
2. $Q_{pump} = Q_d + Q_{fill}$

| | Drawdown | | | Fill | | | Pump |
|---|---|---|---|---|---|---|---|
| Pump # | ΔH (ft) | Δt (sec) | $Q_d$ (gpm) | ΔH (ft) | Δt (sec) | $Q_{fill}$ (gpm) | $Q_{pump}$ (gpm) |
| 1 | 0.57 | 60 | 341 | 0.03 | 60 | 18 | 359 |
| 2 | 0.59 | 60 | 353 | 0.03 | 60 | 18 | 371 |
| 1 | 0.53 | 60 | 317 | 0.06 | 87 | 25 | 342 |
| 2 | 1.04 | 60 | 622 | 0.01 | 60 | 6 | 628 |
| 2 | 1.16 | 60 | 694 | 0.16 | 60 | 96 | 790 |
| 1 | 0.92 | 60 | 551 | 0.14 | 60 | 84 | 634 |
| 1 | 1.23 | 60 | 736 | 0.08 | 60 | 48 | 784 |

**Summary:**

| Pump # | min $Q_{pump}$ | max $Q_{pump}$ | avg $Q_{pump}$ | Count |
|---|---|---|---|---|
| 1 | 342 | 784 | 530 | 4 |
| 2 | 371 | 790 | 596 | 3 |
| 3 | 0 | 0 | | 0 |

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| CHECKED BY | | DATE | |

**Pump Drawdown Test Analysis**

**Pump Station:** Link River
**SCADA data dates:** 5/28/14 12:00 PM - 5/29/14 11:59 AM

Wet well description and dimensions: 9 ft x 9 ft with beveled corners
Wet well plan area: 79 sf
Notes: 1. $Q_d = Q_{drawdown} = Q_{pump} - Q_{fill}$
2. $Q_{pump} = Q_d + Q_{fill}$

| | Drawdown | | | Fill | | | Pump |
|---|---|---|---|---|---|---|---|
| Pump # | ΔH (ft) | Δt (sec) | $Q_d$ (gpm) | ΔH (ft) | Δt (sec) | $Q_{fill}$ (gpm) | $Q_{pump}$ (gpm) |
| 2 | 1.89 | 120 | 558 | 0.60 | 60 | 355 | 913 |
| 3 | 3.44 | 120 | 1016 | 0.51 | 60 | 301 | 1318 |
| 1 | 3.07 | 120 | 907 | 0.47 | 60 | 278 | 1185 |
| 2 | 2.76 | 120 | 815 | 0.26 | 60 | 154 | 969 |
| 3 | 3.36 | 120 | 993 | 0.10 | 62 | 57 | 1050 |
| 2 | 2.78 | 121 | 815 | 0.41 | 60 | 242 | 1057 |
| 3 | 2.96 | 120 | 875 | 0.82 | 60 | 485 | 1359 |
| 1 | 2.76 | 120 | 815 | 0.61 | 60 | 360 | 1176 |
| 2 | 3.17 | 120 | 937 | 0.43 | 60 | 254 | 1191 |
| 2 | 2.65 | 131 | 717 | 0.48 | 60 | 284 | 1001 |
| 3 | 3.91 | 120 | 1155 | 0.15 | 60 | 89 | 1244 |
| 3 | 2.48 | 120 | 733 | 0.40 | 60 | 236 | 969 |
| 1 | 3.73 | 120 | 1102 | 0.50 | 60 | 295 | 1398 |
| 3 | 2.34 | 120 | 691 | 0.84 | 60 | 496 | 1188 |
| 1 | 3.31 | 120 | 978 | 1.00 | 112 | 317 | 1295 |
| 2 | 3.12 | 120 | 922 | 0.57 | 60 | 337 | 1259 |
| 3 | 3.07 | 120 | 907 | 0.53 | 60 | 313 | 1220 |
| 1 | 2.00 | 120 | 591 | 0.78 | 60 | 461 | 1052 |
| 2 | 2.37 | 120 | 700 | 0.57 | 60 | 337 | 1037 |
| 3 | 3.06 | 120 | 904 | 0.43 | 60 | 254 | 1158 |
| 1 | 3.52 | 167 | 747 | 0.47 | 60 | 278 | 1025 |
| 1 | 3.56 | 120 | 1052 | 0.37 | 60 | 219 | 1270 |
| 2 | 3.08 | 120 | 910 | 0.23 | 60 | 136 | 1046 |
| 2 | 3.85 | 160 | 853 | 0.19 | 60 | 112 | 965 |
| 3 | 2.92 | 120 | 863 | 0.44 | 72 | 217 | 1079 |
| 1 | 3.23 | 120 | 954 | 0.90 | 60 | 532 | 1486 |
| 2 | 2.34 | 120 | 691 | 0.63 | 60 | 372 | 1064 |
| 3 | 3.51 | 120 | 1037 | 0.45 | 60 | 266 | 1303 |
| 1 | 3.40 | 120 | 1005 | 0.34 | 60 | 201 | 1205 |

**Summary:**

| Pump # | min $Q_{pump}$ | max $Q_{pump}$ | avg $Q_{pump}$ | Count |
|---|---|---|---|---|
| 1 | 1025 | 1486 | 1232 | 9 |
| 2 | 913 | 1259 | 1050 | 10 |
| 3 | 969 | 1359 | 1189 | 10 |