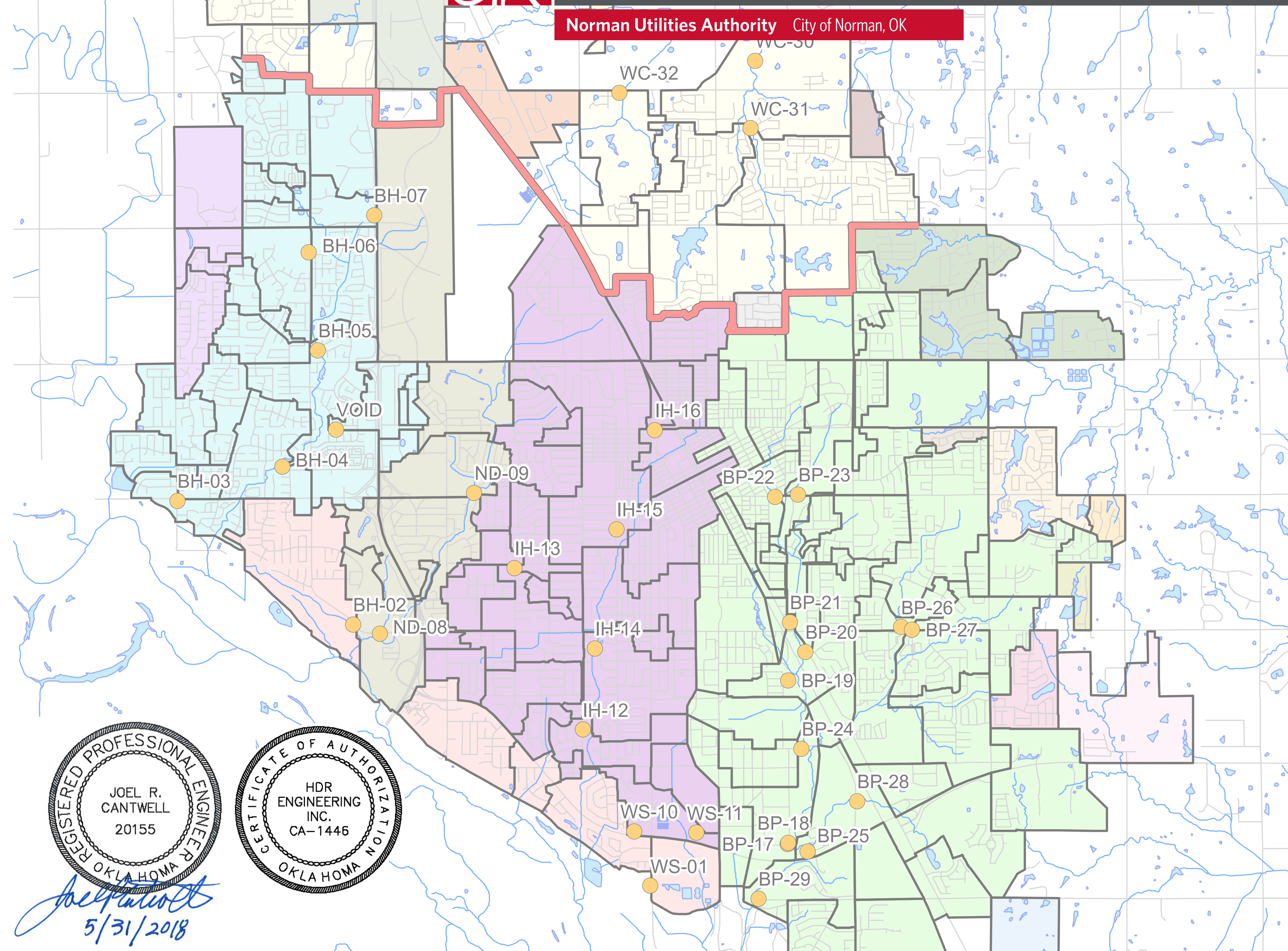
# Wastewater Flow Monitoring & Modeling Report

**FINAL REPORT** | May 2018

**Norman Utilities Authority** City of Norman, OK

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Vieux & Associates, Inc.

# Wastewater Flow Monitoring and Modeling Report

FINAL Report

**Norman Utilities Authority**
City of Norman
May 2018

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# 1 Introduction

## 1.1 Background

Norman's wastewater collection system consists of gravity interceptors and collectors, lift stations, and force mains that convey all wastewater to the existing Norman Water Reclamation Facility (WRF) on the south side of the city for treatment and discharge to the Canadian River. Norman (City) is a growing and thriving community, and as the area served by the system increases in area and population density, the ability of the collection system to adequately collect and convey the resulting wastewater flows is challenged.

In 2001, the City completed a wastewater master plan which recommended improvements to the collection system to convey predicted future flows. Most of the recommended projects have been constructed. Additionally, the master plan recommended a second WRF in the northern part of the City to serve the areas which flow by gravity to the north but are currently pumped to the south. The decision on whether or not to construct the second WRF in the future has not been made to date.

## 1.2 Purpose

The purpose of the Wastewater Flow Monitoring and Modeling Report is three-fold:

- Conduct updated flow monitoring and modeling of the system to determine the effectiveness of the collection system improvements that have been implemented since the last planning process.
- Determine additional collection system improvements that need to be implemented to handle future build-out flows considering updated land use designations, future service areas, and population projections.
- Determine an alternative set of additional collection system improvements that would need to be made to handle future flows if the City constructs a new WRF on the northern side of the City.

## 1.3 Report Organization

This *Wastewater Flow Monitoring and Modeling Report* contains the following content:

- Chapter 2 describes the existing collection system and the development and calibration of an updated hydraulic model.
- Chapter 3 documents the hydraulic modeling analysis of the collection system to determine effectiveness of recent recommended capital improvements and remaining discrepancies that need to be addressed.

- Chapter 4 presents locations, sizes, and estimated costs of additional recommended collection system improvements for two scenarios: 1) continuing to rely on the existing WRF to treat all wastewater or 2) constructing a new WRF to serve the northern side of the City.

- Chapter 5 summarizes the results.

# 2 Collection System Model Development

The purpose of this chapter is to describe the process used to construct the collection system model including the network update, load allocation and calibration conducted to develop the model to a level of completeness for performing the analyses required.

## 2.1 Wastewater Collection System Description

### 2.1.1 Previous Studies

The most recent study of the City of Norman's (City's) wastewater collection system was conducted by Camp Dresser & McKee and is documented in the *Wastewater Master Plan* (*Master Plan*) dated September 2001. Several refinements have been made to the *Master Plan* recommendations since its completion, and the paragraphs below present the most current information on the existing collection system, future service areas, and proposed improvements as they relate to this Project.

According to past wastewater modeling efforts, the full build-out of the *Norman 2025 Land Use and Transportation Plan* (*Norman 2025 Plan*) will result in a total average daily flow of 21.5 mgd, including 17.0 mgd in the south service basins and 4.5 mgd in the north service basins. These values are termed the Full Build-Out Flows.

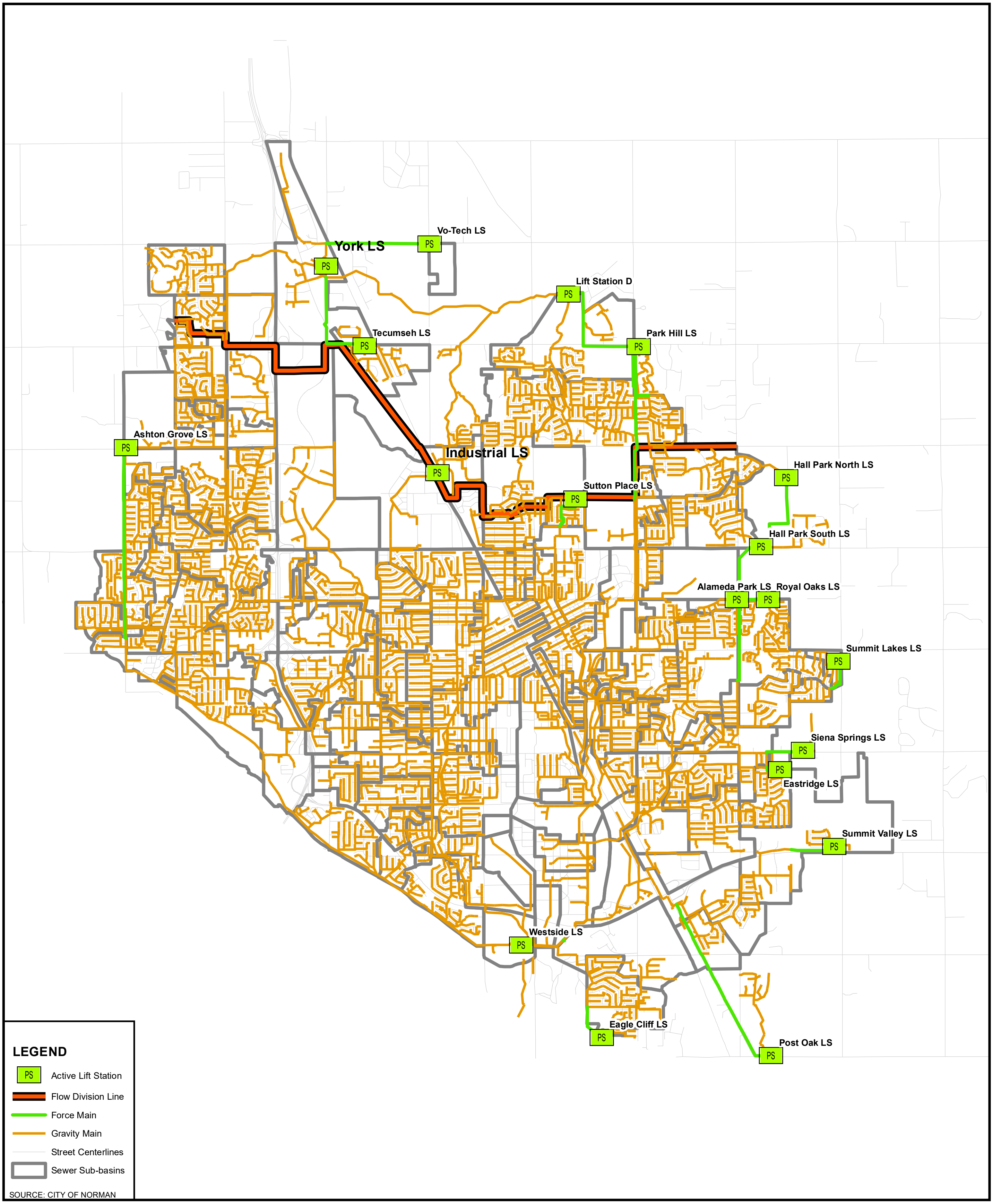
### 2.1.2 Existing Collection System

**Figure 2-1** shows the sewer basins, gravity mains, force mains, and lift stations of the existing system as of May 30, 2010. The entire system is served by the Norman Water Reclamation Facility (WRF) located at the south end of the City.

The red line on **Figure 2-1** represents the geographic boundary between sewersheds that naturally flow to the north and those that naturally flow to the south. The areas to the north of the boundary are currently serviced by lift stations, which pump the wastewater over the ridgeline and into the south gravity system. Including all areas with existing sewer service and those to which the City is obligated to provide service, the average daily wastewater flow to the existing WRF, according to the *Master Plan*, is 13.9 million gallons per day (mgd), which includes the contribution from the developed, vacant final platted and vacant preliminary platted areas as of August 2000.

### 2.1.3 Future Service Areas

The *Norman 2025 Plan* defines 16 Future Service Areas (FSAs), which are wastewater service basins where service is not currently provided but will be needed for future development. As shown on **Figure 2-2**, eight of the FSAs are located north of the flow division boundary while the other eight are located to the south. As part of the *Norman 2025 Plan*, wastewater collection infrastructure will be constructed in the FSAs to serve the full build-out of those areas. For the work associated with this project, the City has also identified future extensions. These are areas of potential extensions to the identified FSAs in the *Norman 2025 Plan.*




**CITY OF NORMAN**
**FLOW MONITORING AND MODELING**
EXISTING WASTEWATER COLLECTION SYSTEM
(AS OF 2010)

N

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Miles

**NORMAN UTILITIES AUTHORITY**

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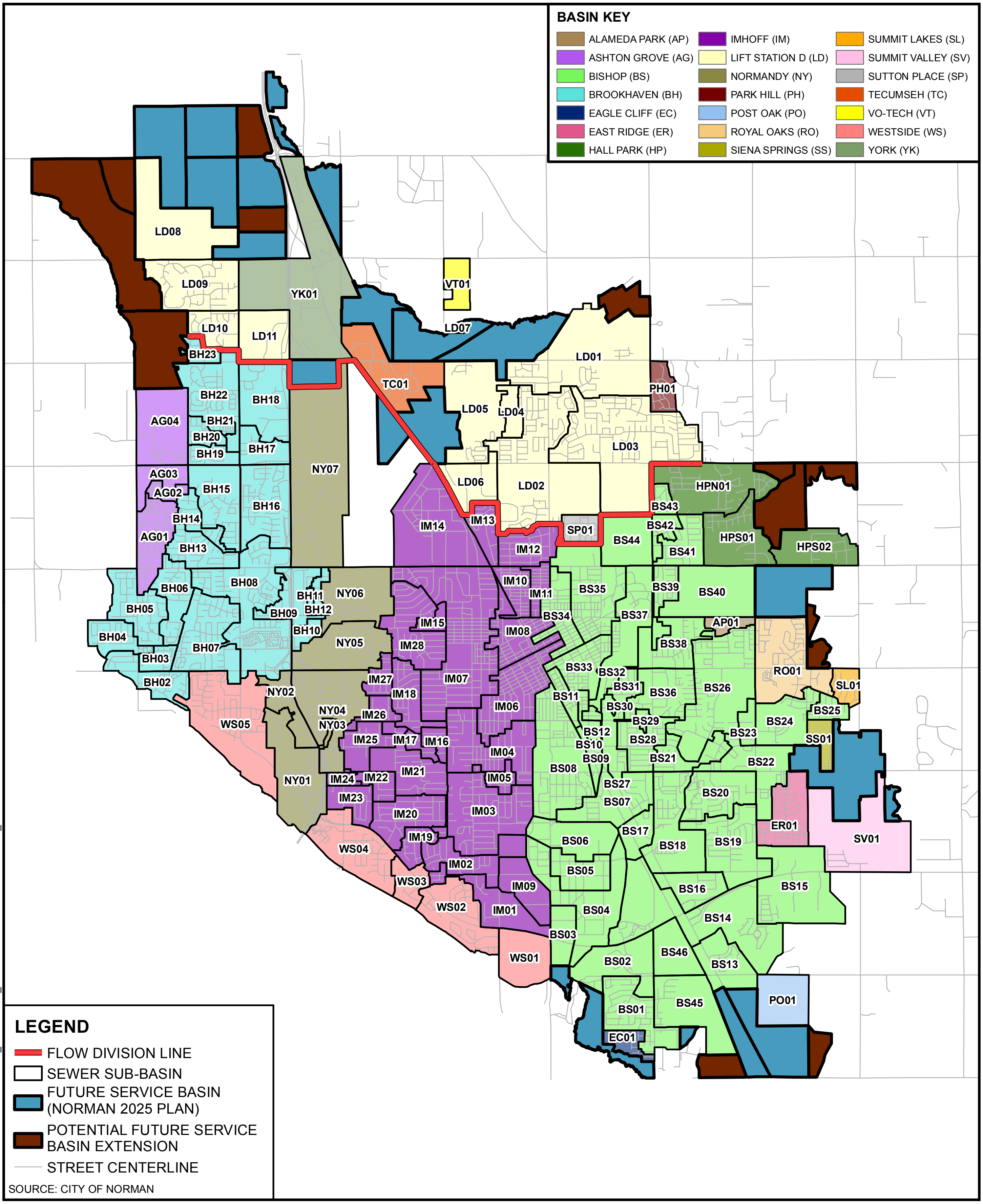
972-960-4400

**FIGURE 2-1**

PATH: C:\USERS\TWHITFIE\DOCUMENTS_WORKING DOCUMENTS_PROJECTS_ACTIVE\NORMAN\2017_FIGURE_RECREATION\FIGURE2-1.MXD - USER: TWHITFIE - DATE: 10/26/2017




FILE: J:\MAP_DOCS\MXD\2005 PROJECT\PN139145_FIGURE1-2_WASTEWATERSERVICEBASINS_8X11.MXD

**CITY OF NORMAN**
**FLOW MONITORING & MODELING**

WASTEWATER SERVICE BASINS

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MILES

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972-960-4400

FIGURE 2-2

## 2.2 Flow Monitoring

Flow monitoring for the project was conducted by the RJN Group (RJN). The flow monitoring report was submitted by RJN in November 2010 under separate cover. The collection system was divided into 32 sub-basins for the purposes of flow monitoring with an average of 83,300 linear feet of pipe per sub-basin. Thirty-two (32) continuously recording flow meters and ten (10) rain gauges were installed.

The original flow monitoring period was from May 1, 2010, through August 1, 2010. Due to a lack of rainfall events, the flow monitoring period was extended. Flow monitoring was terminated on September 15, 2010, after the large rainfall event from Tropical Storm Hermine was captured on September 7-9. The total flow monitoring period was 135 days (4 and a half months).

RJN processed the data and balanced the readings by basin to provide consistent data. Hydrographs were prepared for each sub-basin for subsequent use in model development and calibration.

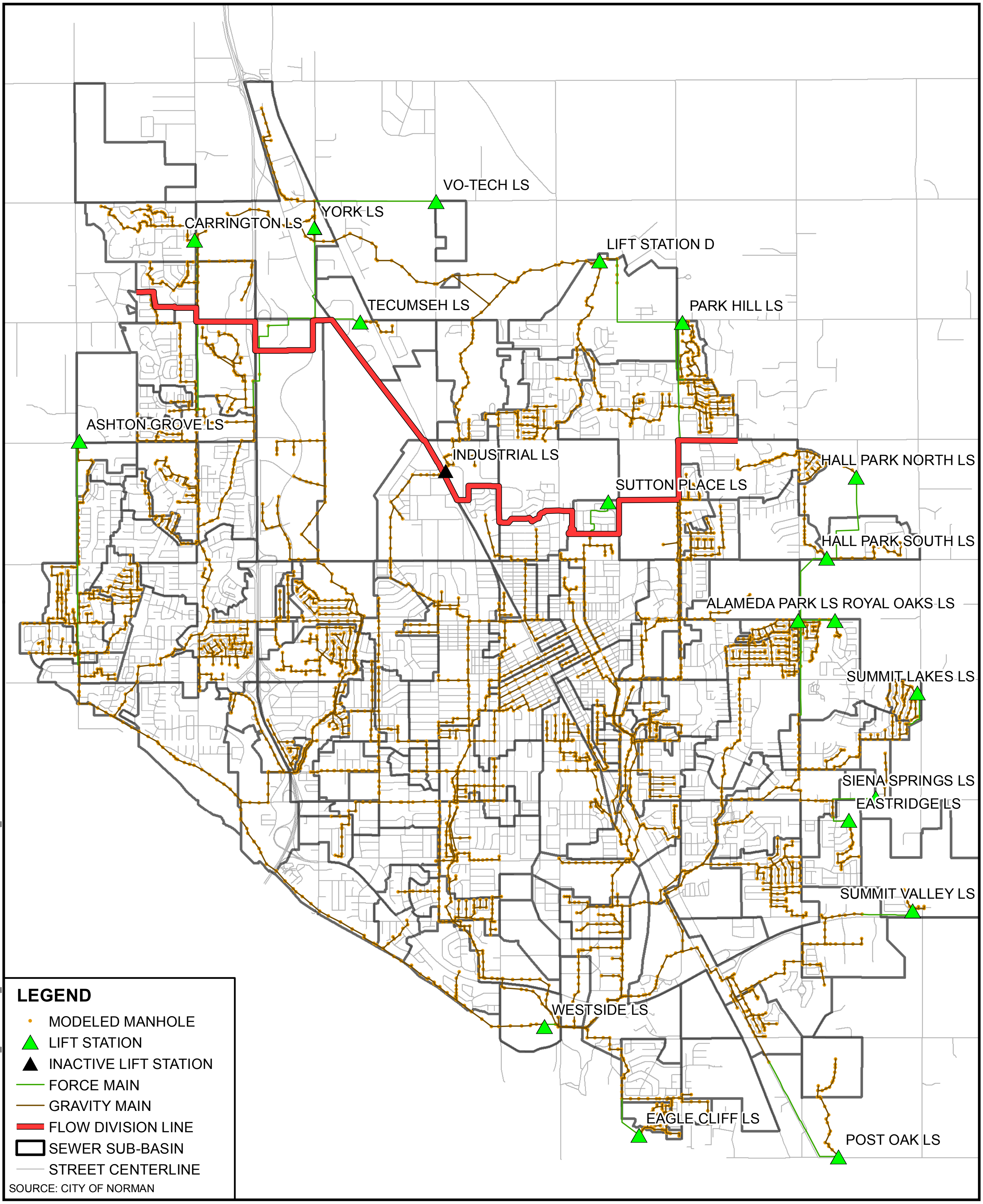
## 2.3 Model Physical Development

### 2.3.1 Previous Model

The City had a skeleton version of the collection system in the MIKE SWMM model platform by DHI software. In order to evaluate the capacity of the existing wastewater collection system, perform scenarios as part of the project scope, and to establish a tool for planning future expansion, a hydraulic model has been created using the InfoWorks™ software, developed by Innovyze. InfoWorks™ is a commercially available program that uses an integrated visual display to present system information and to report hydraulic capacity. The model was originally developed and utilized in InfoWorks™ CS Version 16.5.1 and then later converted to InfoWorks™ ICM 6.5.8 upon Innovyze's announcement that CS would no longer be supported.

### 2.3.2 Physical Development

The model was developed to account for pipes of 10-inch diameter or larger in the City's system. Where connectivity was necessary, smaller diameter pipes were included. In essence, laterals and mains are not included. Collectors taking flow from sub-basins and the trunk lines conveying to the Norman WRF are included. The City has a total of 11,339 pipe segments; the section of pipe between manhole nodes. Of these, 4,012 segments are included in the model, or 35%. Of 462 total miles of pipe, approximately 189 miles are included in the model, or 41%. The infrastructure included in the InfoWorks™ model is shown on **Figure 2-3**.




FILE: J:\MAP_DOCS\MXD\2005 PROJECT\PN139145_FIGURE1-3_MODELEDCOLLECTIONSYSTEM_8X11.MXD

**CITY OF NORMAN**
**FLOW MONITORING & MODELING**
MODELED COLLECTION SYSTEM

NORMAN UTILITIES AUTHORITY

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FIGURE 2-3

All Graphical Information System (GIS) electronic files were provided by City staff, including base maps, sewer basin boundaries, gravity pipes, force mains, land use and population data. The City also provided lift station data, pump curves when available, as-built information, and other corresponding data. These files provide the input data required to construct the model. Because the GIS files were available, there was no advantage to converting the skeleton model in the MIKE SWMM platform to InfoWorks™. The model was solely developed from the GIS files and data provided by the City as part of this work.

Connectivity was established for each pipe segment between upstream and downstream manholes. While manhole inverts were provided through GIS attribute data, manual manipulation was required to reassign inverts that were incorrect or missing information. This step was an iterative process, working with City staff to resolve all connectivity issues.

### 2.3.3 Sewer Basin Topology

The City has identified 21 sewer basins, divided further into 140 sub-basins, as shown on **Figure 2-2**. The City used the sub-basins to generate existing and future populations, as described below. HDR created a topology for the sewer basins to correct gaps or overlapping polygons in the GIS data that was provided. Any gaps or overlaps identified were manually corrected. Utilizing the flow monitoring data for the sewer basins, HDR was able to determine flow characteristics including sub-basin groundwater infiltration and rainfall-dependent infiltration and inflow.

## 2.4 Wastewater Loading

### 2.4.1 City Provided Density Data

The City provided data sets, reflective as of May 31, 2010, to be used for the wastewater loading in the model for the following service areas:

- Current Urban Service Area (CUSA)
    - Developed
    - Vacant Final Platted
    - Vacant Preliminary Platted
    - Vacant Unplatted
- Future Urban Service Area (FUSA)
    - Norman 2025 Plan Future Areas
    - Potential Future Extensions to Norman 2025 Plan

A land use approach was used for developing the model loading using the data sets. A combination of reviewing unit counts for residential land use and using acreage was used to develop the loading for the model, as further described below. The average wastewater demand provided by the City is 85 gallons per capita per day (gpcpd).

### 2.4.2 Unit Count Method

For the CUSA, with the exception of the vacant unplatted service area, the City determined the number of residential units and associated acreage for each residential land use type per sewer sub-basin. Unit densities contained in the data set are listed in **Table 2-1**. The unit counts were multiplied by the unit densities for each residential land use type to give a residential population per sub-basin. The residential population per sub-basin divided by the residential acreage for the sub-basin provides a demand density in persons/acre. For each sub-basin, the residential demand density was applied to parcels assigned with a residential land use. The residential demand density multiplied by the parcel acreage multiplied by 85 gpcpd provided the wastewater load contribution from that parcel. The wastewater load contribution was assigned to the nearest manhole in the model.

**Table 2-1. Residential Unit Density for CUSA**

| Land Use Type | Unit Density (persons/unit count)[1] |
|---|---|
| Low Density Residential (LDR) | 2.55 |
| Medium Density Residential (MDR) [2-4 units] | 1.96 |
| MDR (mobile home park) | 2.56 |
| High Density Residential (HDR) [5 or more] | 1.87 |
| Nursing Home | 1.00 |

[1]Data Source: US Census Bureau/American Community Survey (ACS) 5-Year Estimates 2006-2010 and the City of Norman Planning Department

### 2.4.3 Acreage Method

For non-residential land use in CUSA, all land uses in the vacant unplatted CUSA, all land uses in FUSA, and all land uses in FUSA extensions, the City determined the amount of acreage per land use type per sewer sub-basin. Unit densities contained in the data set are listed in **Table 2-2**. For each land use type, per sub-basin, the acreages were multiplied by the unit and summed to give a population per sub-basin. The population per sub-basin divided by the acreage for the sub-basin provides a demand density in persons/acre for each land use type. For each sub-basin, the demand density was applied to matching parcel land use type in the model. The demand density multiplied by the parcel acreage multiplied by 85 gpcpd provided the wastewater load contribution from that parcel. The wastewater load contribution was assigned to the nearest manhole in the model.

**Table 2-2. Land Use Unity Density for Non-residential CUSA and FUSA**

| Land Use Type | Unit Density (persons/acre)[1] |
|---|---|
| Low Density Residential (LDR) | 8.87 |
| Medium Density Residential (MDR) | 13.17 |
| High Density Residential (HDR) | 28.44 |
| Commercial | 5.00 |
| Office | 5.00 |
| Industrial | 10.00 |
| Institutional | 7.00 |
| Mixed Use (Municipal) | 14.38 |
| Very Low Density Residential (VLDR) | 1.275 |
| Commercial – Residential (CR) | 0.255 |

[1]Data Source: US Census Bureau/American Community Survey (ACS) 5-Year Estimates 2006-2010 and the City of Norman Planning Department

### 2.4.4 City Assumptions for Data Set Development

The following statements of understanding and assumptions were used by the City when developing the data sets:

1. Hotels loaded as commercial. Nursing homes and assisted living centers loaded as medium density residential (MDR) or high density residential (HDR) based on density using bed counts and parcel acreage. Dorms and hospitals loaded as institutional (no individual bed counts or occupancy rates were included for these areas).
2. Floodplain acreages were adjusted in the following ways, according to Development Status:
    a. For developed areas, floodplain was recoded to the primary land use of the parcel unless the parcel was a park or open space, in which case the floodplain was left alone.
    b. For vacant final platted, vacant preliminary platted, and vacant unplatted properties the floodplain was not adjusted, so that the amount of available developable land would not be artificially increased by the floodplain acreage. Future Areas include floodplain acreages as indicated in the *Norman 2025 Plan* and the latest FEMA maps.
3. House counts: Structures layer (2007) was supplemented by the inclusion of permitted structures from January 2005 through December 2009.
4. Aerial Photography used throughout this process was taken in March 2010.
5. Norman 2025 CUSA and FUSA are reflected as of May 31, 2010. Potential extensions to FUSA identified as of May 31, 2010.

6. Structures and parcel layers are reflected as of May 31, 2010.
7. Acreages highlighted in yellow in the data spreadsheet were adjusted to allow certain areas to be counted by total developed lots rather than by acreage. Acreages highlighted in green indicate where acreages were re-added to the table to reflect true total acreage for the sub-basin.
8. Population calculated using US Census Bureau/American Community Survey (ACS) 5-Year Estimates 2006-2010. Additional residential and mixed use population calculations from the *Norman 2025 Plan* were provided by the City of Norman Planning Department.

### 2.4.5 Adjusted Density Data

After review of the data sets, HDR adjusted the population and density within several sub-basins to be more accurately reflected in the model. The adjustments made include:

- Acreage Adjustment per Sub-Basin: The residential density land use categories were compared against the given land use acreage. If a residential land use category was assigned unit counts but not acreage, the unit counts were reallocated to a residential land use category that had acreage assigned. The population and density were then recalculated, effectively increasing the population. Since the model required population and density based loading, and not unit counts, this conversion was required.
- Nursing Homes: Unit or bed counts were provided for nursing homes. However, when determining population for the sub-basin, the nursing homes were included in the high density residential total. This skews the loading in the model. To more accurately model the nursing home load contribution, HDR identified the location of the nursing homes, removed the population from the high density land use category and applied the density directly to the parcel as a point load. The density for the high density land use category was recalculated and applied in the model.
- Norman Housing Authority: The City included land owned by the Norman Housing Authority in the Nursing Home land use category. For simplicity and due to its similarities, the loading was allocated in the model as high density residential.

The reallocation was verified in two ways. First, a revised density calculation was made and compared to the City provided density to make sure the value was not significantly skewed. Second, the total population using the reallocated load was compared to the original data set total population. The percent difference calculated was less than 0.5% for each case (1.5% total), validating the reallocation.

### 2.4.6 Wastewater Loading

**Table 2-3** presents the wastewater loading that is used as the baseline condition in the model (base model file). The wastewater loading was applied on a sub-basin basis in the model; however, due to the large number of sub-basins, the wastewater loading data shown in **Table 2-3** has been summarized per sewer basin for the report discussion.

**Table 2-3. Land-use Based Wastewater Loading**

| Sewer Basin | Land Use Based Load Totals for Developed CUSA[1] (mgd) | | | | |
|---|---|---|---|---|---|
| | Residential[2] | Commercial | Industrial | Institutional | TOTAL |
| Lift Station D | 0.56 | 0.01 | 0.07 | 0.05 | 0.70 |
| Park Hill | 0.02 | 0.00 | 0.00 | 0.00 | 0.02 |
| Tecumseh | 0.00 | 0.00 | 0.08 | 0.00 | 0.08 |
| Vo-Tech | 0.00 | 0.00 | 0.00 | 0.02 | 0.02 |
| York | 0.00 | 0.00 | 0.09 | 0.04 | 0.13 |
| **Subtotal North Basins** | **0.58** | **0.01** | **0.24** | **0.12** | **0.95** |
| Alameda Park | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Ashton Grove | 0.10 | 0.00 | 0.00 | 0.00 | 0.10 |
| Bishop | 3.13 | 0.11 | 0.16 | 0.36 | 3.77 |
| Brookhaven | 1.06 | 0.09 | 0.04 | 0.04 | 1.23 |
| Eagle Cliff | 0.01 | 0.00 | 0.00 | 0.00 | 0.01 |
| East Ridge | 0.08 | 0.00 | 0.00 | 0.00 | 0.08 |
| Hall Park | 0.09 | 0.00 | 0.00 | 0.00 | 0.09 |
| Imhoff | 2.51 | 0.08 | 0.06 | 0.22 | 2.88 |
| Normandy | 0.24 | 0.09 | 0.02 | 0.02 | 0.36 |
| Post Oak | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Royal Oaks | 0.10 | 0.00 | 0.00 | 0.00 | 0.10 |
| Siena Springs | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Summit Lakes | 0.01 | 0.00 | 0.00 | 0.00 | 0.01 |
| Summit Valley | 0.01 | 0.00 | 0.00 | 0.00 | 0.01 |
| Sutton Place | 0.03 | 0.00 | 0.00 | 0.00 | 0.03 |
| Westside | 0.40 | 0.02 | 0.00 | 0.04 | 0.46 |
| **Subtotal South Basins** | **7.77** | **0.39** | **0.29** | **0.68** | **9.14** |
| **Total All Basins** | **8.94** | **0.42** | **0.76** | **0.92** | **11.04** |

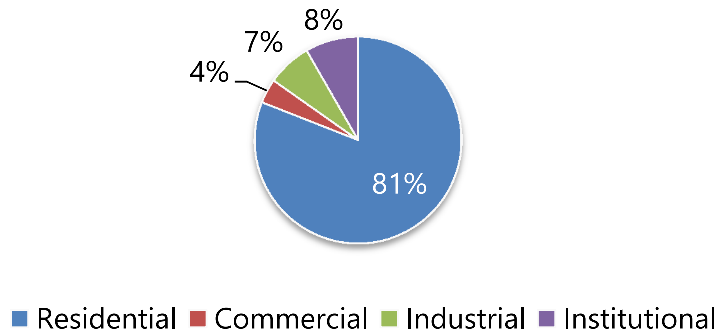
[1]Includes the developed platted lots only. Vacant lots are included in the FBO load allocation.
[2]Residential load includes all residential categories, mobile home parks, and nursing homes.

# 2.5 Model Flow Components

## 2.5.1 Sanitary Flow

The total flow in a sanitary sewer system originates from several different sources. Wastewater discharges from municipal sources typically comprise the majority of the total load. Other sources of flow include infiltration of groundwater and inflow from storm events, often referred to together as I&I. Most of the City's wastewater discharge comes from sources designated as residential land use, as shown in **Figure 2-4**.

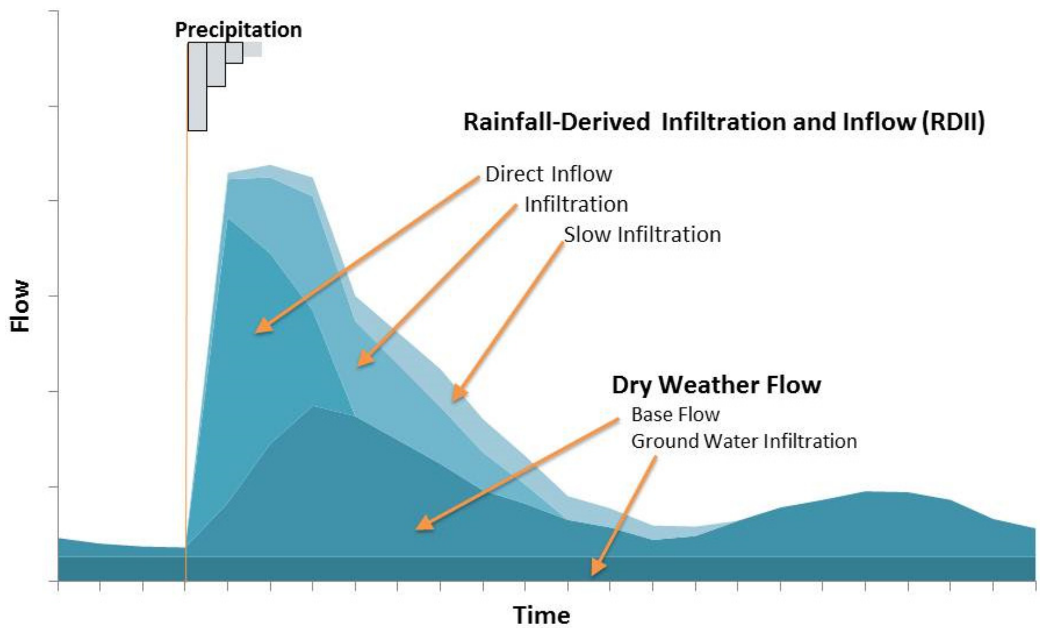
### Figure 2-4. Wastewater Discharge Sources




From May 1 to September 15, 2010, flow monitoring was conducted by the RJN Group (RJN) for the purpose of measuring dry weather and wet weather flow from the sewer basins. Results of the flow monitoring are presented under separate cover in the *Wastewater Flow Monitoring Report* (RJN, November 2010). **Appendix 2A** contains a table listing the flow meters and the sub-basins that contributed to each flow meter.

The flow metering data was analyzed to determine base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall dependent infiltration and inflow (RDII) in order to perform model calibration. The flow components are illustrated on **Figure 2-5**.

### Figure 2-5. Flow Component Illustration



### 2.5.2 Average and Discrete Flows

Average flows were calculated based on evaluating dry weekday days observed in the flow meter data and taking an average value of the measured flow over the monitoring period. Discrete flows for each metered sewer basin were calculated based by taking the average dry weather flow at each meter and subtracting the upstream metered flows. Therefore, discrete flows indicate the characteristics of the individual sewer basin at a given meter location. The calculated average discrete flows are summarized in **Table 2-4**. In some instanced, including BP-20 and BP-24, the discrete basin flows were calculated to be negative. Likely sources of error include flow transfers via bypass piping near the flow monitor locations, timing delays, and timestep differences between the meters and not the actual contribution of that basin.

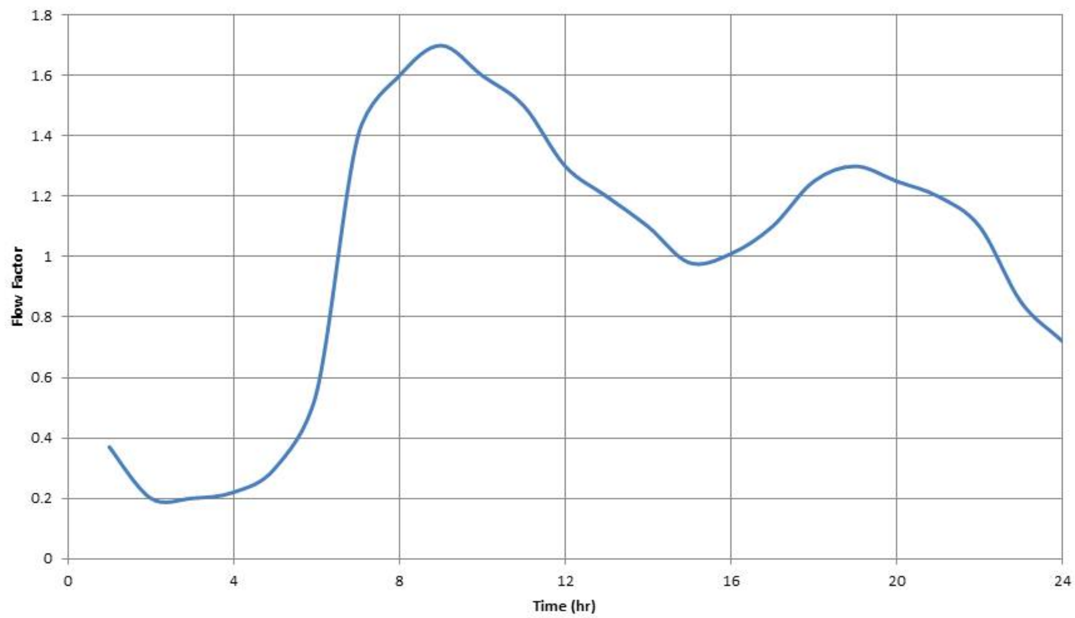
## Table 2-4. Average and Discrete Flows from Flow Monitoring Data

| Flow Monitor | Average Daily Flow (mgd) | Discrete Basin Average Daily Flow (mgd) |
|---|---|---|
| BH-02 | 1.99 | 0.28 |
| BH-03 | 0.31 | 0.31 |
| BH-04 | 1.40 | 0.56 |
| BH-05 | 0.83 | 0.51 |
| BH-06 | 0.32 | 0.11 |
| BH-07 | 0.21 | 0.21 |
| BP-17 | 2.82 | 0.24 |
| BP-18 | 0.58 | 0.23 |
| BP-17/18 Combined | 3.40 | 0.46 |
| BP-19 | 0.65 | 0.65 |
| BP-20 | 0.11 | -0.20 |
| BP-21 | 1.82 | 0.67 |
| BP-22 | 0.32 | 0.32 |
| BP-23 | 1.14 | 0.59 |
| BP-24 | 0.35 | -0.21 |
| BP-25 | 2.10 | 1.53 |
| BP-26 | 0.57 | 0.57 |
| BP-27 | 0.56 | 0.56 |
| BP-28 | 0.37 | 0.37 |
| BP-29 | 0.13 | 0.13 |
| IH-12 | 0.61 | 0.40 |
| IH-13 | 0.20 | 0.20 |
| IH-14 | 1.13 | 0.32 |
| IH-15 | 0.81 | 0.44 |
| IH-16 | 0.38 | 0.38 |
| ND-08 | 0.69 | 0.36 |
| ND-09 | 0.34 | 0.34 |
| WC-30 | 0.52 | 0.29 |
| WC-31 | 0.23 | 0.23 |
| WC-32 | 0.03 | 0.03 |
| WS-01 | 2.88 | 0.20 |
| WS-10 | 0.73 | 0.12 |
| WS-11 | 1.63 | 0.50 |
| **Total All Basins** | | **10.88** |

### 2.5.3 Base Wastewater Flow (BWF)

Base wastewater flow originates from residential, commercial, industrial, and institutional facilities and are discharged into the sewer system. Unlike infiltration, which enters the collection system at a constant rate throughout the day, BWF typically exhibits a diurnal flow pattern similar to the one shown in **Figure 2-6**.

**Figure 2-6. Typical Residential Diurnal Flow Pattern**




Diurnal flow patterns were developed for each land use category by evaluating the flow monitoring data, water usage of the top 25 water users in the service area, and typical industry standards for the given land use types. A residential diurnal flow pattern was assigned in the model to all parcels with a residential, mobile home park and nursing home land use designation; a commercial diurnal flow pattern was assigned in the model to all parcels with commercial land use designations, and so forth. An exception to this methodology was made to three industrial sites that are listed within the top 25 water users of the service area. Information about flow contribution, shift work and water usage was obtained from the City which allowed site specific diurnal flow patterns to be developed allowing for a more accurate representation of this loading in the model. **Appendix 2B** contains the diurnal flow patterns developed for the land use categories that are applied in the model.

### 2.5.4 Groundwater Infiltration (GWI)

Groundwater infiltration is defined as the constant inflow of ground water into the collection system irrespective of rainfall availability. This value is determined by several factors including the location of the groundwater table and the overall physical condition of the collection system. Furthermore, GWI can also be impacted by periods of wet and dry weather conditions.

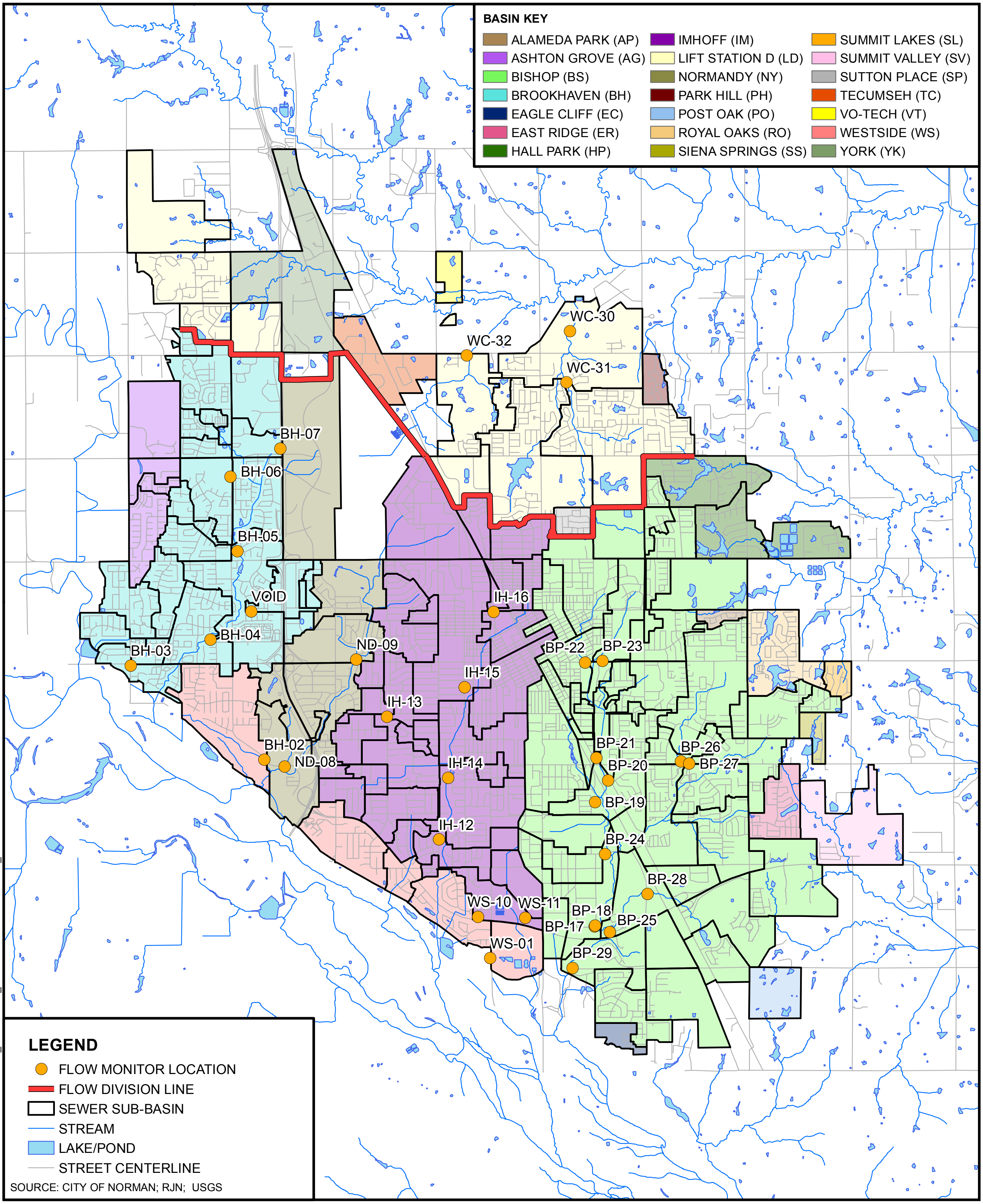
For wastewater collection systems, such as Norman, that do not typically experience substantial industrial and/or commercial flows throughout a given 24-hour period, GWI is equated to collection system minimum flows that occur during the late night and early morning hours as measured by the flow monitors. Several factors, including the presence of high groundwater, proximity to lakes and streams, and the predominant land use within each sewer basin help to determine if these flows are indicative of groundwater flow. The data is summarized in **Table 2-5**. **Figure 2-7** illustrates the relationship of sewer basins to stream locations.

**Table 2-5. Groundwater Infiltration Analysis**

| Flow Monitor | Discrete Average Daily Flow (mgd) | Groundwater Infiltration (mgd) | Percent of Discrete Average Daily Flow | Predominant Land Use | Probability of High Groundwater or Proximity to Lakes or Streams[1] |
|---|---|---|---|---|---|
| BH-02 | 0.28 | 0.10 | 36% | Residential | Low to Moderate |
| BH-03 | 0.31 | 0.03 | 10% | Residential | Low |
| BH-04 | 0.56 | 0.27 | 49% | Commercial | Low to Moderate |
| BH-05 | 0.51 | 0.17 | 33% | Residential | Low to Moderate |
| BH-06 | 0.11 | 0.04 | 31% | Residential | Low |
| BH-07 | 0.21 | 0.04 | 17% | Industrial | Low to Moderate |
| BP-17 | 0.24 | 0.22 | 92% | Institutional | Low |
| BP-18 | 0.23 | 0.22 | 98% | Institutional | Low |
| BP-17/18 Combined | 0.46 | 0.44 | 95% | Institutional | Low |
| BP-19 | 0.65 | 0.26 | 41% | Institutional | Low to Moderate |
| BP-202 | -0.20 | -0.06 | 32% | Residential | Low |
| BP-21 | 0.67 | 0.30 | 44% | Residential | Low |
| BP-22 | 0.32 | 0.09 | 28% | Residential | Low |
| BP-23 | 0.59 | 0.08 | 13% | Residential | Low |
| BP-242 | -0.21 | 0.00 | 0% | Residential | Low |
| BP-25 | 1.53 | 0.60 | 39% | Residential | Low |
| BP-26 | 0.57 | 0.35 | 61% | Residential | Low |
| BP-27 | 0.56 | 0.15 | 27% | Residential | Low |
| BP-28 | 0.37 | 0.16 | 43% | Residential | Low |
| BP-29 | 0.13 | 0.02 | 18% | Residential | Low |

| Flow Monitor | Discrete Average Daily Flow (mgd) | Groundwater Infiltration (mgd) | Percent of Discrete Average Daily Flow | Predominant Land Use | Probability of High Groundwater or Proximity to Lakes or Streams[1] |
|---|---|---|---|---|---|
| IH-12 | 0.40 | 0.13 | 32% | Residential | Low |
| IH-13 | 0.20 | 0.07 | 37% | Residential | Low |
| IH-14 | 0.32 | 0.07 | 20% | Residential | Low |
| IH-15 | 0.44 | 0.28 | 64% | Industrial | Low |
| IH-16 | 0.38 | 0.05 | 14% | Residential | Low |
| ND-08 | 0.36 | 0.16 | 44% | Residential | Moderate to High |
| ND-09 | 0.34 | 0.12 | 37% | Residential | Low |
| WC-30 | 0.29 | 0.10 | 34% | Residential | Moderate to High |
| WC-31 | 0.23 | 0.07 | 31% | Residential | Low |
| WC-32 | 0.03 | 0.00 | 17% | Residential | Low |
| WS-013 | 0.20 | -0.40 | | Residential | Low |
| WS-10 | 0.12 | 0.11 | 92% | Institutional | Low |
| WS-11 | 0.50 | 0.60 | 121% | Residential | Low |
| **Total All Basins** | **10.88** | **4.85** | | | |

[1]Qualitative review only based on Figure 2-7 and calculation of flood plain acreage by basin.
[2]Discrete minimum flows could not be effectively determined based on likely impacts from bypass piping and flow routing.
[3]WS-01 is the flow monitor that received all flow from the western portion of the system. Due to the large volume of upstream flow at this meter, local flows, particularly groundwater infiltration, could not be determined.




FILE: J:\MAP_DOCS\MXD\2005 PROJECT\PN139145_FIGURE1-7_HYDROLOGICFEATURES_8X11.MXD

**CITY OF NORMAN**
**FLOW MONITORING & MODELING**
HYDROLOGIC FEATURES

N
0 0.5 1
MILES

NORMAN UTILITIES AUTHORITY

HDR ENGINEERING, INC.
17111 PRESTON RD. SUITE 300
DALLAS, TX 75248-1232
972-960-4400

FIGURE 2-7

### 2.5.5 Rainfall-Dependent Infiltration and Inflow (RDII)

Rainfall-dependent infiltration and inflow represents the total wet weather flow that enters the collection system in the form of inflow and infiltration.

- Inflow is water that enters the sewer system relatively quickly after a wet weather event begins and recedes quickly once the rain event ends. It occurs through unintended pathways such as depressed manhole covers, holes in manhole covers, down spouts connected to the system, cross connections, etc.
- Infiltration occurs when water that is contained in the soil enters the pipe through leaking joints, damaged pipe sections, or poor connections to manholes. Infiltration typically continues well past the end of a wet weather event.

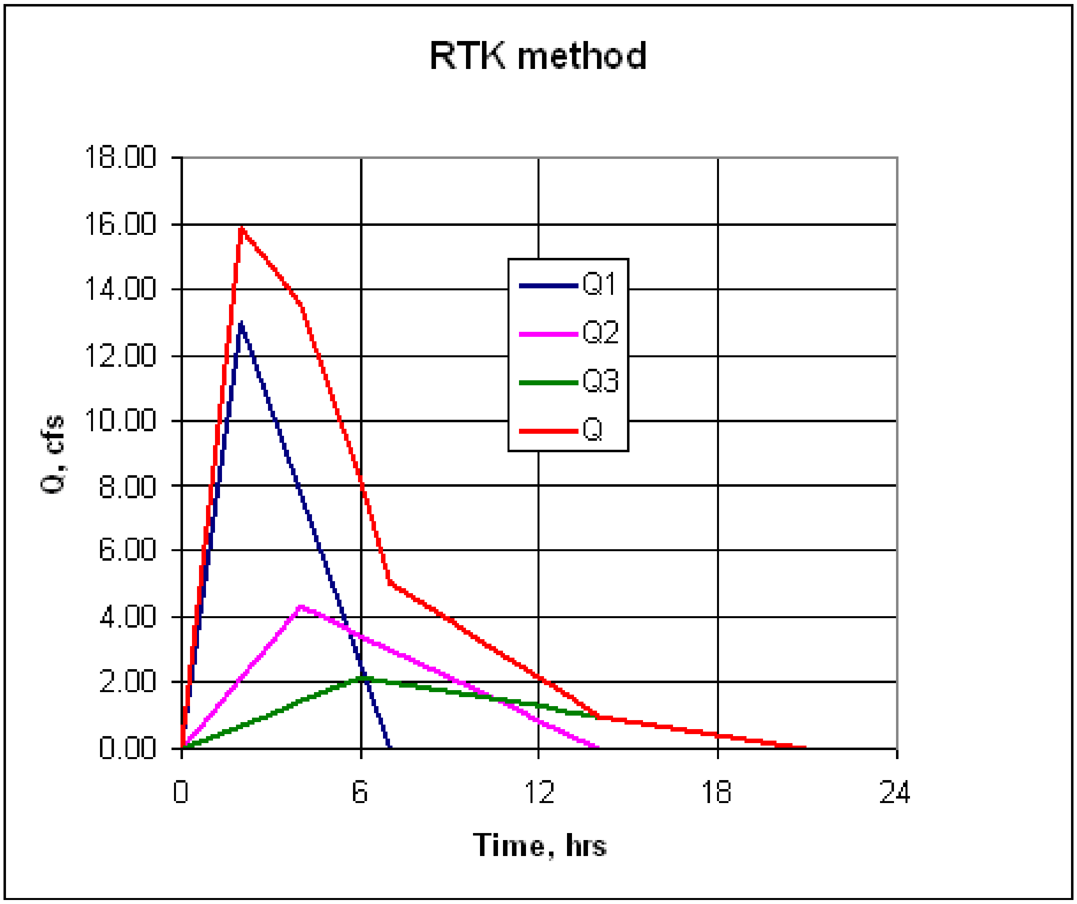
The RTK unit hydrograph method is used to derive the RDII response of the sewer system based on the collected rainfall and flow monitoring data. This method is based on fitting three triangular unit hydrographs to an actual RDII hydrograph derived from the flow meter data. R, T, and K factors are developed for each triangular unit hydrograph and represent the following:

- R is the percentage of rainfall that entered the system within each sewer basin during the monitoring period.
- T, the time of concentration, is the time difference between when the rainfall started and when the peak flow hydrograph was observed at each of the monitoring locations.
- K, the recession coefficient, is the ratio between the peak of the flow hydrograph and the point at which the RDII receded.

The three triangular unit hydrographs are defined as follows, and an example hydrograph is shown in **Figure 2-8**:

- $Q_1$ represents the initial event response and includes some inflow from sources that are directly connected to the sewer system including roof downspouts, leaking manholes and pipeline cross-connections. T values for this hydrograph are generally on the order of 0.5 to 2 hours.
- $Q_2$ includes primarily RDII and infiltration, with longer T values, generally from 2.5 to 5 hours.
- $Q_3$ includes infiltration that occurs long after the event is over and represents a long soaking storm, since infiltration would slowly seep into the collection system. T values for this are generally in the 5 to 10 hour range.

**Figure 2-8. Example RTK Hydrograph**




Eight rainfall events of greater than 1 inch of rainfall in a 24-hour period were recorded during the flow monitoring period for this project. Ultimately, the rainfall received during the Tropical Storm Hermine event that occurred from September 7-9, 2010, was used to develop the RTK hydrographs for wet weather calibration. This storm was selected due to the intensity of the storm and a duration that was suitable for calibration purposes. Doppler data was obtained to verify rain gauge information for each basin.

RTK hydrographs were developed based on basins containing similar land use breakdowns and included similar percentages of floodplain. The R, T, K factors developed are listed in **Table 2-6**.

**Table 2-6. RDII Hydrograph Parameters**

| Flow Monitor | Initial Inflow Response ($Q_1$) | | | RDII Response ($Q_2$) | | | Post-Storm Infiltration Response ($Q_3$) | | |
|---|---|---|---|---|---|---|---|---|---|
| | $R_1$ | $T_1$ | $K_1$ | $R_2$ | $T_2$ | $K_2$ | $R_3$ | $T_3$ | $K_3$ |
| BH-02 | 0.0016 | 1.5 | 0.8 | 0.0022 | 4 | 0.8 | 0.009 | 4.5 | 1.5 |
| BH-03 | 0.005 | 0.5 | 1 | 0.007 | 3 | 2 | 0.006 | 7 | 3 |
| BH-04 | 0.005 | 1 | 1 | 0.007 | 1.2 | 2 | 0.005 | 5 | 2 |
| BH-05 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BH-06 | 0.0008 | 1.8 | 1 | 0.0004 | 1.8 | 2.5 | 0.0002 | 3 | 2 |
| BH-07 | 0.005 | 1 | 1 | 0.007 | 1.2 | 2 | 0.005 | 5 | 2 |
| BP-17 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BP-18 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BP-19 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BP-20 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BP-21 | 0.0008 | 1.8 | 1 | 0.0004 | 1.8 | 2.5 | 0.0002 | 3 | 2 |
| BP-22 | 0.0008 | 1.8 | 1 | 0.0004 | 1.8 | 2.5 | 0.0002 | 3 | 2 |
| BP-23 | 0.0008 | 1.8 | 1 | 0.0004 | 1.8 | 2.5 | 0.0002 | 3 | 2 |
| BP-24 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BP-25 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BP-26 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BP-27 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BP-28 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| BP-29 | 0.0004 | 0.9 | 2 | 0.0009 | 4 | 1 | 0.0005 | 4.5 | 1.5 |
| IH-12 | 0.005 | 0.5 | 1 | 0.007 | 3 | 2 | 0.006 | 7 | 3 |
| IH-13 | 0.0008 | 1.8 | 1 | 0.0004 | 1.8 | 2.5 | 0.0002 | 3 | 2 |
| IH-14 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| IH-15 | 0.003 | 1.7 | 1.5 | 0.0025 | 3.5 | 2.5 | 0.0014 | 4 | 3 |
| IH-16 | 0.005 | 0.5 | 1 | 0.007 | 3 | 2 | 0.006 | 7 | 3 |
| ND-08 | 0.0016 | 1.5 | 0.8 | 0.0022 | 4 | 0.8 | 0.009 | 4.5 | 1.5 |
| ND-09 | 0.005 | 1 | 1 | 0.007 | 1.2 | 2 | 0.005 | 5 | 2 |
| WC-30 | 0.0008 | 1.8 | 1 | 0.0004 | 1.8 | 2.5 | 0.0002 | 3 | 2 |
| WC-31 | 0.0016 | 1.5 | 0.8 | 0.0022 | 4 | 0.8 | 0.009 | 4.5 | 1.5 |
| WC-32 | 0.0013 | 1.5 | 0.5 | 0.0013 | 4.9 | 2 | 0.0008 | 5 | 5 |
| WS-01 | 0.005 | 0.5 | 1 | 0.007 | 3 | 2 | 0.006 | 7 | 3 |
| WS-10 | 0.005 | 0.5 | 1 | 0.007 | 3 | 2 | 0.006 | 7 | 3 |
| WS-11 | 0.005 | 0.5 | 1 | 0.007 | 3 | 2 | 0.006 | 7 | 3 |

### 2.5.6 Peaking Flow

As described above, a flow pattern was determined for each land use type. The flow pattern was assigned to the matching land use type in the system model. As a time-step analysis is performed, the wastewater loading is adjusted for each parcel by the flow factor from the assigned pattern at the given time step. As an example, refer to the residential flow pattern shown in **Figure 2-6**. For example, given a residential parcel, the wastewater loading will be adjusted by 1.6 times at 8:00 a.m., 1.3 times at noon, remain unadjusted at 4:00 p.m., adjusted by 1.25 times at 8:00 p.m., and so forth.

## 2.6 Calibration

The wastewater model was calibrated for both dry and wet weather flows:

- Dry weather calibration is a process of reducing the amount of error between the wastewater loading (described in Section 2.3) when compared to the BWF component derived by analyzing the flow meter data (described in Section 2.4).
- Wet weather calibration is a process of reducing the amount of error between the flow volume observed when the RTK hydrographs are applied to the sub-basins in the model versus the actual flow volume measured during the wet weather event.

Both are iterative processes to attempt to reduce the amount of error to 10% or less between observed and measured flows.

### 2.6.1 Dry Weather Flow

**Table 2-7** shows the uncalibrated flow comparison between the model and the measured flow monitoring values. There were 27 flow monitoring locations whose modeled percentage of observed flows were outside the ±10% calibration target. These are the raw results comparing the discrete flow monitoring data to the model data (after initial loading but prior to calibration).

**Table 2-8** presents the final calibrated results for dry weather flow of the base model achieved by iteratively adjusting the factors in the model. **Appendix 2C** shows the pre-calibration and post-calibration dry weather hydrographs for each flow monitor.

## Table 2-7: Uncalibrated Dry Weather Flow Comparison

| Flow Monitor | Measured Discrete Daily Flow (mgd) | Modeled Discrete Daily Flow (mgd) | Percent of Measured Discrete Daily Flow (%) |
|---|---|---|---|
| BH-02 | 0.28 | 0.21 | -25% |
| BH-03 | 0.31 | 0.32 | 1% |
| BH-04 | 0.56 | 0.40 | -30% |
| BH-05 | 0.51 | 0.40 | -23% |
| BH-06 | 0.11 | 0.07 | -35% |
| BH-07 | 0.21 | 0.14 | -31% |
| BP-17 | 0.24 | 0.02 | -92% |
| BP-18 | 0.23 | 0.80 | 257% |
| BP-19 | 0.46 | 0.31 | -33% |
| BP-20 | 0.65 | 0.24 | -62% |
| BP-21 | -0.20 | -0.61 | 197% |
| BP-22 | 0.67 | 0.46 | -32% |
| BP-23 | 0.32 | 0.23 | -29% |
| BP-24 | 0.59 | 0.53 | -10% |
| BP-25 | -0.21 | -0.16 | -25% |
| BP-26 | 1.53 | 0.94 | -39% |
| BP-27 | 0.57 | 0.38 | -34% |
| BP-28 | 0.56 | 0.46 | -18% |
| BP-29 | 0.37 | 1.22 | 232% |
| IH-12 | 0.13 | 0.15 | 13% |
| IH-13 | 0.40 | 0.30 | -25% |
| IH-14 | 0.20 | 0.15 | -24% |
| IH-15 | 0.32 | 0.32 | 0% |
| IH-16 | 0.44 | 0.32 | -27% |
| ND-08 | 0.38 | 0.22 | -42% |
| ND-09 | 0.36 | 0.16 | -55% |
| WC-30 | 0.34 | 0.14 | -57% |
| WC-31 | 0.29 | 0.26 | -9% |
| WC-32 | 0.23 | 0.26 | 9% |
| WS-01 | 0.03 | 0.09 | 224% |
| WS-10 | 0.20 | 0.13 | -31% |
| WS-11 | 0.12 | 0.10 | -21% |

## Table 2-8. Calibrated Dry Weather Flow Comparison

| Flow Monitor | Measured Discrete Daily Flow (mgd) | Modeled Discrete Daily Flow (mgd) | Percent of Measured Discrete Daily Flow (%) |
|---|---|---|---|
| BH-02 | 0.28 | 0.30 | 6% |
| BH-03 | 0.31 | 0.32 | 2% |
| BH-04 | 0.56 | 0.60 | 6% |
| BH-05 | 0.51 | 0.48 | -6% |
| BH-06 | 0.11 | 0.10 | -12% |
| BH-07 | 0.21 | 0.20 | -2% |
| BP-17 | 0.24 | 0.25 | 5% |
| BP-18 | 0.23 | 0.25 | 11% |
| BP-19 | 0.46 | 0.50 | 8% |
| BP-20 | 0.65 | 0.70 | 8% |
| BP-21 | -0.20 | 0.00 | -100% |
| BP-22 | 0.67 | 0.60 | -11% |
| BP-23 | 0.32 | 0.35 | 10% |
| BP-24 | 0.59 | 0.60 | 1% |
| BP-25 | -0.21 | 0.00 | -100% |
| BP-26 | 1.53 | 1.54 | 1% |
| BP-27 | 0.57 | 0.58 | 2% |
| BP-28 | 0.56 | 0.58 | 3% |
| BP-29 | 0.37 | 0.41 | 11% |
| IH-12 | 0.13 | 0.12 | -9% |
| IH-13 | 0.40 | 0.41 | 1% |
| IH-14 | 0.20 | 0.20 | -1% |
| IH-15 | 0.32 | 0.32 | 0% |
| IH-16 | 0.44 | 0.48 | 10% |
| ND-08 | 0.38 | 0.40 | 6% |
| ND-09 | 0.36 | 0.34 | -4% |
| WC-30 | 0.34 | 0.34 | 1% |
| WC-31 | 0.29 | 0.31 | 8% |
| WC-32 | 0.23 | 0.24 | 2% |
| WS-01 | 0.03 | 0.03 | 5% |
| WS-10 | 0.20 | 0.21 | 7% |
| WS-11 | 0.12 | 0.13 | 6% |
| **Total All Basins** | **11.21** | **11.89** | **6%** |

The resulting calibration reduced the difference between the model flows and the flow monitoring data; however, by adjusting the upstream flows in the calibration process, the downstream flows did not match as closely as they initially did. This is to be expected with the calibration process; as the upstream flows are more closely matched, the downstream flows are adversely altered.

Possible reasons for the flow monitoring locations that still do not match within the target range after calibration include:

- Since some of the flow monitors had low average daily dry-weather flows (<0.2 mgd), the percent error between the measured and modeled flow appears a lot greater than the volume difference really is.
- The flow monitoring data could have been off as much as ±4% itself due to equipment accuracy. This can make the comparison off from the start. This error is especially possible for flow monitors with low flows.
- The dry-weather period may have not been representative of average dry-weather flows experienced over an entire year. Certain areas of the system may not match as well as others due to the land use composition and percent occupancy at the time of flow monitoring and may have caused the flows to be abnormally low or high during the selected flow monitoring time period.
- Seasonal effects on wastewater flow from groundwater-induced infiltration along creeks and other areas with high groundwater can affect the dry-weather calibration. Since the characterization of the effects of groundwater on the system is largely unknown, for this model the groundwater-induced base infiltration was assigned globally across the system within the sanitary unit flow factors. However, over different seasons and depending on recent storm events, groundwater levels can fluctuate and not be accounted entirely in the unit flow factors.

The WRF influent flow records match within 10% and are lower than the summation of flow monitors, which is expected due to flow attenuation in the system downstream of the contributing flow monitoring locations. The flows are still higher than the recorded flows at the WRF.

The overall difference between model results and temporary flow meter data is considered adequate for this planning study given 1) the conservative nature of the model results compared to WRF influent data 2) the relatively small amount of flow under scrutiny, compared to system total, 3) overall system calibration at the WRF is within standard modeling tolerances, and 4) the majority of the flow monitoring locations are within the calibration target.

### 2.6.2 Wet Weather Flows

As discussed previously, the RTK method was used to develop wet weather hydrographs to match the measured storm event on September 7-9, 2010. Five RTK hydrographs were developed based on land use and assigned to similar land uses across the service area.

Calibration was achieved by adjusting the infiltration percentage for each land use and slope and width for each sub-basin so that the modeled wet weather response closely approximated the observed wet weather response. **Table 2-9** presents the final calibrated results for wet weather flow of the base model. **Appendix 2D** shows the calibrated wet weather hydrographs for each flow monitor.

Where discrete RDII flows were less than average discrete dry weather flows, or negative in value, RDII was modeled at zero. This primarily occurred in basins with large incoming flows but with a very small contribution to that flow. So while the overall modeled flows corresponded well to the observed data, smaller basins would result in having very little impact to the wet weather flow leaving the basin. And because negative flow cannot be added to the model, the discrete wet weather flow components were modeled as zero. This results in a large difference in measured discrete and modeled discrete flows based on totals. If the negative values are removed, the margin of error is much smaller (2%).

### Table 2-9. Calibrated Wet Weather Flow Comparison

| Flow Monitor | Measured Discrete RDII Flow (mgd) | Modeled Discrete Peak Wet Weather Flow (mgd) | Percent of Measured Discrete RDII Flow (%) |
|---|---|---|---|
| BH-03 | 0.08 | 0.08 | 0% |
| BH-04 | -0.02 | 0.00 | N/A |
| BH-05 | 0.14 | 0.13 | -7% |
| BH-06 | 0.00 | 0.00 | 0% |
| BH-07 | 0.09 | 0.09 | -2% |
| BP-17 | 0.95 | 0.94 | -1% |
| BP-18 | -0.09 | 0.00 | N/A |
| BP-19 | 0.86 | 0.85 | -1% |
| BP-20 | 0.11 | 0.10 | -6% |
| BP-21 | -0.17 | 0.00 | N/A |
| BP-22 | 0.44 | 0.43 | -3% |
| BP-23 | 0.16 | 0.16 | -2% |
| BP-24 | -0.01 | 0.00 | N/A |
| BP-25 | 0.15 | 0.15 | 0% |
| BP-26 | -0.28 | 0.00 | N/A |
| BP-27 | 0.18 | 0.19 | 7% |
| BP-28 | -0.04 | 0.00 | N/A |
| BP-29 | 0.06 | 0.07 | 20% |
| IH-12 | -0.02 | 0.00 | N/A |
| IH-13 | 0.24 | 0.25 | 3% |
| IH-14 | -0.01 | 0.00 | N/A |
| IH-15 | 0.56 | 0.52 | -8% |
| IH-16 | -0.20 | 0.00 | N/A |
| ND-08 | 0.16 | 0.16 | 1% |
| ND-09 | -0.11 | 0.00 | N/A |
| WC-30 | 0.09 | 0.09 | 5% |
| WC-31 | -0.03 | 0.00 | N/A |
| WC-32 | 0.07 | 0.07 | 0% |
| WS-01 | 0.00 | 0.00 | 0% |
| WS-10 | 0.04 | 0.04 | -2% |
| WS-11 | -0.01 | 0.00 | 0% |
| **Total All Basins (with negative values)** | **3.38** | **4.31** | **22%** |
| **Total All Basins (without negative values)** | **4.38** | **4.31** | **2%** |

# 3 Collection System Capacity Analysis

The purpose of this chapter is to document the collection system capacity analysis and infiltration and inflow (I/I) evaluation. This chapter includes a description of the modeling criteria used, identification of model-predicted overflow locations at full build-out 2030 loading conditions (FBO 2030) and a comparison of modeling results to the previous Wastewater System Master Plan (*Master Plan*, CDM, September 2001) to generally ascertain the effectiveness of I/I reduction based on sewer improvements the City has implemented.

## 3.1 System Analysis Criteria

### 3.1.1 Sewer Infrastructure

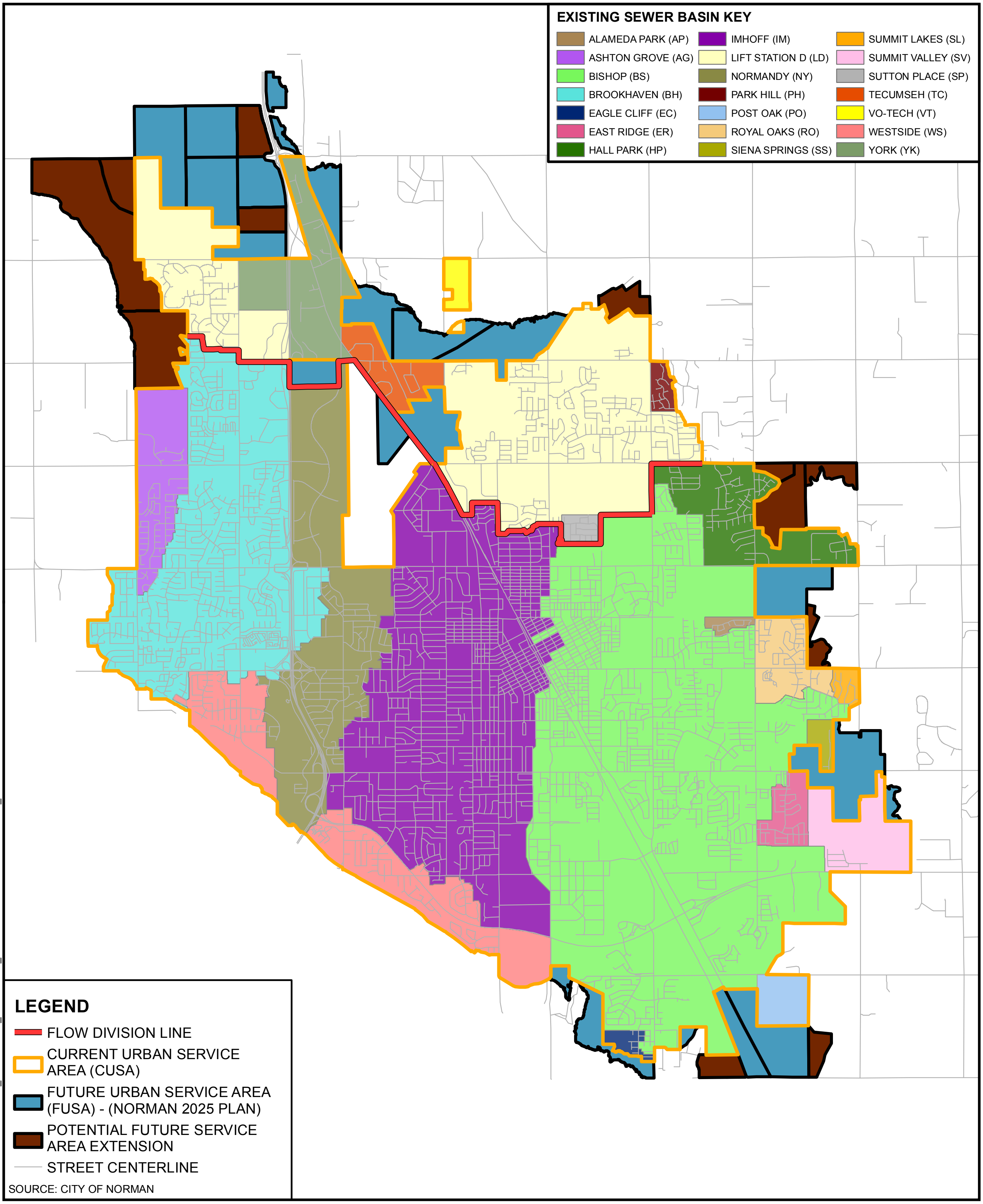
Chapter 2 describes the land use, service areas, wastewater loading, and infrastructure data (i.e., pipes, manholes, and lift stations) used in model development and calibration. The model includes all pipes with a diameter of 10-inches and larger and was calibrated to the existing sewer system infrastructure reflective of May 2010 to coincide with the flow monitoring period. After the model was successfully calibrated, significant system improvements made since May 2010 were incorporated into the model. This updated model (circa April 2012) was used to conduct the capacity analysis.

### 3.1.2 Service Areas

As described in Chapter 2, the City has three discrete service areas: the Current Urban Service Area (CUSA), the Future Urban Service Area (FUSA), and the Future Urban Service Area Extensions (FUSA-EXT) which are shown on **Figure 3-1**. The CUSA includes the developed, vacant final platted, vacant preliminary platted, and vacant unplatted areas and is generally serviced with the existing sewer infrastructure. The FUSA includes the *Norman 2025 Plan* future areas which will need new infrastructure to service proposed development. The FUSA-EXT includes potential future extensions beyond the FUSA areas identified in the *Norman 2025 Plan* and will need new infrastructure to support the development.

### 3.1.3 FBO 2030 Loading Scenario

The FBO 2030 scenario was developed in the InfoWorks CS$^{TM}$ V10.5 model by Innovyze to perform the system capacity analysis. The FBO 2030 scenario represents future wastewater loading corresponding to the full build-out conditions from the CUSA, FUSA, and FUSA-EXT applied to the existing sewer infrastructure, which conveys all flows to the existing Norman Water Reclamation Facility (WRF). By applying the FBO 2030 wastewater load to the model, one can identify the future system capacity constraints corresponding to FBO conditions, as identified by the locations in the existing collection system that exceed capacity (defined below) or overflow onto the ground. For this analysis, no future or proposed infrastructure needs are included.




**CITY OF NORMAN**
**FLOW MONITORING & MODELING**

WASTEWATER SERVICE AREAS

N

0 0.5 1
MILES

NORMAN UTILITIES AUTHORITY

HDR ENGINEERING, INC.
17111 PRESTON RD. SUITE 300
DALLAS, TX 75248-1232
972-960-4400

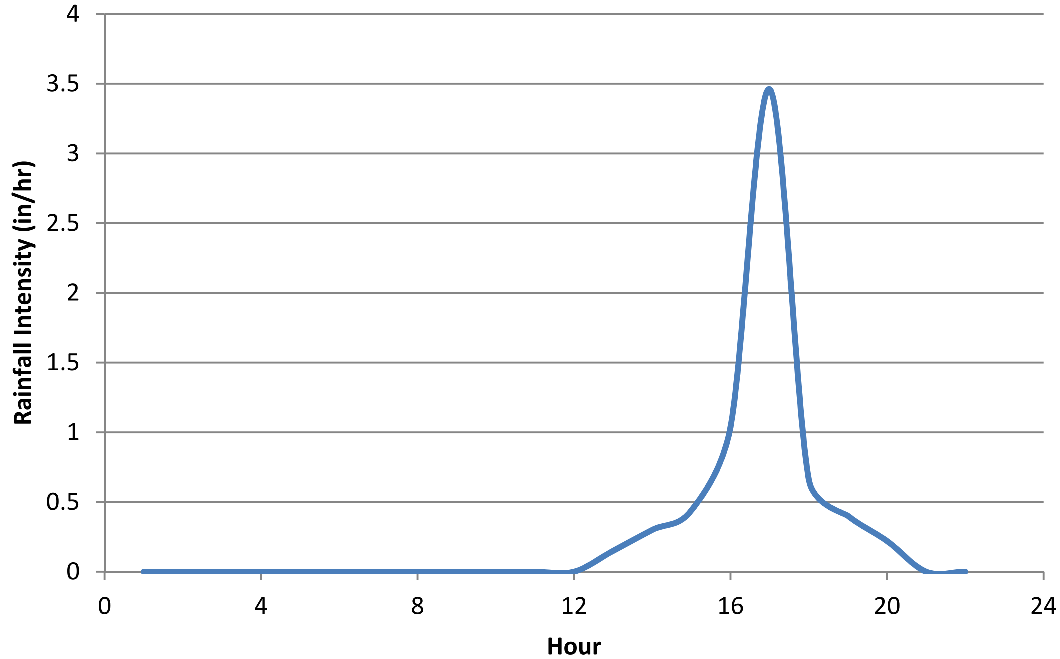
FIGURE 3-1

FILE: J:\MAP_DOCS\MXD\2005 PROJECT\TM_2\PN139145_FIGURE2-1_WASTEWATERSERVICEAREAS_8X11.MXD

### 3.1.4 Design Storm Event

A 5-year frequency storm of 4-hour duration was established as the design storm event in the *Master Plan.* The local IDF curve (Zone II IDF curve for Oklahoma), shown in **Figure 3-2**, was used for the design hyetograph in the *Master Plan* and applied to the current modeling work. The total rainfall volume of this storm is 3.33 inches.

**Figure 3-2. 5-year, 4-hour Storm Hyetograph**




### 3.1.5 Capacity Criteria

The City's collection system is designed to allow for surcharging in the manholes to within one foot of the rim elevation during a 5-year, 4-hour storm event. Capacity is defined to be exceeded when the water level is less than one foot from the manhole rim, or a sanitary sewer overflow (SSO) occurs. This capacity criteria is used for both the dry weather flow (DWF) and 5-year, 4-hour design storm wet weather flow (WWF) modeled conditions.

### 3.1.6 Major System Improvements

In 2012, the City implemented Lift Station D improvements. Lift Station D receives all flow from the north collection system and pumps this flow over the ridgeline to the Bishop Creek (BS) basin. From this point, the wastewater in the collection system flows under gravity flow condition to the existing Norman WRF. An equalization basin was constructed as part of the Lift Station D improvements to provide system storage when the peak flows exceed the pump station capacity of 6 million gallons per day (mgd). Lift Station D is designed to be the headworks to a proposed future North WRF.

With the improvements made at Lift Station D, the following lift stations are now inactive:

- Carrington Lift Station
- York Lift Station
- Industrial Lift Station
- Sutton Place Lift Station

The Vo-Tech Lift Station (LS) and Tecumseh LS are still active; however, the discharge of these stations has been redirected to the Little River Interceptor which flows into Lift Station D.

The capacity analysis was performed including these system improvements.

## 3.2 System Capacity

### 3.2.1 Existing System Overall Flow

The overall system flow, as modeled for FBO 2030, is listed in **Table 3-1**.

**Table 3-1. FBO 2030 Flow Summary**

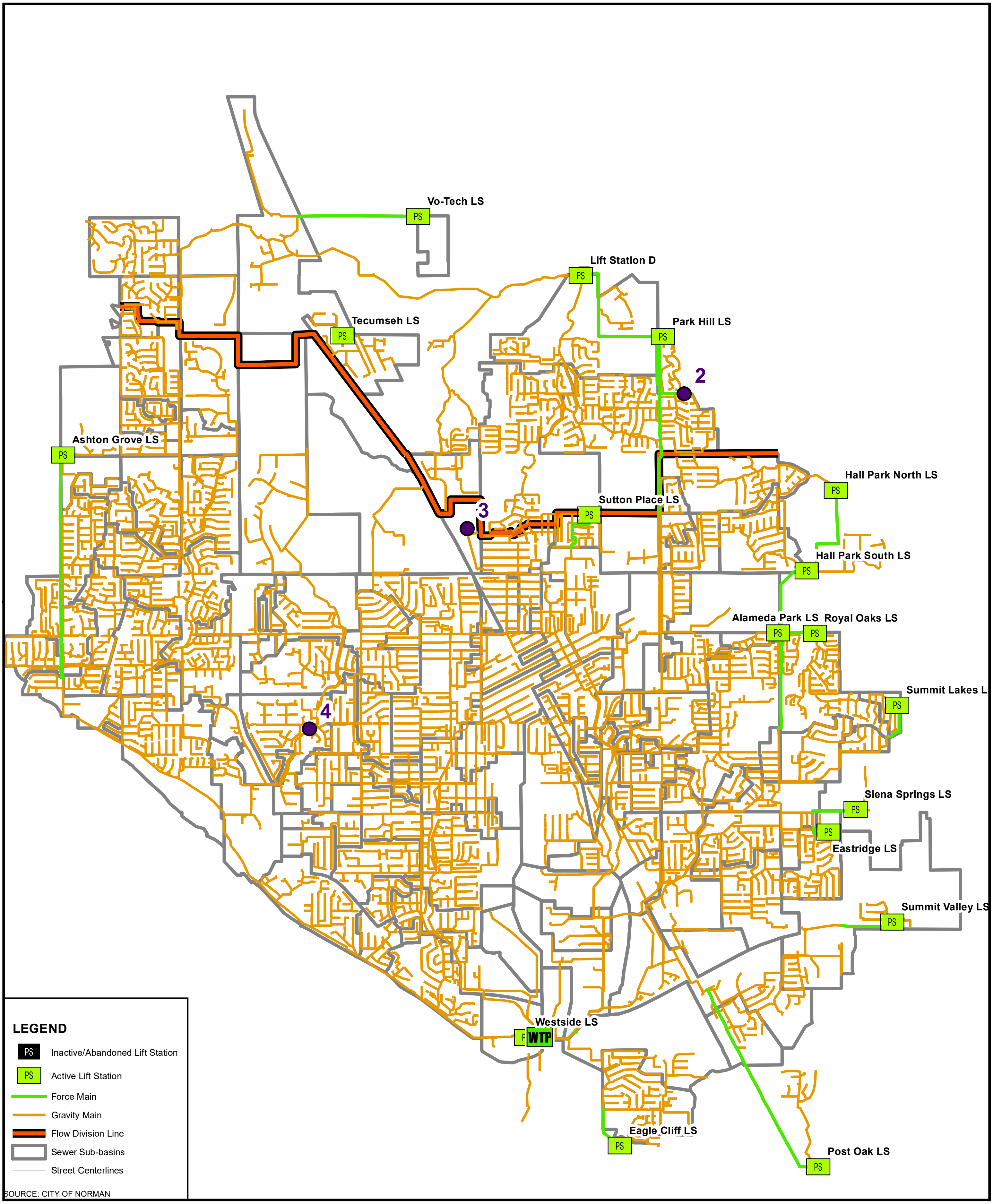
| Location | FBO 2030 Loading Condition | |
|---|---|---|
| | Dry Weather Flow (mgd) | Peak 2-hr Wet Weather Flow[1] (mgd) |
| Lift Station D | 3.2 | 17.7 |
| Norman WRF | 18.6 | 70.8 |

[1] 5-Year, 4-hour storm

### 3.2.2 Dry Weather Flow Analysis

As discussed in Chapter 2, diurnal flow patterns were established for each land use type and applied in the model. The use of these patterns simulates the daily variation of DWF allowing the peak DWF to be determined.

The model was used to predict capacity and SSO issues during peak DWF with the FBO 2030 loading applied to the existing sewer system. Three manholes were identified as having a maximum water level in the manhole less than 1 foot from the manhole rim. Of these 3 locations, none were predicted to be an SSO. **Table 3-2** lists these locations, and **Figure 3-3** illustrates the manhole locations in the system.




**CITY OF NORMAN**
**FLOW MONITORING AND MODELING**
OVERFLOW LOCATIONS
FBO 2030 DRY WEATHER FLOW CONDITION

**NORMAN UTILITIES AUTHORITY**

HDR ENGINEERING, INC.
17111 PRESTON RD, SUITE 300
DALLAS, TX 75248-1232
972-960-4400

**FIGURE 3-3**

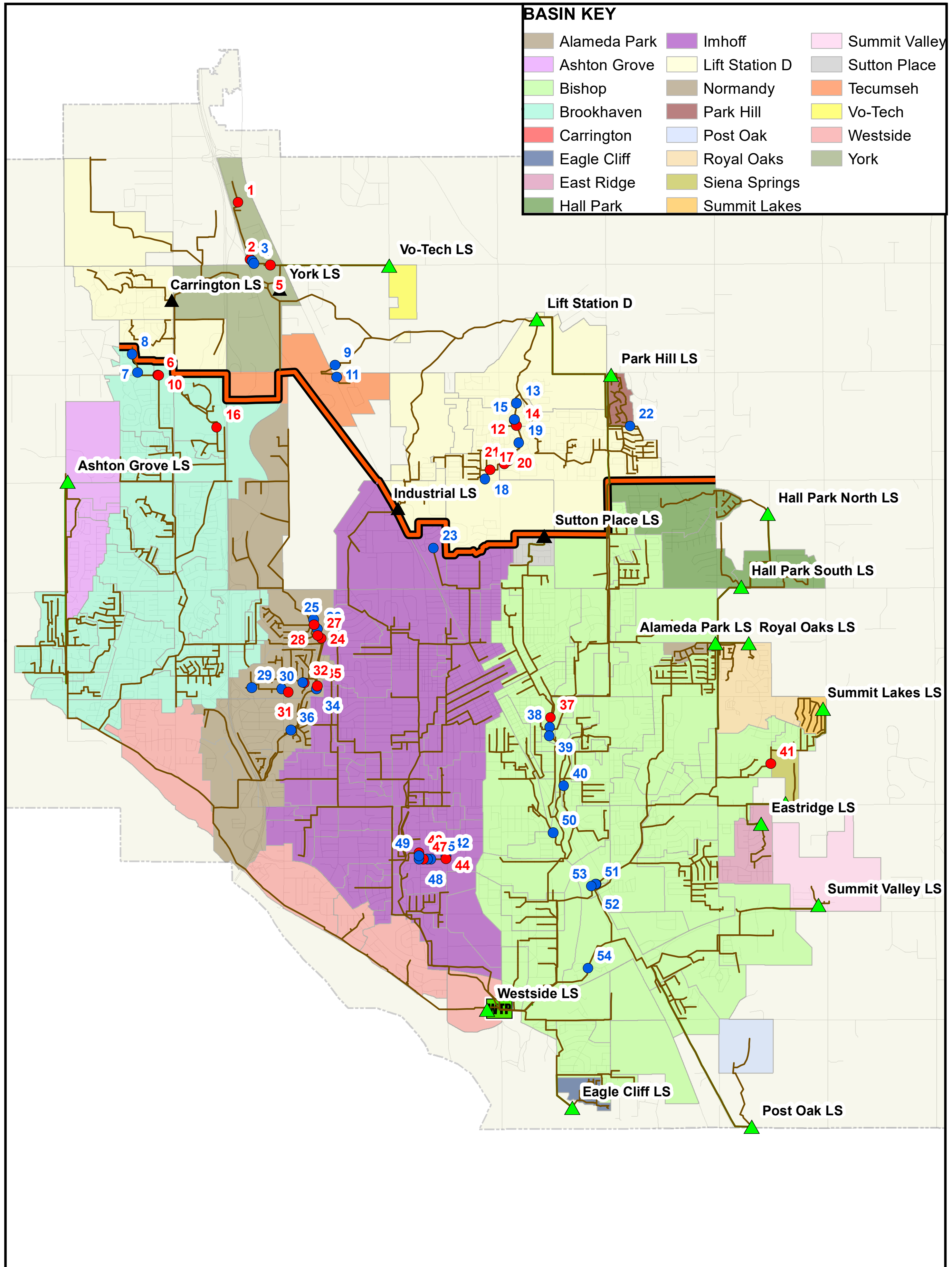
PATH: C:\USERS\TWHITFIE\DOCUMENTS_WORKING DOCUMENTS_PROJECTS_ACTIVE\NORMAN\2017_FIGURE_RECREATION\FIGURE3-3.MXD - USER: TWHITFIE - DATE: 10/27/2017

**Table 3-2. Peak DWF Capacity Issues at FBO 2030**

| Map ID | Manhole ID | Rim Elevation | Maximum Water Level Elevation | Freeboard to Manhole Rim (ft) | SSO (Y or N) | Estimated Overflow Quantity (gallons) | Sewer Basin |
|---|---|---|---|---|---|---|---|
| 2 | 121012 | 1211.65 | 1210.68 | 0.97 | N | - | Park Hill |
| 3 | 162002 | 1192.00 | 1191.33 | 0.67 | N | - | Imhoff |
| 4 | 239042 | 1136.10 | 1135.13 | 0.97 | N | - | Normandy |
| **Total Estimated Overflow Quantity** | | | | | | **0** | |

### 3.2.3 Wet Weather Flow Analysis

Next, the model was run to predict capacity and SSO issues during a 5-year, 4-hour design storm event when the FBO 2030 loading is applied to the existing sewer system. A total of 54 manholes were identified at a maximum water level less than 1 foot from the manhole rim. Of those 54 locations, 21 locations were predicted as SSOs with a total volume of 0.42 million gallons. **Table 3-3** lists these locations, and **Figure 3-4** illustrates the manhole locations in the system. Model results for the FBO 2030 loading at the design storm event for the existing sewer system are provided in **Appendix 3A** (manholes) and **Appendix 3B** (gravity pipe).

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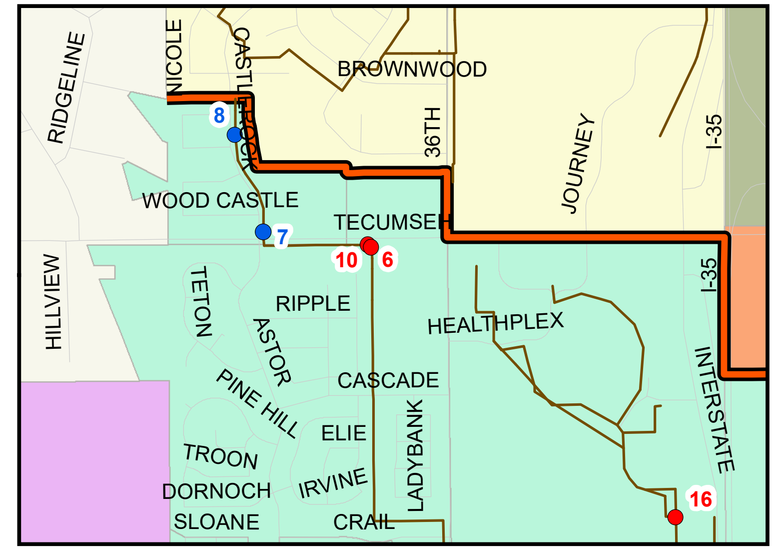
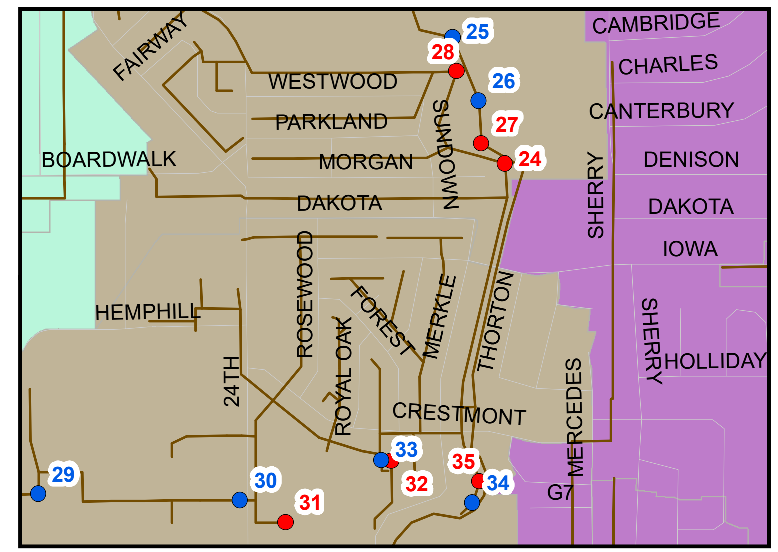
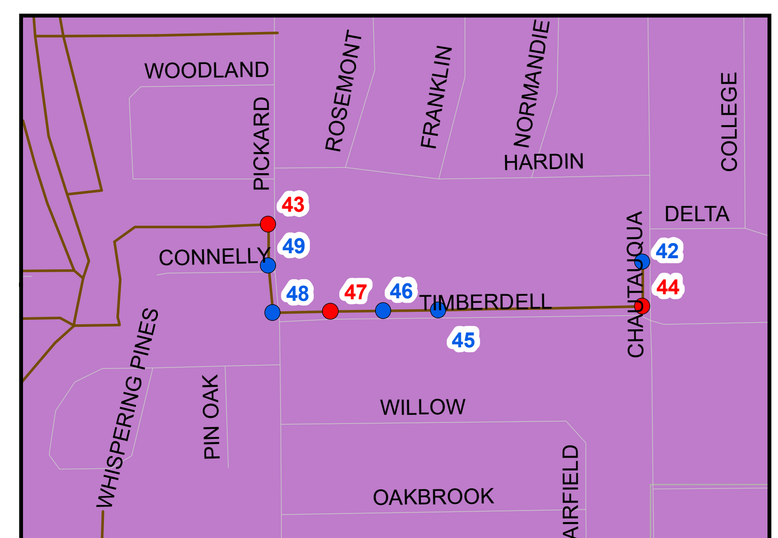
INSET 1




INSET 2

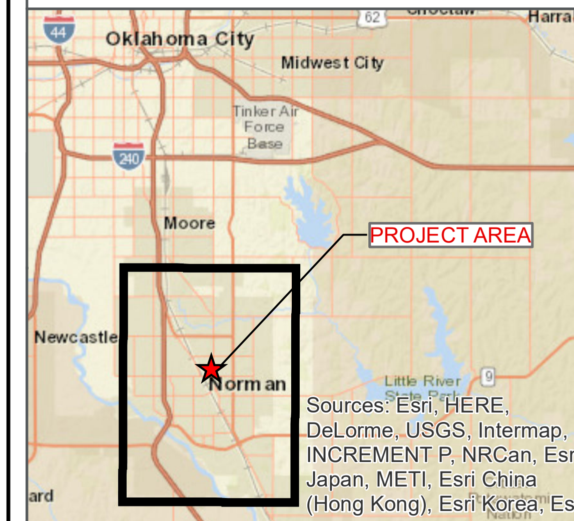



INSET 3




# CITY OF NORMAN FLOW MONITORING AND MODELING

CAPACITY ISSUE LOCATIONS
ONE PLANT SCENARIO
FBO 2030 5-YEAR 4-HOUR STORM
FLOW CONDITION




## MODELED SYSTEM LEGEND

- Inactive Lift Station
- Active Lift Station
- <1-FT FREEBOARD
- SSO
- Gravity Main
- WTP Outfall
- Force Main
- Flow Division Line
- City Boundary
- Street Centerlines

SOURCE: CITY OF NORMAN INFRASTRUCTURE AS OF APRIL 2012

INSET MAPS NOT TO SCALE

N

NORMAN UTILITIES AUTHORITY

HDR ENGINEERING, INC.
17111 PRESTON ROAD, SUITE 300
DALLAS, TEXAS 75248-1232
(972) 960-4400

FIGURE 3-4

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### Table 3-3. Design Storm Capacity Issues at FBO 2030

| Map ID | Sewer Basin | Manhole ID | Rim Elevation | Maximum Water Level Elevation | Freeboard to Manhole Rim (ft) | SSO (Y or N) | Estimated Overflow (gallons) |
|---|---|---|---|---|---|---|---|
| 37 | Bishop | 243056 | 1141.00 | 1141.00 | 0 | Y | 6,600 |
| 38 | Bishop | 243088 | 1138.40 | 1138.35 | -0.05 | N | 0 |
| 39 | Bishop | 243089 | 1136.70 | 1136.42 | -0.28 | N | 0 |
| 40 | Bishop | 260067 | 1131.16 | 1130.71 | -0.45 | N | 0 |
| 41 | Bishop | 263037 | 1208.88 | 1208.88 | 0 | Y | 45,400 |
| 50 | Bishop | 285069 | 1126.00 | 1125.31 | -0.69 | N | 0 |
| 51 | Bishop | 297006 | 1120.81 | 1120.39 | -0.42 | N | 0 |
| 52 | Bishop | 297007 | 1120.81 | 1120.36 | -0.45 | N | 0 |
| 53 | Bishop | 297010 | 1120.76 | 1120.10 | -0.66 | N | 0 |
| 54 | Bishop | 321075 | 1117.42 | 1117.22 | -0.2 | N | 0 |
| | | | | | | **Subtotal Bishop** | **52,000** |
| 6 | Brookhaven | 72002 | 1188.24 | 1188.24 | 0 | Y | 42,100 |
| 7 | Brookhaven | 72008 | 1193.46 | 1192.55 | -0.91 | N | 0 |
| 8 | Brookhaven | 72024 | 1193.00 | 1192.78 | -0.23 | N | 0 |
| 10 | Brookhaven | 98065 | 1188.19 | 1188.19 | 0 | N | 0 |
| 16 | Brookhaven | 113008 | 1180.03 | 1180.03 | 0 | Y | 9,600 |
| | | | | | | **Subtotal Brookhaven** | **51,700** |
| 23 | Imhoff | 162002 | 1192.00 | 1191.41 | -0.59 | N | 0 |
| 42 | Imhoff | 283037 | 1152.00 | 1152.00 | 0 | N | 0 |
| 43 | Imhoff | 283118 | 1144.00 | 1144.00 | 0 | Y | 12,200 |
| 44 | Imhoff | 283125 | 1151.24 | 1151.24 | 0 | Y | 1,000 |
| 45 | Imhoff | 283128 | 1148.08 | 1147.80 | -0.28 | N | 0 |
| 46 | Imhoff | 283129 | 1147.43 | 1146.91 | -0.52 | N | 0 |
| 47 | Imhoff | 283130 | 1146.00 | 1146.00 | 0 | Y | 1,500 |
| 48 | Imhoff | 283131 | 1145.73 | 1145.28 | -0.45 | N | 0 |
| 49 | Imhoff | 283132 | 1145.39 | 1144.63 | -0.76 | N | 0 |
| | | | | | | **Subtotal Imhoff** | **14,700** |
| 9 | Lift Station D | 76008 | 1144.00 | 1143.34 | -0.66 | N | 0 |
| 12 | Lift Station D | 105012 | 1134.00 | 1134.00 | 0 | Y | 8,200 |
| 13 | Lift Station D | 105014 | 1128.00 | 1127.13 | -0.87 | N | 0 |
| 14 | Lift Station D | 105039 | 1136.50 | 1136.50 | 0 | Y | 7,700 |
| 15 | Lift Station D | 105043 | 1134.50 | 1133.62 | -0.88 | N | 0 |
| 17 | Lift Station D | 118008 | 1158.86 | 1158.86 | 0 | Y | 1,600 |

| Map ID | Sewer Basin | Manhole ID | Rim Elevation | Maximum Water Level Elevation | Freeboard to Manhole Rim (ft) | SSO (Y or N) | Estimated Overflow (gallons) |
|---|---|---|---|---|---|---|---|
| 18 | Lift Station D | 118015 | 1162.75 | 1162.12 | -0.63 | N | 0 |
| 19 | Lift Station D | 119085 | 1142.46 | 1142.10 | -0.36 | N | 0 |
| 20 | Lift Station D | 119092 | 1148.42 | 1148.42 | 0 | Y | 20,500 |
| 21 | Lift Station D | 119093 | 1150.19 | 1150.19 | 0 | Y | 9,400 |
| **Subtotal Lift Station D** | | | | | | | **47,400** |
| 24 | Normandy | 192020 | 1147.10 | 1147.10 | 0 | Y | 23,200 |
| 25 | Normandy | 192029 | 1152.34 | 1151.83 | -0.51 | N | 0 |
| 26 | Normandy | 192030 | 1150.16 | 1149.74 | -0.42 | N | 0 |
| 27 | Normandy | 192031 | 1148.29 | 1148.29 | 0 | Y | 1,200 |
| 28 | Normandy | 192033 | 1151.73 | 1151.73 | 0 | Y | 100 |
| 29 | Normandy | 207003 | 1163.21 | 1162.36 | -0.85 | N | 0 |
| 30 | Normandy | 207061 | 1159.05 | 1158.90 | -0.15 | N | 0 |
| 31 | Normandy | 208022 | 1157.13 | 1157.13 | 0 | Y | 22,500 |
| 32 | Normandy | 208044 | 1143.00 | 1143.00 | 0 | Y | 59,300 |
| 33 | Normandy | 208045 | 1143.92 | 1143.44 | -0.48 | N | 0 |
| 34 | Normandy | 208072 | 1142.00 | 1141.10 | -0.9 | N | 0 |
| 35 | Normandy | 208079 | 1141.06 | 1141.06 | 0 | Y | 6,700 |
| 36 | Normandy | 239042 | 1136.10 | 1135.25 | -0.85 | N | 0 |
| **Subtotal Normandy** | | | | | | | **113,000** |
| 22 | Park Hill | 121012 | 1211.65 | 1210.71 | -0.94 | N | 0 |
| **Subtotal Park Hill** | | | | | | | **0** |
| 11 | Tecumseh | 102008 | 1153.73 | 1153.26 | -0.47 | N | 0 |
| **Subtotal Tecumseh** | | | | | | | **0** |
| 1 | York | 42006 | 1168.12 | 1168.12 | 0 | Y | 78,600 |
| 2 | York | 44008 | 1166.11 | 1166.11 | 0 | Y | 18,300 |
| 3 | York | 44009 | 1166.99 | 1166.05 | -0.94 | N | 0 |
| 4 | York | 44010 | 1166.70 | 1166.03 | -0.67 | N | 0 |
| 5 | York | 70031 | 1160.00 | 1160.00 | 0 | Y | 44,700 |
| **Subtotal York** | | | | | | | **141,600** |
| **Total North Sewer Shed** | | | | | | | **189,000** |
| **Total South Sewer Shed** | | | | | | | **231,400** |
| **Total All Basins** | | | | | | | **420,400** |

# 3.3 I/I Reduction Efficiency Analysis

## 3.3.1 Background

To perform the capacity analysis and capital project identification documented in the *Master Plan,* CDM utilized a skeleton version of the collection system model on MIKE URBAN SWMM™ software platform by DHI. Previous flow monitoring data collected in 1998 was used to calibrate the MIKE URBAN SWMM model. While the focus of the capital projects identified in the *Master Plan* was to alleviate SSOs in the system, I/I reduction is also realized when projects are implemented.

The rainfall derived I/I (RDII) analysis performed in 2001, and updated in 2004 by CDM, utilized the same RTK methodology used for the RDII analysis in this modeling effort. The R values (R1, R2, and R3) correspond to the percentage of the RDII that enters the system through direct runoff, intermediate and delayed RDII response. The sum of R1, R2, and R3 is an indicator of the total RDII response. By comparing the sum of R values estimated during the 2004 modeling update and current model calibration, a general system-wide observation can be made regarding effectiveness of I/I reduction in the system as a result of the improvements implemented since the *Master Plan*.

## 3.3.2 RDII Calibration Comparison

The sewer infrastructure from the two modeling efforts (2004 and the current modeling work) was compared to determine which sewer segments could be identified in both models. Then, the identified sewers were analyzed to determine if the pipe diameter had changed and which sewer basin(s) the sewer serviced. The major basins identified where diameter changes occurred include Lift Station D (formerly identified as Little River in the *Master Plan*), Bishop, Brookhaven, Imhoff and Normandy. The R-value comparison; therefore, focused on these basins.

Some differences that were identified between the two modeling efforts that could affect the difference in rainfall response include:

- Intensity, duration, and antecedent conditions to categorize RDII response may differ widely.
- The *Master Plan* indicates that three storm events captured with the flow monitoring in 1998 were used to perform model calibration; one from April, one from September, and one from November. The flow monitoring for the current project observed two rainfall events which were used to calibrate RDII response in the model; one in June and one in September.
- The reconfiguration of some sewer basins since 2001, including addition/subtraction of land area, impact contributing sewer area characteristics and size, and thus RDII response.
- Flow monitoring locations were different, possibly capturing different system conditions.
- The approaches used in the two models to account for dry-weather infiltration might not be the same.

- The methodology used in the two models to model dry-weather sanitary loading might not be the same. For this modeling work, the CUSA, FUSA and FUSA-EXT loading is as of May 31, 2010. For the 2004 modeling work, the CUSA loading is as of July 1, 2003, the FUSA loading is as of August 2, 2004, and there were no designated FUSA-EXT areas.
- Model extent. The MIKE URBAN SWMM model was more simplified than the InfoWorks CS model, and it used RDII response to simulate both rainfall response and also non-modeled system hydraulics. This would include flow attenuation in RDII response and reduce R1.

However; while being mindful of these limitations, a general conclusion about the reduction of the total rainfall volume entering the system due to implementation of sewer improvements can be derived by the comparison of RDII calibration between the two models. **Table 3-4** compares the weighted sum of R values from the 2004 model to those derived from this flow modeling calibration.

**Table 3-4. R-Value Comparison of Current Model to Previous Model**

| Sewer Basin | 2004 Weighted Average Sum of R | 2010 Weighted Average Sum of R | Estimated % RDII Reduction |
|---|---|---|---|
| Lift Station D | 0.015 | 0.008 | 47% |
| Bishop | 0.019 | 0.005 | 72% |
| Brookhaven | 0.015 | 0.012 | 17% |
| Imhoff | 0.019 | 0.012 | 37% |
| Normandy | 0.015 | 0.015 | 0% |

Three R-values from each major sewer basin, corresponding to direct, intermediate and delayed RDII, from both models were summed and then averaged by overall basin acreage to normalize the results. Based on this comparison, observed RDII in the Norman collection system during the 2010 flow monitoring appears to be lower than that indicated by the data from the 2004 model. The largest difference in R values occurs in the Bishop basin, where the majority of the capital improvements have taken place. No substantial change was indicated for the Normandy basin.

## 3.4 Conclusions

As stated in the *Master Plan*, a total of 53 SSO locations were identified under the 5-year, 4-hour design storm event resulting in an overflow volume of approximately 8 million gallons (Reference Figure 2-5 in the *Master Plan*). This is based on FBO of the existing, approved and contractual service areas defined in the *Master Plan.*

Under the current FBO 2030 model scenario, which includes FBO loading of the CUSA, FUSA and FUSA-EXT service areas, a total of 22 SSO locations were identified resulting in a total overflow volume of approximately 420,000 gallons. The improvements the City has performed on the collection system since the *Master Plan* have greatly reduced predicted SSO occurrences and volumes. Additional collection system improvements needed to further eliminate SSOs and to convey the FBO 2030 wastewater loading is discussed in the next chapter.

# 4 Collection System Scenario Analysis

## 4.1 Introduction

The purpose of this chapter is to document the existing collection system improvements needed to convey FBO 2030 flows without overflows for two alternative wastewater reclamation facility (WRF) planning scenarios. The two planning scenarios include:

- One-Plant Scenario: Conveyance of all wastewater to the existing Norman WRF
- Two-Plant Scenario: Conveyance of the north basin wastewater to a proposed North WRF and conveyance of the south basin wastewater to the existing Norman WRF

Detailed model output is included in the appendices.

In addition, the FBO 2030 design flows and estimated construction costs for each scenario are included in this chapter. The work associated with this project does not include developing or sizing future interceptor or trunk line extensions.

## 4.2 Evaluation Criteria

### 4.2.1 Sewer and Lift Station Capacity

The City's collection system capacity criteria allows for surcharging in the manholes to within one foot of the rim elevation during a 5-year, 4-hour storm event. Capacity is defined to be exceeded when the water level is less than one foot from the manhole rim or top of lift station wetwell, or a sanitary sewer overflow (SSO) occurs during the 5-year, 4-hour design storm wet weather flow (WWF) modeled condition.

Using the calibrated model representing the City's physical infrastructure from April 2012, node (manhole and wetwell) locations predicted to overflow under the design 5-year, 4-hour storm event at the FBO 2030 wastewater loading condition were identified and documented in Chapter 3. The same calibrated model used is used in the evaluation of the two planning scenarios documented in this chapter to identify the collection system improvements needed to eliminate the previously identified capacity issues.

### 4.2.2 Interceptor Sizing

Where needed, interceptors are sized to meet the sewer capacity criteria described above. Whether the interceptor is a replacement or parallel relief sewer is typically based on engineering judgment. The costs of providing a replacement interceptor, for example, can be significant when reviewing construction feasibility and bypass pumping needs for large diameter pipes. In these cases, a parallel relief interceptor resulting in a smaller diameter pipe with less bypass pumping requirements could be more economical. Generally speaking, pipes that are larger than 18 inches in diameter are usually good candidates for designing relief sewers. This evaluation is typically performed as a cost-saving measure during the design phase of an individual pipeline project.

To be conservative in developing a planning level cost estimate for proposed improvements, it is assumed in this project that pipes will be replaced with a larger diameter pipe rather than paralleled with a relief sewer.

### 4.2.3 Force Main Sizing

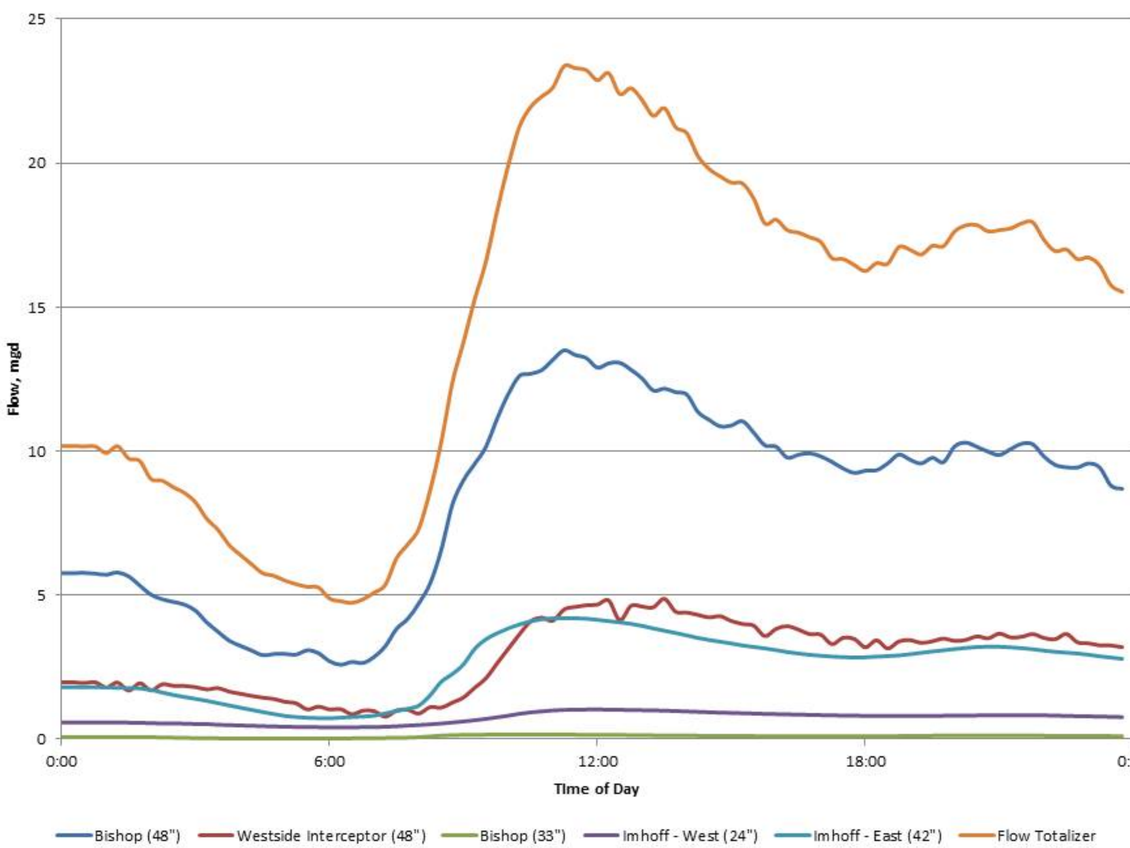
Where needed, force mains are sized to convey the firm pumping capacity required at the lift station with a velocity range of 2 to 7 fps.

### 4.2.4 WRF Design Flow

The methodology used to determine flow parameters in the *Master Plan* (CDM, 2001) was utilized for this evaluation, as defined below.

**Average Dry Weather Flow:** Average dry weather flow is determined from the calibrated model based on the FBO 2030 loading applied to the existing sewer infrastructure during a dry weather condition during the seasonal period when the University of Oklahoma is in session. The Norman WRF is modeled as an outfall. There are five main interceptors that convey flow into this outfall. The total of these flows represents the incoming flow to the Norman WRF. **Figure 4-1** illustrates the diurnal curve for an average dry weather flow day from flow from each interceptor and the overall total (labeled as flow totalizer). The predicted average dry weather flow is 17.8 mgd.

**Figure 4-1. FBO 2030 Dry Weather Flow**



**Annual Average Flow:** Flow monitoring data was compared to the wastewater treatment plant historical data from the same time period to determine a multiplier for converting Average Dry Weather Flow to Annual Average Flow (included as **Appendix 4A**). The historical data average dry weather flow (from 1993 to 2011) is 10.5 mgd. Plant data during the monitoring period was recorded at an average flow of 11.14 mgd for the three-month period of May, June, and July. Modeled existing dry weather flow was 10.98 mgd, resulting in a multiplier of 1.01. Compared to the annual wastewater flow average at the WRF of 10.8 for the same period, the calculated ratio would be less than 1.0. This is largely due to higher groundwater flows in the summer months during the monitoring. For the purposes of this analysis, the ratio used is 1.01. This results in an Annual Average Flow of 18.0 (17.8 x 1.01).

**Planning Capacity:** The 5% planning capacity developed during the *Master Plan* is utilized as part of this evaluation. Refer to Section 1.5 of the *Master Plan* for a detailed explanation of how the 5% planning capacity was developed (see **Appendix 4B**).

**Maximum Month Flow:** The maximum month flow is the largest volume of flow to be received during a continuous 30-day period as predicted by the model under the FBO 2030 loading, wet weather condition. Based on a Maximum Month to Annual Average ratio of 1.2:1, the Maximum Month Flow is 21.6 mgd.

**Maximum Day Flow:** The maximum day flow is the largest volume of flow to be received during a continuous 24-hour period as predicted by the model under the FBO 2030 loading, wet weather condition. The Maximum Day Flow calculated as 36.0 mgd.

**Peak 2-hour Flow:** The peak 2-hour flow is the largest volume of flow to be received during a two-hour period as predicted by the model under the FBO 2030 loading, wet weather condition. The Peak 2-hour Flow is calculated as 81.9 mgd.

### 4.2.5 Cost Estimating Criteria for Gravity Piping

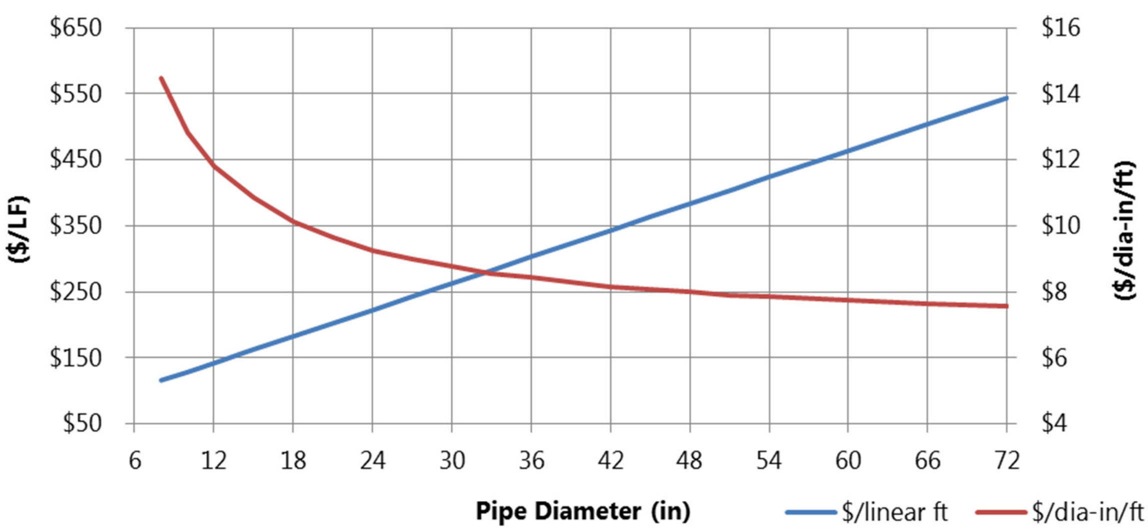
The construction estimates are based on the construction costs presented in Table 3-3 from the *Northside Lift Station Preliminary Engineering Report* (HDR, May 2007). Estimated interceptor costs per linear foot (LF) were developed using cost data from actual bid tabulations at the time the previous report was written. These replacement pipe unit costs are inclusive of all pipeline construction aspects, including mobilization, site work, installation, and overhead and profit. The unit costs from the *Northside Lift Station Preliminary Engineering Report* have been updated from 2007 to 2014 using the *RS Means Construction Cost Indices (CCI) for 1st Quarter 2014*. See **Appendix 4C**. The 2014 estimated unit construction costs for wastewater interceptors are shown in **Table 4-1**.

Table 4-1. Estimated Unit Construction Costs for Wastewater Interceptors (2014 Dollars)

| Pipe Diameter (in) | Unit Cost ($/LF) | Unit Cost ($/dia-in/LF) |
|---|---|---|
| 8 | $ 115.93 | $ 14.49 |
| 10 | $ 128.26 | $ 12.83 |
| 12 | $ 141.83 | $ 11.82 |
| 15 | $ 162.79 | $ 10.85 |
| 18 | $ 182.53 | $ 10.14 |
| 21 | $ 202.26 | $ 9.63 |
| 24 | $ 221.99 | $ 9.25 |
| 27 | $ 242.96 | $ 9.00 |
| 30 | $ 262.69 | $ 8.76 |
| 33 | $ 282.42 | $ 8.56 |
| 36 | $ 303.39 | $ 8.43 |
| 39 | $ 323.12 | $ 8.29 |
| 42 | $ 342.86 | $ 8.16 |
| 45 | $ 363.82 | $ 8.08 |
| 48 | $ 383.55 | $ 7.99 |
| 51 | $ 403.29 | $ 7.91 |
| 54 | $ 424.25 | $ 7.86 |
| 60 | $ 463.72 | $ 7.73 |
| 66 | $ 504.42 | $ 7.64 |
| 72 | $ 543.88 | $ 7.55 |

The unit costs listed in **Table 4-1** are shown graphically on **Figure 4-2**. The construction cost per linear foot of interceptor construction increases linearly with increased pipe diameters. The corresponding cost per diameter inch of interceptor per linear foot follows an exponential pattern that is higher for smaller diameter interceptors and decreases for larger diameter pipes.

Figure 4-2. Estimated Unit Construction Costs for Wastewater Interceptors (2014 Dollars)



### 4.2.6 Cost Estimating Criteria for Lift Stations and Force Mains

Based on recent project experience, the cost estimating criteria used for lift station expansion and force main is:

- $1,000/gpm of firm pumping capacity required for lift station expansion
- $8/dia-in/ft for shallow (less than 8 feet) PVC force main installation

## 4.3 One-Plant Scenario

This section describes the evaluation of the collection system with all flow conveyed to the existing Norman WRF, given the FBO 2030 loading condition under a 5-year, 4-hour storm event.

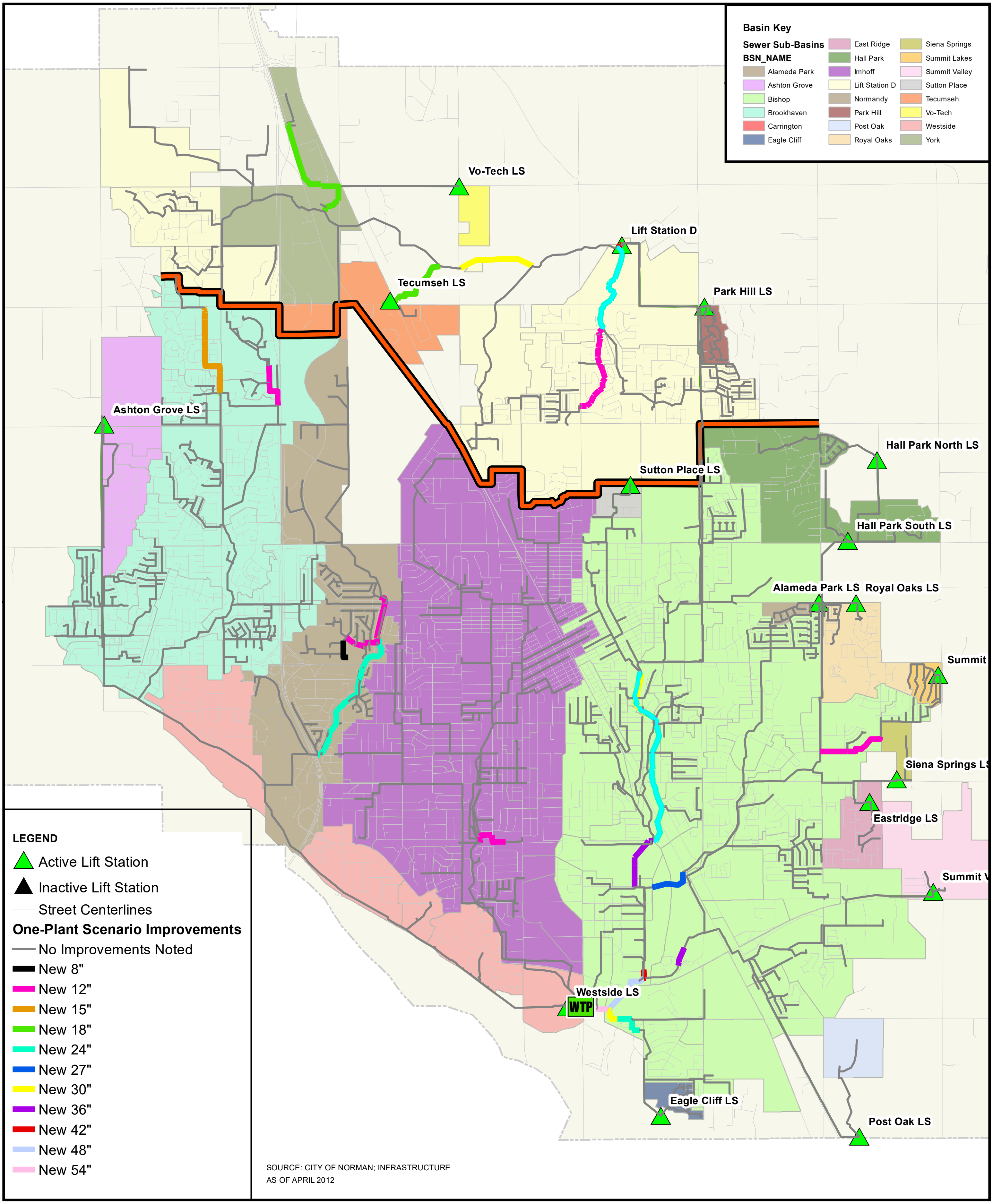
### 4.3.1 Pipeline Evaluation

A total of 54 manholes were identified at a maximum water level less than 1 foot from the manhole rim. Of those 54 locations, 22 locations were predicted as SSOs with a total volume of 0.42 million gallons. These locations are shown on **Figure 3-4** and listed in **Table 3-3**.

The locations of the proposed pipeline improvements to eliminate these 54 capacity excursions are shown collectively on **Figure 4-3** and discussed per individual sewer basin below. It is assumed all gravity pipeline improvements are replacement lines for this planning level evaluation. In addition, it is assumed as part of this analysis that sufficient capacity at the headworks of the Norman WRF exists. Wastewater flow entering the WRF is purposely allowed to surcharge. This backwater condition is included in the model at a water surface elevation of 1111.10.

The lift station improvements are discussed in the Lift Station Evaluation section.

The model output data for all conduits (pipe segments) after all the improvements described herein are implemented are included in **Appendix 4D**.




**CITY OF NORMAN**
**FLOW MONITORING AND MODELING**
RECOMMENDED PIPING IMPROVEMENTS
ONE PLANT SCENARIO
FBO 2030, 5-YEAR 4-HOUR STORM
FLOW CONDITION

N
0 0.5 1 Miles

NORMAN UTILITIES AUTHORITY

HDR ENGINEERING, INC.
17111 PRESTON RD, SUITE 300
DALLAS, TX 75248-1232
972-960-4400

**FIGURE 4-3**

PATH: C:\USERS\TWHITFIE\DOCUMENTS_WORKING DOCUMENTS_PROJECTS_ACTIVE\NORMAN_NORMAN_GIS\MAP_DOCS\MXD\2017_FIGURE_RECREATION\FIGURE4-3.MXD - USER: TWHITFIE - DATE: 11/1/2017

## Bishop Sewer Basin

As illustrated on **Figure 3-4**, a total of 10 nodes (manholes) are identified within the Bishop Sewer Basin as having less than 1 foot of freeboard in the manhole. Of these nodes, two are predicted to have an SSO resulting in an approximate overflow quantity of 52,031 gallons under the design storm and FBO 2030 condition.

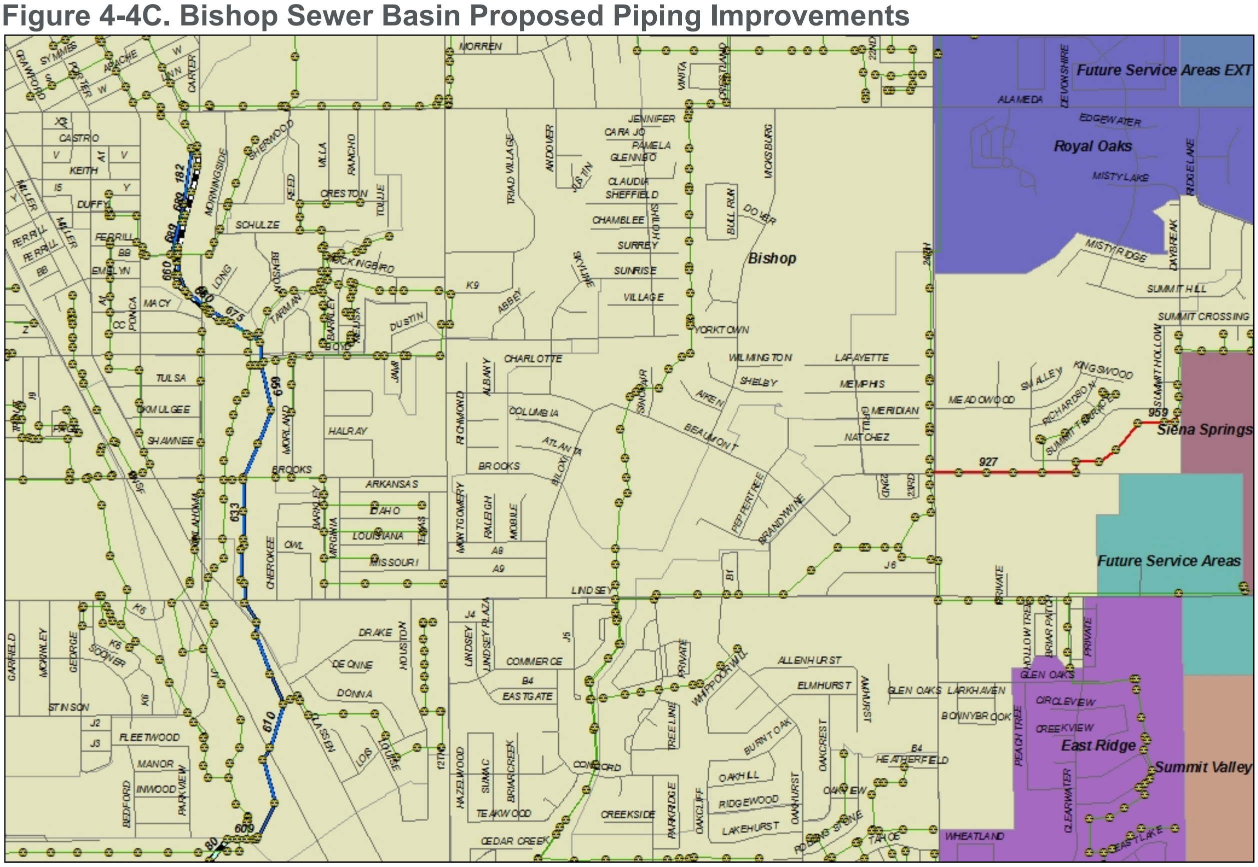
Approximately 24,223 LF (4.6 mi) of sewer improvements, as shown on **Figure 4-3** and below on **Figures 4-4A, 4-4B, and 4-4C** are identified in order to eliminate the identified nodes and SSO locations to meet the required design criteria previously described.

No lift stations or associated force mains are located within the Bishop Sewer Basin.

**Figure 4-4A. Bishop Sewer Basin Proposed Piping Improvements**

## Figure 4-4B. Bishop Sewer Basin Proposed Piping Improvements


## Figure 4-4C. Bishop Sewer Basin Proposed Piping Improvements



The individual pipe segments are listed in **Table 4-2**. The information presented includes the pipe ID (with corresponding upstream manhole), Branch ID (corresponding to **Figures 4-4A to 4-4C**) that identifies the grouping, the pipe asset identification number, the existing diameter, the proposed diameter, the peak velocity, the peak flow, and the peak water depth. Peak values represent the maximum value observed during the storm event in the model analysis assuming the piping improvements have been applied.

One manhole, Manhole ID 358001 (shown on **Figure 4-5**), would need to be sealed to contain flow.

**Figure 4-5. Bishop Sewer Basin Proposed Manhole Improvement Location**

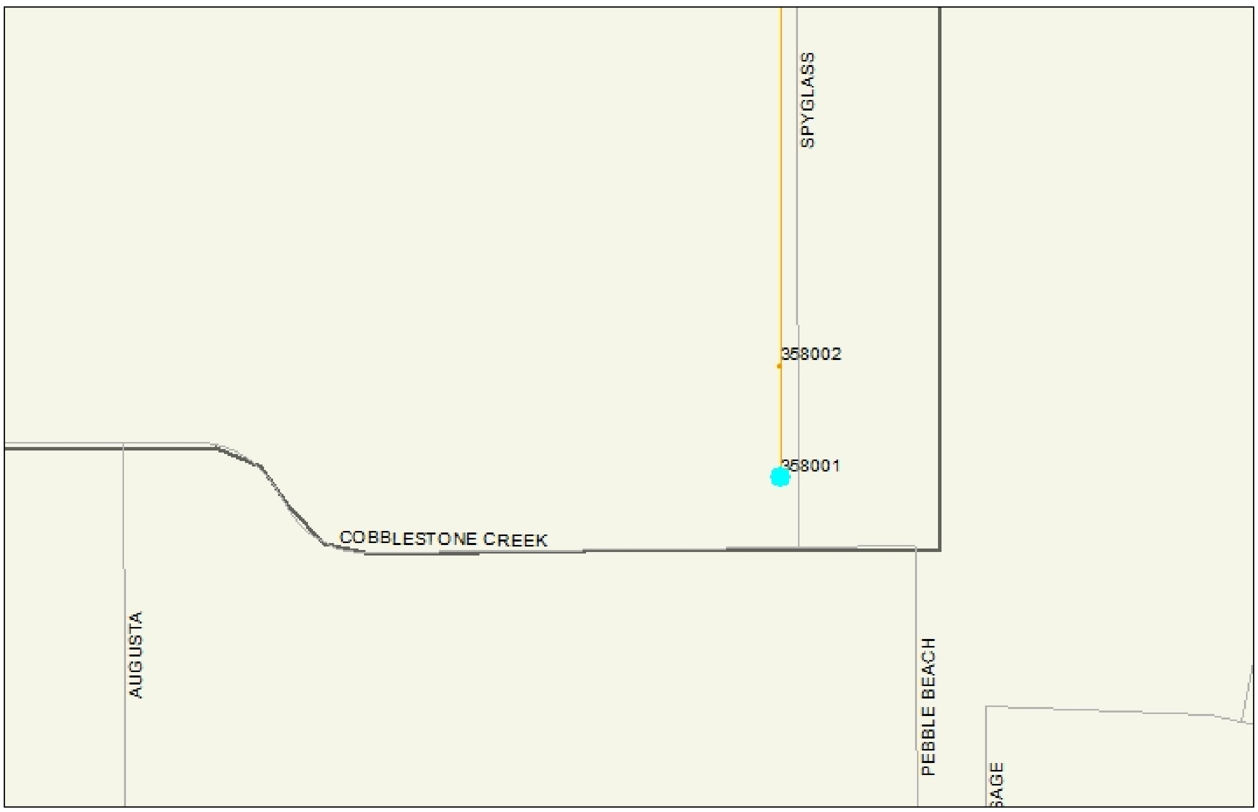


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**Table 4-2. Bishop Sewer basin Proposed Piping Improvements**

| Pipe ID | Asset ID | Branch ID | Pipe Material | L (ft) | Existing Dia (in) | Proposed Dia (in) | Peak After Improvements | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Velocity (ft/s) | Flow (mgd) | Water Depth (ft) |
| 286067.1 | 286067286080 | 80 | UNKN | 104 | 33 | 36 | 3.8 | 15.4 | 4.5 |
| 286080.1 | 286080296032 | 80 | UNKN | 307 | 33 | 36 | 3.8 | 15.4 | 3.7 |
| 296026.1 | 296026296046 | 80 | PVC | 35 | 36 | 36 | 4.3 | 15.5 | 2.8 |
| 296027.1 | 296027296026 | 80 | UNKN | 345 | 33 | 36 | 5.1 | 15.5 | 2.7 |
| 296028.1 | 296028296027 | 80 | UNKN | 303 | 33 | 36 | 4.5 | 15.5 | 2.8 |
| 296029.1 | 296029296028 | 80 | UNKN | 297 | 33 | 36 | 4.2 | 15.5 | 3.0 |
| 296030.1 | 296030296029 | 80 | UNKN | 348 | 33 | 36 | 4.2 | 15.5 | 3.2 |
| 296031.1 | 296031296030 | 80 | UNKN | 356 | 33 | 36 | 4.4 | 15.4 | 3.3 |
| 296032.1 | 296032296031 | 80 | UNKN | 291 | 33 | 36 | 4.6 | 15.4 | 3.2 |
| 243055.1 | 243055243056 | 182 | UNKN | 601 | 18 | 21 | 3.3 | 3.9 | 2.3 |
| 243056.1 | 243056243057 | 182 | UNKN | 560 | 18 | 21 | 2.8 | 3.4 | 2.4 |
| 329022.1 | 329022329024 | 551 | FRP | 523 | 48 | 54 | 4.1 | 24.8 | 5.4 |
| 329016.1 | 329016329017 | 553 | FRP | 224 | 42 | 48 | 3.2 | 20.7 | 6.2 |
| 329017.1 | 329017329018 | 553 | FRP | 417 | 42 | 48 | 3.2 | 20.7 | 6.0 |
| 329018.1 | 329018329019 | 553 | FRP | 331 | 42 | 48 | 3.2 | 20.7 | 5.7 |
| 329019.1 | 329019329020 | 553 | FRP | 253 | 42 | 48 | 3.2 | 20.9 | 5.6 |
| 329020.1 | 329020329021 | 553 | FRP | 561 | 42 | 48 | 3.3 | 20.9 | 5.4 |
| 329021.1 | 329021329022 | 553 | FRP | 496 | 42 | 48 | 3.9 | 20.9 | 5.2 |
| 329059.1 | 329059329016 | 553 | FRP | 31 | 42 | 48 | 3.2 | 20.7 | 6.4 |
| 329023.1 | 329023329022 | 554 | FRP | 95 | 24 | 30 | 1.5 | 3.1 | 5.0 |
| 329049.1 | 329049329023 | 554 | PVC | 35 | 24 | 30 | 1.4 | 3.1 | 5.0 |
| 329050.1 | 329050329049 | 554 | PVC | 343 | 24 | 30 | 1.5 | 3.0 | 4.8 |
| 329051.1 | 329051329052 | 554 | UNKN | 295 | 18 | 24 | 2.4 | 1.9 | 2.7 |
| 329052.1 | 329052329053 | 554 | UNKN | 350 | 18 | 24 | 2.8 | 1.9 | 3.2 |
| 329053.1 | 329053329050 | 554 | UNKN | 353 | 24 | 30 | 2.9 | 2.8 | 3.9 |
| 349001.1 | 349001329051 | 554 | UNKN | 302 | 12 | 24 | 3.2 | 1.4 | 2.3 |
| 349002.1 | 349002349001 | 554 | UNKN | 207 | 12 | 24 | 2.7 | 1.4 | 2.7 |
| 349003.1 | 349003349002 | 554 | UNKN | 350 | 12 | 24 | 3.5 | 1.3 | 2.6 |
| 329010.1 | 329010329059 | 560 | FRP | 362 | 36 | 42 | 2.1 | 10.1 | 6.6 |
| 321058.1 | 321058321059 | 567 | UNKN | 488 | 33 | 36 | 3.1 | 11.7 | 5.2 |
| 321059.1 | 321059321075 | 567 | UNKN | 195 | 33 | 36 | 3.2 | 11.7 | 5.0 |
| 321075.1 | 321075330021 | 567 | UNKN | 161 | 33 | 36 | 3.4 | 11.7 | 4.9 |
| 329074.1 | 329074329059 | 567 | DIP | 42 | 36 | 42 | 4.4 | 11.8 | 4.9 |
| 330003.1 | 330003329074 | 567 | DIP | 48 | 36 | 42 | 4.0 | 11.8 | 4.9 |
| 297010.1 | 297010297011 | 579 | UNKN | 429 | 21 | 27 | 2.3 | 3.9 | 5.2 |
| 297011.1 | 297011297012 | 579 | UNKN | 265 | 21 | 27 | 2.5 | 3.9 | 4.9 |
| 297012.1 | 297012297013 | 579 | UNKN | 201 | 21 | 27 | 3.3 | 3.9 | 4.8 |
| 297013.1 | 297013297014 | 579 | UNKN | 306 | 21 | 27 | 2.4 | 3.9 | 5.0 |
| 297014.1 | 297014297015 | 579 | UNKN | 197 | 21 | 27 | 2.7 | 4.0 | 4.9 |
| 297015.1 | 297015297022 | 579 | UNKN | 307 | 21 | 27 | 3.7 | 4.0 | 4.8 |
| 297022.1 | 297022297019 | 579 | UNKN | 160 | 21 | 27 | 3.1 | 4.1 | 5.2 |
| 286092.1 | 286092286067 | 609 | UNKN | 100 | 33 | 36 | 3.8 | 15.2 | 4.7 |
| 286008.1 | 286008286054 | 610 | UNKN | 492 | 18 | 21 | 4.1 | 3.2 | 3.2 |
| 286054.1 | 286054286055 | 610 | UNKN | 154 | 18 | 21 | 3.1 | 3.2 | 3.5 |
| 286055.1 | 286055286056 | 610 | UNKN | 489 | 18 | 21 | 3.1 | 3.2 | 3.6 |
| 286056.1 | 286056286082 | 610 | UNKN | 407 | 18 | 21 | 2.8 | 3.2 | 3.7 |
| 286066.1 | 286066286065 | 610 | UNKN | 26 | 18 | 21 | 2.7 | 3.2 | 4.7 |
| 286082.1 | 286082286066 | 610 | UNKN | 25 | 18 | 21 | 4.9 | 3.2 | 4.0 |
| 260063.2 | 260063260067 | 633 | UNKN | 343 | 18 | 21 | 2.7 | 3.1 | 3.0 |
| 260067.1 | 260067260068 | 633 | UNKN | 296 | 18 | 21 | 2.7 | 3.1 | 2.9 |
| 260068.1 | 260068260103 | 633 | UNKN | 68 | 18 | 21 | 2.8 | 3.1 | 2.9 |
| 260103.1 | 260103260104 | 633 | PVC | 58 | 18 | 21 | 2.9 | 3.1 | 2.8 |
| 260104.1 | 260104260105 | 633 | PVC | 323 | 18 | 21 | 2.9 | 3.1 | 2.8 |
| 260105.1 | 260105260106 | 633 | PVC | 58 | 18 | 21 | 2.7 | 3.1 | 2.9 |
| 260106.1 | 260106286010 | 633 | UNKN | 215 | 18 | 21 | 3.3 | 3.1 | 2.7 |
| 286010.1 | 286010286011 | 633 | UNKN | 242 | 18 | 21 | 2.9 | 3.0 | 2.9 |
| 286011.1 | 286011286012 | 633 | UNKN | 139 | 18 | 21 | 3.2 | 3.1 | 2.8 |
| 286012.1 | 286012286013 | 633 | UNKN | 448 | 18 | 21 | 3.2 | 3.0 | 2.9 |
| 286013.1 | 286013286008 | 633 | UNKN | 361 | 18 | 21 | 2.9 | 3.0 | 3.1 |
| 260014.1 | 260014260063 | 654 | PVC | 21 | 18 | 21 | 3.5 | 4.2 | 3.3 |
| 244048.1 | 244048244144 | 659 | UNKN | 164 | 18 | 21 | 4.6 | 3.7 | 3.1 |
| 244144.1 | 244144260011 | 659 | UNKN | 259 | 18 | 21 | 3.6 | 3.7 | 3.2 |
| 260011.1 | 260011260012 | 659 | UNKN | 494 | 18 | 21 | 3.1 | 3.7 | 3.1 |
| 260012.1 | 260012260013 | 659 | UNKN | 389 | 18 | 21 | 3.1 | 3.8 | 2.6 |
| 260013.1 | 260013260014 | 659 | UNKN | 404 | 18 | 21 | 4.5 | 3.8 | 2.4 |
| 243052.1 | 243052243059 | 660 | UNKN | 138 | 24 | 30 | 4.4 | 9.5 | 3.5 |
| 243058.1 | 243058243052 | 660 | UNKN | 58 | 24 | 30 | 4.4 | 9.5 | 3.8 |
| 243059.1 | 243059243089 | 660 | UNKN | 112 | 24 | 30 | 4.5 | 9.5 | 3.2 |

| Pipe ID | Asset ID | Branch ID | Pipe Material | L (ft) | Existing Dia (in) | Proposed Dia (in) | Peak After Improvements | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Velocity (ft/s) | Flow (mgd) | Water Depth (ft) |
| 243060.1 | 243060243086 | 660 | UNKN | 296 | 24 | 30 | 4.7 | 9.5 | 2.6 |
| 243086.1 | 243086244143 | 660 | UNKN | 134 | 24 | 30 | 4.7 | 9.5 | 2.4 |
| 243089.1 | 243089243060 | 660 | UNKN | 170 | 24 | 30 | 4.8 | 9.5 | 2.7 |
| 243057.1 | 243057243063 | 675 | UNKN | 83 | 18 | 21 | 3.2 | 3.7 | 1.9 |
| 243063.1 | 243063243064 | 675 | UNKN | 200 | 18 | 21 | 4.1 | 3.7 | 1.4 |
| 243064.1 | 243064243065 | 675 | UNKN | 197 | 18 | 21 | 3.2 | 3.7 | 1.6 |
| 243065.1 | 243065244023 | 675 | UNKN | 315 | 18 | 21 | 5.0 | 3.7 | 0.9 |
| 244023.1 | 244023244025 | 675 | UNKN | 38 | 18 | 21 | 5.1 | 3.7 | 2.2 |
| 244025.1 | 244025244026 | 675 | VCP | 196 | 18 | 21 | 4.9 | 3.6 | 3.0 |
| 244026.1 | 244026244047 | 675 | VCP | 53 | 18 | 21 | 3.0 | 3.6 | 3.6 |
| 244047.1 | 244047244048 | 675 | UNKN | 209 | 18 | 21 | 2.9 | 3.2 | 3.4 |
| 243041.1 | 243041243083 | 689 | UNKN | 134 | 24 | 30 | 4.7 | 9.4 | 5.4 |
| 243083.1 | 243083243084 | 689 | UNKN | 281 | 24 | 30 | 4.4 | 9.4 | 5.2 |
| 243084.1 | 243084243085 | 689 | UNKN | 253 | 24 | 30 | 4.3 | 9.4 | 5.0 |
| 243085.1 | 243085243087 | 689 | UNKN | 78 | 24 | 30 | 4.3 | 9.4 | 4.8 |
| 243087.1 | 243087243088 | 689 | UNKN | 282 | 24 | 30 | 4.4 | 9.4 | 4.4 |
| 243088.1 | 243088243058 | 689 | UNKN | 104 | 24 | 30 | 4.4 | 9.4 | 4.1 |
| 263008.1 | 263008263009 | 927 | VCP | 319 | 8 | 12 | 4.1 | 1.0 | 2.6 |
| 263009.1 | 263009263033 | 927 | PVC | 290 | 10 | 12 | 2.9 | 1.1 | 2.1 |
| 263033.1 | 263033263034 | 927 | PVC | 298 | 10 | 12 | 3.1 | 1.1 | 1.4 |
| 263034.1 | 263034262082 | 927 | PVC | 303 | 10 | 12 | 4.3 | 1.2 | 0.6 |
| 263037.1 | 263037263038 | 959 | PVC | 112 | 8 | 12 | 3.1 | 0.9 | 9.7 |
| 263038.1 | 263038263039 | 959 | PVC | 106 | 8 | 12 | 3.1 | 0.9 | 9.0 |
| 263039.1 | 263039263040 | 959 | PVC | 296 | 8 | 12 | 3.4 | 0.9 | 7.3 |
| 263040.1 | 263040263041 | 959 | PVC | 367 | 8 | 12 | 3.3 | 0.9 | 5.2 |
| 263041.1 | 263041263042 | 959 | PVC | 235 | 8 | 12 | 3.2 | 0.8 | 4.2 |
| 263042.1 | 263042263043 | 959 | PVC | 246 | 8 | 12 | 3.4 | 0.8 | 2.6 |
| 263043.1 | 263043263044 | 959 | PVC | 119 | 8 | 12 | 3.7 | 0.8 | 1.7 |
| 263044.1 | 263044263008 | 959 | PVC | 357 | 8 | 12 | 4.3 | 0.8 | 0.5 |

UNKN – information is unknown

## Brookhaven Sewer Basin

As illustrated on **Figure 3-4**, a total of 5 nodes are identified within the Brookhaven Sewer Basin as having less than 1 foot of freeboard in the manhole. Of these nodes, 3 are predicted to have an SSO resulting in an approximate overflow quantity of 51,761 gallons under the design storm and FBO 2030 condition.

Approximately 6,607 LF (1.2 mi) of sewer improvements, as shown on **Figure 4-3** and below in **Figure 4-6**, are identified in order to eliminate the identified nodes and SSO location to meet the required design criteria previously described.

No lift stations or associated force mains are located within the Brookhaven Sewer Basin.

**Figure 4-6. Brookhaven Sewer Basin Proposed Piping Improvements**


The individual pipe segments are listed in **Table 4-3**. The information presented includes the pipe ID (with corresponding upstream manhole), Branch ID (corresponding to **Figure 4-6**) that identifies the grouping, the pipe asset identification number, the existing diameter, the proposed diameter, the peak velocity, the peak flow, and the peak water depth. Peak values represent the maximum value observed during the storm event in the model analysis assuming the piping improvements have been applied.

### Table 4-3. Brookhaven Sewer Basin Proposed Piping Improvements

| Pipe ID | Asset ID | Branch ID | Pipe Material | L (ft) | Existing Dia (in) | Proposed Dia (in) | Peak After Improvements | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Velocity (ft/s) | Flow (mgd) | Water Depth (ft) |
| 112038.1 | 112038112039 | 430 | UNKN | 249.10 | 10 | 15 | 3.17 | 1.14 | 3.05 |
| 112039.1 | 112039112040 | 430 | UNKN | 317.50 | 10 | 15 | 3.03 | 1.14 | 2.28 |
| 112040.1 | 112040112041 | 430 | UNKN | 355.60 | 10 | 15 | 3.38 | 1.15 | 1.40 |
| 72002.1 | 072002098065 | 430 | PVC | 38.70 | 8 | 15 | 3.28 | 0.74 | 2.95 |
| 98007.1 | 098007098008 | 430 | PVC | 161.40 | 10 | 15 | 2.52 | 0.86 | 4.30 |
| 98008.1 | 098008098009 | 430 | PVC | 357.80 | 10 | 15 | 2.43 | 0.85 | 4.85 |
| 98009.1 | 098009098010 | 430 | PVC | 325.30 | 10 | 15 | 2.34 | 0.90 | 5.41 |
| 98010.1 | 098010098033 | 430 | PVC | 82.90 | 10 | 15 | 2.52 | 0.84 | 5.41 |
| 98033.1 | 098033098108 | 430 | PVC | 22.50 | 10 | 15 | 2.43 | 0.83 | 5.42 |
| 98034.1 | 098034098035 | 430 | PVC | 284.10 | 10 | 15 | 2.63 | 0.81 | 5.60 |
| 98035.1 | 098035098036 | 430 | PVC | 282.90 | 10 | 15 | 2.48 | 0.81 | 5.71 |
| 98036.1 | 098036098037 | 430 | PVC | 283.90 | 10 | 15 | 2.04 | 0.81 | 5.81 |
| 98037.1 | 098037098038 | 430 | PVC | 249.60 | 10 | 15 | 2.68 | 1.01 | 5.21 |
| 98038.1 | 098038098039 | 430 | PVC | 398.50 | 10 | 15 | 2.52 | 1.01 | 4.67 |
| 98039.1 | 098039112038 | 430 | UNKN | 285.90 | 10 | 15 | 2.83 | 1.12 | 3.94 |
| 98043.1 | 098043098007 | 430 | UNKN | 296.10 | 10 | 15 | 2.55 | 0.69 | 3.79 |
| 98056.1 | 098056098043 | 430 | UNKN | 96.40 | 10 | 15 | 2.19 | 0.70 | 3.43 |
| 98065.1 | 098065098056 | 430 | UNKN | 100.40 | 10 | 15 | 2.59 | 0.73 | 3.18 |
| 98108.1 | 098108098034 | 430 | PVC | 261.90 | 10 | 15 | 2.73 | 0.81 | 5.42 |
| 112041.1 | 112041112130 | 861 | PVC | 29.90 | 10 | 18 | 5.52 | 1.83 | 0.74 |
| 113004.1 | 113004113017 | 906 | VCP | 403.00 | 8 | 12 | 3.16 | 0.97 | 4.79 |
| 113008.1 | 113008113004 | 906 | VCP | 372.10 | 8 | 12 | 2.93 | 0.98 | 5.69 |
| 113017.1 | 113017113018 | 898 | VCP | 374.10 | 8 | 12 | 3.93 | 0.95 | 1.49 |
| 113018.1 | 113018113019 | 898 | VCP | 339.80 | 10 | 12 | 3.48 | 0.96 | 0.66 |
| 113019.1 | 113019113020 | 879 | VCP | 360.70 | 10 | 12 | 2.82 | 0.74 | 0.84 |
| 113020.1 | 113020113036 | 879 | VCP | 277.30 | 10 | 12 | 2.89 | 0.73 | 0.89 |

UNKN – information is unknown

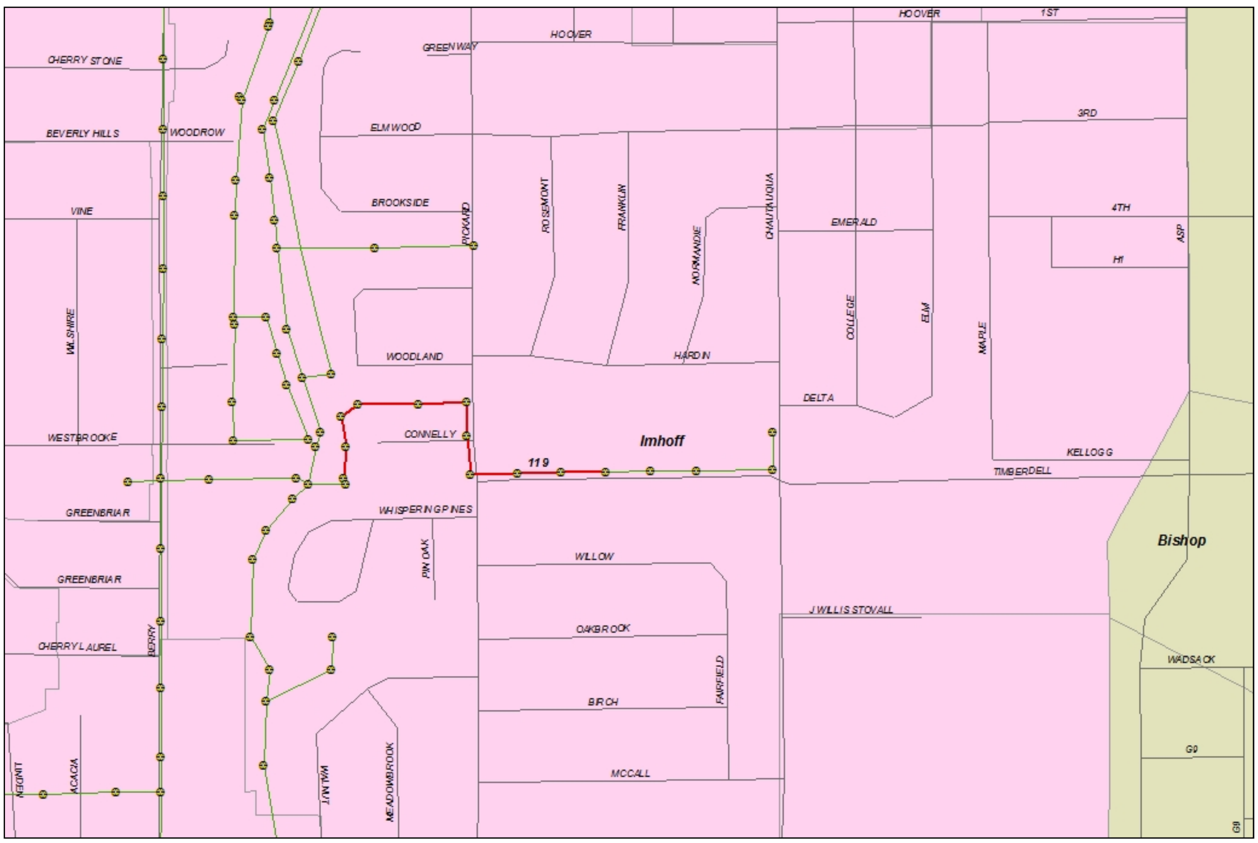
## Imhoff Sewer Basin

As illustrated on **Figure 3-4**, a total of 9 nodes are identified within the Imhoff Sewer Basin as having less than 1 foot of freeboard in the manhole. Of these nodes, 3 are predicted to have an SSO resulting in an approximate overflow quantity of 14,630 gallons under the design storm and FBO 2030 condition.

Approximately 1,750 LF (0.3 mi) of sewer improvements, as shown on **Figure 4-3** and below on **Figure 4-7**, are identified in order to eliminate the identified nodes and SSO locations to meet the required design criteria previously described.

No lift stations or associated force mains are located within the Imhoff Sewer Basin.

**Figure 4-7. Imhoff Sewer Basin Proposed Piping Improvements**




The individual pipe segments are listed in **Table 4-4**. The information presented includes the pipe ID (with corresponding upstream manhole), Branch ID (corresponding to **Figure 4-7**) that identifies the grouping, the pipe asset identification number, the existing diameter, the proposed diameter, the peak velocity, the peak flow, and the peak water depth. Peak values represent the maximum value observed during the storm event in the model analysis assuming the piping improvements have been applied.

**Table 4-4. Imhoff Sewer Basin Proposed Piping Improvements**

| Pipe ID | Asset ID | Branch ID | Pipe Material | L (ft) | Existing Dia (in) | Proposed Dia (in) | Peak After Improvements | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Velocity (ft/s) | Flow (mgd) | Water Depth (ft) |
| 283094.1 | 283094283097 | 119 | UNKN | 134.30 | 10 | 12 | 3.16 | 0.83 | 1.61 |
| 283095.1 | 283095283096 | 119 | UNKN | 258.20 | 6 | 12 | 5.83 | 0.83 | 2.73 |
| 283096.1 | 283096283094 | 119 | UNKN | 88.00 | 6 | 12 | 6.05 | 0.83 | 2.03 |
| 283097.1 | 283097283098 | 119 | UNKN | 132.40 | 10 | 12 | 2.91 | 0.84 | 1.84 |
| 283098.1 | 283098283101 | 119 | UNKN | 28.90 | 10 | 12 | 2.88 | 0.85 | 1.87 |
| 283118.1 | 283118283095 | 119 | UNKN | 212.00 | 6 | 12 | 5.32 | 0.83 | 6.39 |
| 283128.1 | 283128283129 | 119 | PVC | 192.20 | 10 | 12 | 3.16 | 1.03 | 13.03 |
| 283129.1 | 283129283130 | 119 | PVC | 187.30 | 10 | 12 | 2.05 | 1.03 | 13.58 |
| 283130.1 | 283130283131 | 119 | PVC | 203.40 | 10 | 12 | 2.05 | 0.92 | 13.10 |
| 283131.1 | 283131283132 | 119 | PVC | 165.60 | 10 | 12 | 2.92 | 0.92 | 12.79 |
| 283132.1 | 283132283118 | 119 | PVC | 146.20 | 10 | 12 | 1.98 | 0.92 | 13.63 |

UNKN – information is unknown

### Lift Station D Sewer Basin

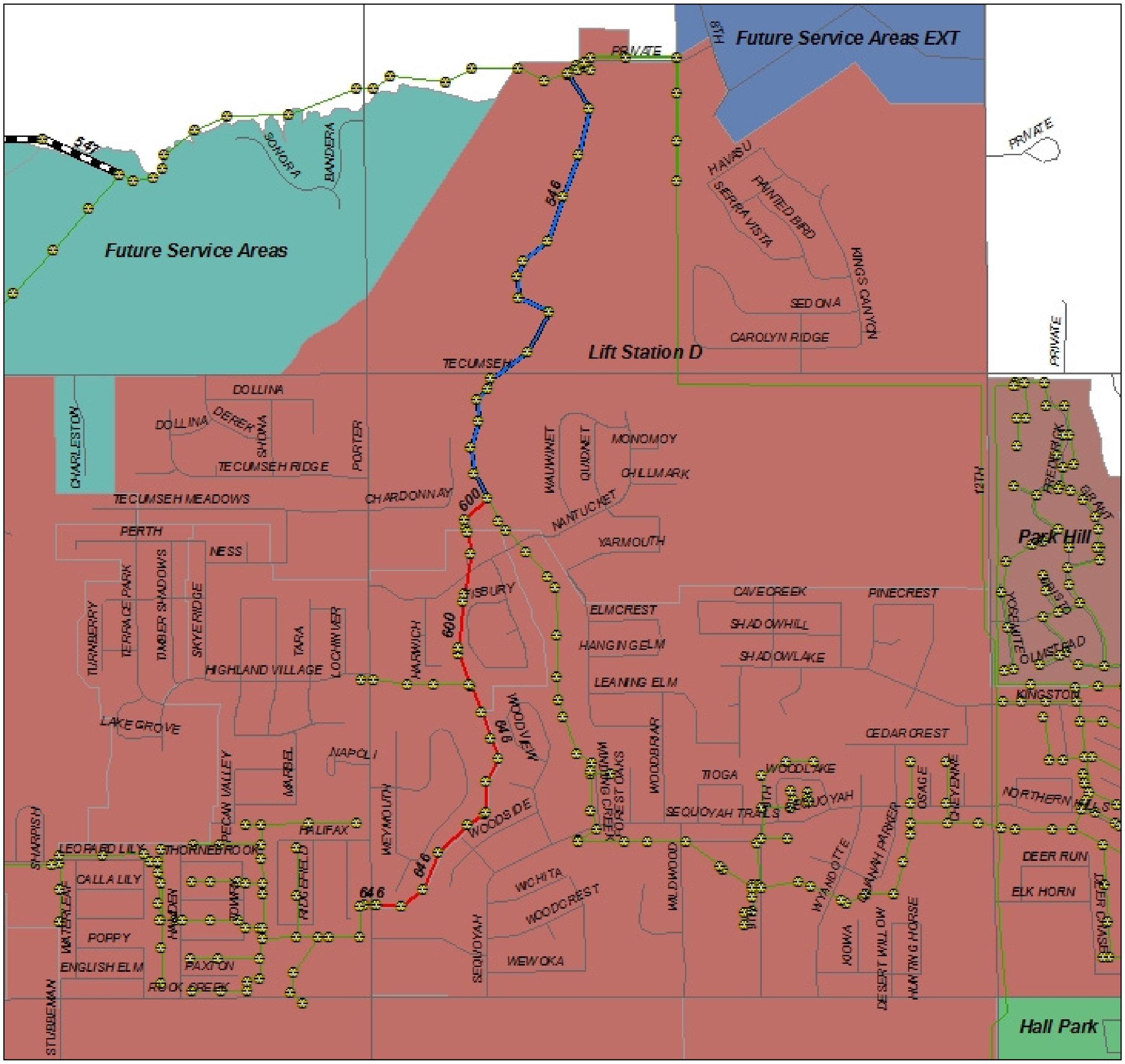
As illustrated on **Figure 3-4**, a total of 10 nodes are identified in the Lift Station D Sewer Basin as having less than 1 foot of freeboard in the manhole. Of these nodes, 5 are predicted to have an SSO resulting in an approximate overflow quantity of 47,360 gallons under the design storm and FBO 2030 condition.

Approximately 14,583 LF (2.8 mi) of sewer improvements, as shown on **Figure 4-3** and below on **Figures 4-8A and 4-8B**, are identified in order to eliminate the identified nodes and SSO locations to meet the required design criteria previously described. The proposed 15-inch pipe starting just north of Nantucket Road to Ridgefield Road was previously identified in the *Northside Lift Station Preliminary Engineering Report* (HDR, May 2007).

Carrington Lift Station and Lift Station D are located in the Lift Station D Sewer Basin. Carrington Lift Station is modeled as an inactive lift station, meaning the lift station is abandoned. Refer to the Lift Station Evaluation Section for Lift Station D information. No force main improvements have been identified for Lift Station D under the design storm and FBO 2030 condition.

The individual pipe segments are listed in **Table 4-5**. The information presented includes the pipe ID (with corresponding upstream manhole), Branch ID (corresponding to **Figures 4-8A and 4-8B**) that identifies the grouping, the pipe asset identification number, the existing diameter, the proposed diameter, the peak velocity, the peak flow, and the peak water depth. Peak values represent the maximum value observed during the storm event in the model analysis assuming the piping improvements have been applied.

**Figure 4-8A. Lift Station D Sewer Basin Proposed Piping Improvements**



## Figure 4-8B. Lift Station D Sewer Basin Proposed Piping Improvements

### Table 4-5. Lift Station D Sewer Basin Proposed Piping Improvements[1]

| Pipe ID | Asset ID | Branch ID | Pipe Material | L (ft) | Existing Dia (in) | Proposed Dia (in) | Peak After Improvements | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Velocity (ft/s) | Flow (mgd) | Water Depth (ft) |
| 105012.1 | 105012105043 | 600 | UNKN | 44.1 | 10 | 12 | 3.65 | 1.50 | 9.00 |
| 105013.1 | 105013105057 | 600 | UNKN | 66.6 | 10 | 12 | 4.02 | 1.49 | 7.75 |
| 105014.1 | 105014105048 | 600 | UNKN | 190.8 | 10 | 12 | 3.52 | 1.46 | 7.06 |
| 105015.1 | 105015105016 | 600 | UNKN | 267 | 10 | 12 | 5.06 | 1.88 | 5.15 |
| 105016.1 | 105016105017 | 546 | UNKN | 251.6 | 18 | 21 | 3.51 | 3.83 | 4.98 |
| 105017.1 | 105017105018 | 546 | UNKN | 213.7 | 18 | 21 | 3.10 | 3.87 | 4.97 |
| 105018.1 | 105018105019 | 546 | UNKN | 233.1 | 18 | 21 | 3.13 | 3.89 | 4.66 |
| 105019.1 | 105019105020 | 546 | UNKN | 186.7 | 18 | 21 | 3.15 | 3.89 | 4.30 |
| 105020.1 | 105020105021 | 546 | UNKN | 120.8 | 18 | 21 | 3.37 | 4.14 | 3.92 |
| 105021.1 | 105021105022 | 546 | UNKN | 95.8 | 18 | 21 | 3.38 | 4.14 | 3.64 |
| 105022.1 | 105022079011 | 546 | UNKN | 393.2 | 18 | 21 | 3.43 | 4.14 | 2.84 |
| 105031.1 | 105031105015 | 600 | UNKN | 80.3 | 10 | 12 | 4.69 | 1.87 | 6.48 |
| 105039.1 | 105039105012 | 600 | UNKN | 282.6 | 10 | 12 | 4.14 | 1.51 | 9.01 |
| 105043.1 | 105043105013 | 600 | UNKN | 394.4 | 10 | 12 | 3.55 | 1.49 | 8.21 |
| 105057.1 | 105057105014 | 600 | UNKN | 343.1 | 10 | 12 | 4.05 | 1.45 | 7.43 |
| 119078.1 | 119078105011 | 646 | UNKN | 234.9 | 10 | 12 | 3.07 | 1.20 | 9.81 |
| 119083.1 | 119083119078 | 646 | UNKN | 235.5 | 10 | 12 | 3.58 | 1.19 | 9.93 |
| 119084.1 | 119084119083 | 646 | UNKN | 183.3 | 10 | 12 | 4.64 | 1.14 | 8.62 |
| 119085.1 | 119085119084 | 646 | UNKN | 222 | 10 | 12 | 4.15 | 1.13 | 7.56 |
| 119088.1 | 119088119086 | 646 | UNKN | 183.2 | 10 | 12 | 4.14 | 1.21 | 7.49 |
| 119089.1 | 119089119088 | 646 | UNKN | 329.5 | 10 | 12 | 3.66 | 1.22 | 7.42 |
| 119092.1 | 119092119089 | 646 | UNKN | 335.4 | 10 | 12 | 3.58 | 1.23 | 7.30 |
| 119093.1 | 119093119092 | 646 | UNKN | 236.5 | 10 | 12 | 3.53 | 1.28 | 7.17 |
| 119095.1 | 119095119129 | 646 | UNKN | 97.1 | 10 | 12 | 4.60 | 1.40 | 7.00 |
| 119123.1 | 119123119085 | 646 | UNKN | 256 | 10 | 12 | 3.22 | 1.13 | 7.60 |
| 119143.1 | 119143119093 | 646 | UNKN | 205.3 | 10 | 12 | 3.46 | 1.41 | 7.51 |
| 77005.1 | 077005077006 | 557 | PVC | 401.7 | 27 | 30 | 4.19 | 11.36 | 4.15 |
| 77006.1 | 7700677007 | 557 | PVC | 383.1 | 27 | 30 | 5.20 | 11.40 | 3.55 |
| 77007.1 | 077007077008 | 547 | PVC | 659.3 | 27 | 30 | 4.73 | 11.40 | 2.86 |
| 77008.1 | 077008077009 | 547 | PVC | 444.2 | 27 | 30 | 4.44 | 11.40 | 2.64 |
| 77009.1 | 077009077010 | 547 | PVC | 751.2 | 27 | 30 | 4.76 | 11.41 | 2.05 |
| 77010.1 | 077010078002 | 547 | PVC | 717.3 | 27 | 30 | 6.41 | 11.42 | 1.47 |
| 75016.1 | 7501676008 | 175 | UNKN | 158.6 | 12 | 18 | 6.54 | 2.52 | 5.46 |
| 76008.1 | 7600876009 | 175 | UNKN | 247 | 12 | 18 | 4.51 | 2.53 | 7.16 |
| 76009.1 | 7600976010 | 175 | UNKN | 120 | 12 | 18 | 4.94 | 2.52 | 6.46 |
| 76010.1 | 7601076011 | 175 | UNKN | 237 | 12 | 18 | 5.09 | 2.52 | 6.05 |
| 76011.1 | 7601176012 | 175 | UNKN | 235 | 12 | 18 | 4.85 | 2.54 | 5.77 |
| 76012.1 | 7601276013 | 175 | UNKN | 276 | 12 | 18 | 5.13 | 2.53 | 5.13 |
| 76013.1 | 7601376014 | 175 | UNKN | 387.3 | 12 | 18 | 5.03 | 2.53 | 4.79 |
| 76014.1 | 7601476015 | 175 | UNKN | 362 | 12 | 18 | 4.58 | 2.54 | 4.19 |
| 76015.1 | 7601576016 | 175 | UNKN | 209.8 | 12 | 18 | 4.68 | 2.54 | 3.32 |
| 76016.1 | 7601676017 | 175 | UNKN | 211.3 | 12 | 18 | 5.34 | 2.54 | 2.23 |
| 76017.1 | 7601776005 | 175 | UNKN | 343.5 | 12 | 18 | 5.58 | 2.56 | 2.08 |
| 79003.1 | 079003079002 | 546 | UNKN | 327.1 | 18 | 24 | 5.45 | 4.56 | 1.03 |
| 79004.1 | 079004079003 | 546 | UNKN | 402.7 | 18 | 24 | 4.25 | 4.56 | 1.34 |
| 79005.1 | 079005079004 | 546 | UNKN | 373.4 | 18 | 24 | 4.41 | 4.47 | 1.39 |
| 79006.1 | 079006079005 | 546 | UNKN | 400.2 | 18 | 24 | 4.53 | 4.46 | 1.36 |
| 79007.1 | 079007079006 | 546 | UNKN | 267.9 | 18 | 24 | 4.36 | 4.35 | 1.36 |
| 79008.1 | 079008079007 | 546 | UNKN | 149.2 | 18 | 24 | 5.33 | 4.33 | 1.00 |
| 79009.1 | 079009079008 | 546 | UNKN | 174.8 | 18 | 24 | 3.85 | 4.33 | 1.43 |
| 79010.1 | 079010079009 | 546 | UNKN | 281.5 | 18 | 24 | 3.54 | 4.22 | 1.99 |
| 79011.1 | 079011079010 | 546 | UNKN | 380.2 | 18 | 24 | 4.70 | 4.15 | 2.26 |

UNKN – information is unknown

[1]At the time of flow monitoring described, Lift Station D was still under construction so all flow was conveyed to the Brookhaven Sewer Basin. As described in the previous chapter, the model development included the Lift Station D improvements, where flow is now conveyed to the Lift Station D Sewer Basin and ultimately to Lift Station D.

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## Normandy Sewer Basin

As illustrated in **Figure 3-4**, a total of 13 nodes are identified within the Normandy Sewer Basin as having less than 1 foot of freeboard in the manhole. Of these nodes, 6 are predicted to have an SSO resulting in an approximate overflow quantity of 118,790 gallons under the design storm and FBO 2030 condition.

Approximately 11,350 LF (2.2 mi) of sewer improvements, shown on **Figure 4-3** and below on **Figures 4-9A and 4-9B**; are identified in order to eliminate the identified nodes and SSO locations to meet the required design criteria previously described. These sewer improvements are associated with a trunk line conveying flow out of the Normandy Sewer Basin.

No lift stations or associated force mains are located within the Normandy Sewer Basin.

The individual pipe segments are listed in **Table 4-6**. The information presented includes the pipe ID (with corresponding upstream manhole), Branch ID (corresponding to **Figures 4-9A and 4-9B**) that identifies the grouping, the pipe asset identification number, the existing diameter, the proposed diameter, the peak velocity, the peak flow, and the peak water depth. Peak values represent the maximum value observed during the storm event in the model analysis assuming the piping improvements have been applied.

**Figure 4-9A. Normandy Sewer Basin Proposed Piping Improvements**

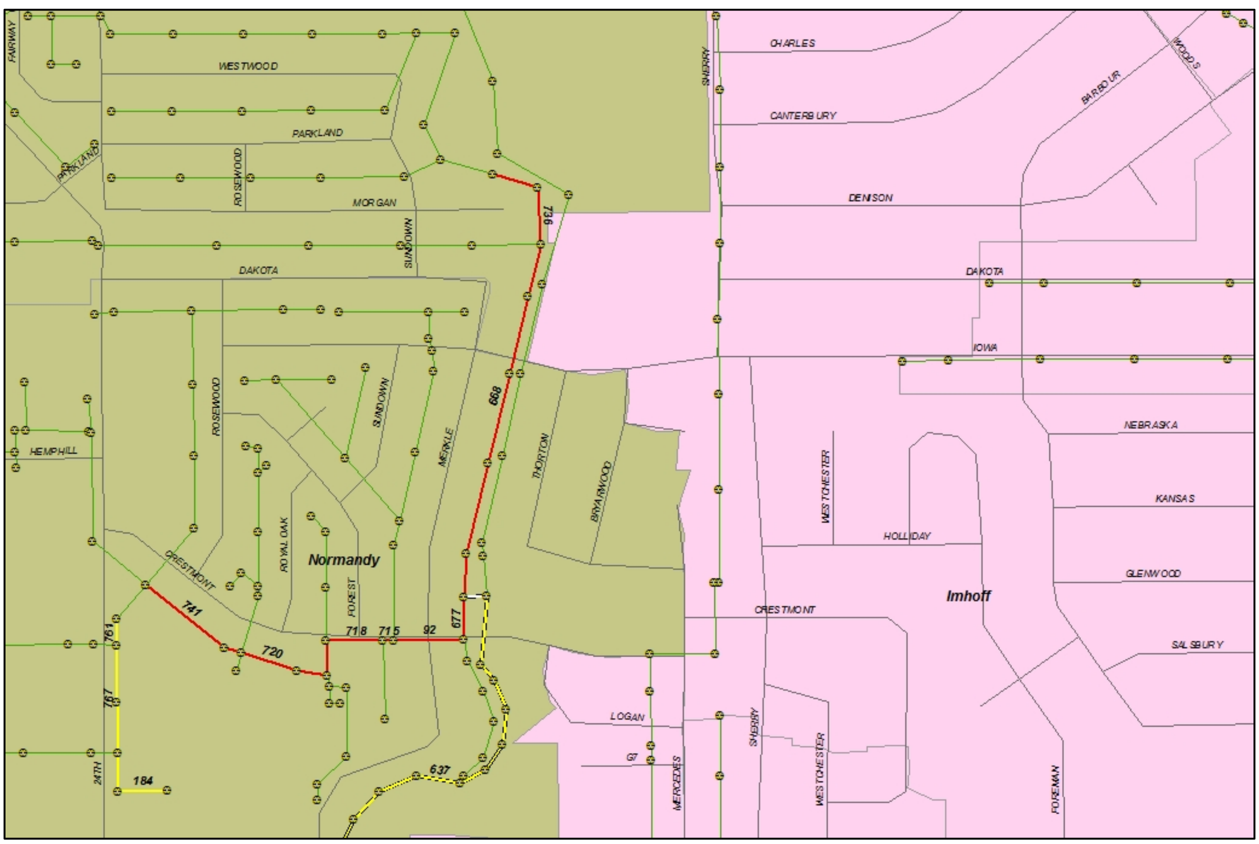

**Figure 4-9B. Normandy Sewer Basin Proposed Piping Improvements**

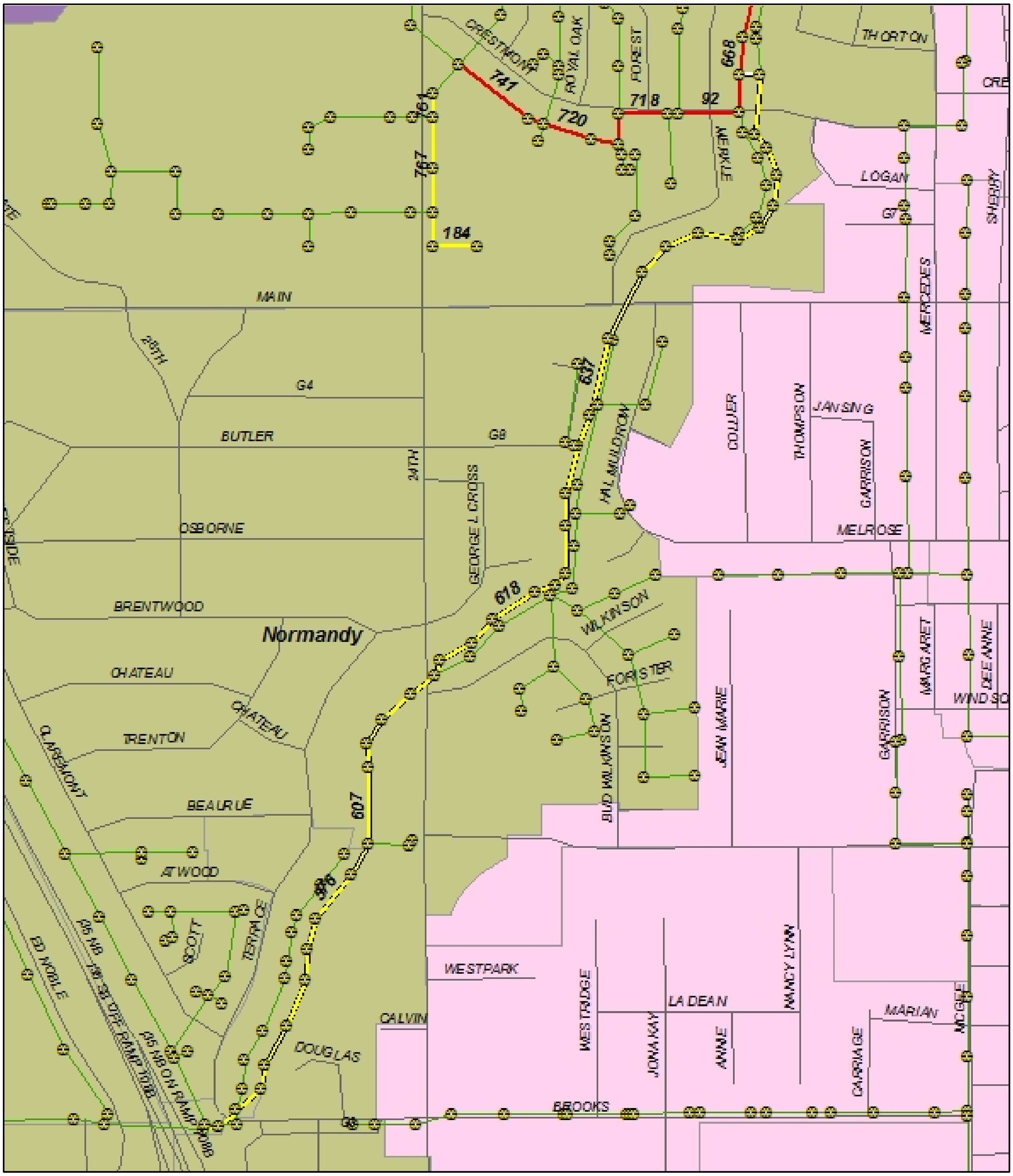

**Table 4-6. Normandy Sewer Basin Proposed Piping Improvements**

| Pipe ID | Asset ID | Branch ID | Pipe Material | L (ft) | Existing Dia (in) | Proposed Dia (in) | Peak After Improvements | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Velocity (ft/s) | Flow (mgd) | Water Depth (ft) |
| 208066.1 | 208066208067 | 92 | UNKN | 301.5 | 8 | 12 | 4.22 | 0.82 | 4.63 |
| 208022.1 | 208022208023 | 184 | VCP | 216.5 | 6 | 8 | -1.51 | -0.24 | 4.39 |
| 208023.1 | 208023208024 | 184 | VCP | 165.7 | 6 | 8 | -1.37 | -0.23 | 5.94 |
| 238087.1 | 238087254080 | 576 | PVC | 169.3 | 21 | 24 | 4.81 | 5.97 | 3.68 |
| 254010.1 | 254010254076 | 576 | PVC | 221.6 | 21 | 24 | 3.76 | 6.01 | 3.15 |
| 254017.1 | 254017254027 | 576 | PVC | 277.1 | 21 | 24 | 5.25 | 5.97 | 2.60 |
| 254027.1 | 254027254020 | 576 | PVC | 74.5 | 21 | 24 | 4.40 | 5.95 | 2.71 |
| 254046.1 | 254046254047 | 576 | PVC | 152.1 | 21 | 24 | 3.67 | 6.00 | 3.93 |
| 254047.1 | 254047254077 | 576 | PVC | 151.4 | 21 | 24 | 3.69 | 6.01 | 3.72 |
| 254076.1 | 254076254017 | 576 | PVC | 110.9 | 21 | 24 | 4.03 | 6.01 | 2.91 |
| 254077.1 | 254077254010 | 576 | PVC | 242.1 | 21 | 24 | 3.88 | 6.01 | 3.29 |
| 254080.1 | 254080254046 | 576 | PVC | 281 | 21 | 24 | 3.66 | 6.00 | 4.12 |
| 238065.1 | 238065238084 | 607 | PVC | 189.2 | 21 | 24 | 4.76 | 5.96 | 3.82 |
| 238084.1 | 238084238085 | 607 | PVC | 136 | 21 | 24 | 3.64 | 5.96 | 3.88 |
| 238085.1 | 238085238086 | 607 | PVC | 122 | 21 | 24 | 3.65 | 5.96 | 3.66 |
| 238086.1 | 238086238087 | 607 | PVC | 369.1 | 21 | 24 | 5.49 | 5.96 | 2.98 |
| 239109.1 | 239109238065 | 607 | PVC | 142.5 | 21 | 24 | 5.93 | 5.75 | 2.71 |
| 239012.1 | 239012239106 | 618 | PVC | 231.8 | 21 | 24 | 3.79 | 5.81 | 2.54 |
| 239015.1 | 239015239016 | 618 | PVC | 106.9 | 21 | 24 | 3.84 | 5.64 | 2.07 |
| 239016.1 | 239016239017 | 618 | PVC | 246.5 | 21 | 24 | 3.93 | 5.63 | 2.01 |
| 239017.1 | 239017239107 | 618 | PVC | 148.6 | 21 | 24 | 4.20 | 5.62 | 1.94 |
| 239106.1 | 239106239125 | 618 | PVC | 163.9 | 21 | 24 | 3.77 | 5.81 | 2.43 |
| 239107.1 | 239107239108 | 618 | PVC | 183 | 21 | 24 | 5.40 | 5.61 | 1.88 |
| 239108.1 | 239108239109 | 618 | PVC | 82.7 | 21 | 24 | 5.74 | 5.59 | 2.10 |
| 239125.1 | 239125239131 | 618 | PVC | 228.6 | 21 | 24 | 3.63 | 5.64 | 2.38 |
| 239131.1 | 239131239015 | 618 | PVC | 74.7 | 21 | 24 | 3.73 | 5.64 | 2.21 |
| 208011.1 | 208011208114 | 637 | PVC | 296 | 18 | 24 | 4.51 | 5.80 | 7.64 |
| 208114.1 | 208114208115 | 637 | PVC | 88.3 | 18 | 24 | 4.51 | 5.79 | 7.56 |
| 208115.1 | 208115208116 | 637 | PVC | 139.4 | 18 | 24 | 4.56 | 5.78 | 6.75 |
| 208116.1 | 208116208117 | 637 | PVC | 149.8 | 18 | 24 | 4.62 | 5.78 | 5.85 |
| 208117.1 | 208117208118 | 637 | PVC | 128.7 | 18 | 24 | 4.68 | 5.78 | 5.07 |
| 208118.1 | 208118208119 | 637 | PVC | 122.6 | 18 | 24 | 4.73 | 5.80 | 4.68 |
| 208119.1 | 208119208120 | 637 | PVC | 192.4 | 18 | 24 | 4.80 | 5.79 | 3.63 |
| 208120.1 | 208120208121 | 637 | PVC | 171.8 | 18 | 24 | 5.18 | 5.79 | 2.79 |
| 208121.1 | 208121208122 | 637 | PVC | 167.7 | 21 | 24 | 4.67 | 5.77 | 2.72 |
| 208122.1 | 208122239130 | 637 | PVC | 363.7 | 27 | 24 | 2.60 | 5.77 | 3.33 |
| 239120.1 | 239120239012 | 637 | PVC | 167 | 21 | 24 | 3.80 | 5.76 | 2.67 |
| 239130.1 | 239130239120 | 637 | PVC | 380.2 | 21 | 24 | 3.65 | 5.76 | 2.84 |
| 208005.1 | 208005208006 | 668 | UNKN | 228.1 | 8 | 12 | 2.77 | 0.65 | 6.93 |
| 208006.1 | 208006208007 | 668 | UNKN | 345.2 | 8 | 12 | 2.64 | 0.63 | 7.18 |
| 208007.1 | 208007208008 | 668 | UNKN | 396.8 | 8 | 12 | 2.76 | 0.61 | 7.23 |
| 208008.1 | 208008208009 | 668 | UNKN | 401.1 | 8 | 12 | 3.03 | 0.61 | 7.43 |
| 208009.1 | 208009208010 | 668 | UNKN | 185.3 | 8 | 12 | 2.28 | 0.60 | 8.08 |
| 208010.1 | 208010208011 | 668 | UNKN | 99.7 | 8 | 15 | 5.91 | 1.33 | 7.71 |
| 208067.1 | 208067208010 | 677 | UNKN | 185.9 | 8 | 12 | 3.41 | 0.83 | 8.07 |
| 208050.1 | 208050208066 | 715 | UNKN | 50.8 | 8 | 12 | 2.83 | 0.68 | 4.61 |
| 208046.1 | 208046208050 | 718 | UNKN | 240.7 | 8 | 12 | 2.73 | 0.66 | 4.50 |
| 208028.1 | 208028208039 | 720 | UNKN | 248.7 | 8 | 12 | 3.23 | 0.86 | 4.40 |
| 208039.1 | 208039208040 | 720 | UNKN | 131.5 | 8 | 12 | 3.31 | 0.86 | 3.78 |
| 208040.1 | 208040208046 | 720 | UNKN | 148.2 | 8 | 12 | 2.59 | 0.65 | 4.03 |
| 192019.1 | 192019192020 | 736 | VCP | 198.6 | 8 | 12 | 2.40 | 0.60 | 4.21 |
| 192020.1 | 192020208005 | 736 | VCP | 246.3 | 8 | 12 | 3.26 | 0.47 | 4.98 |
| 208019.1 | 208019208027 | 741 | UNKN | 430.3 | 8 | 12 | 3.40 | 0.80 | 5.75 |
| 208027.1 | 208027208028 | 741 | UNKN | 79.2 | 8 | 12 | 2.94 | 0.81 | 5.64 |
| 208136.1 | 208136208026 | 761 | VCP | 115.5 | 6 | 8 | 3.16 | 0.45 | 7.57 |
| 208024.1 | 208024208025 | 767 | VCP | 218.1 | 6 | 8 | 3.63 | 0.40 | 5.74 |
| 208025.1 | 208025208136 | 767 | VCP | 245.2 | 6 | 8 | 2.74 | 0.40 | 8.67 |

UNKN – information is unknown

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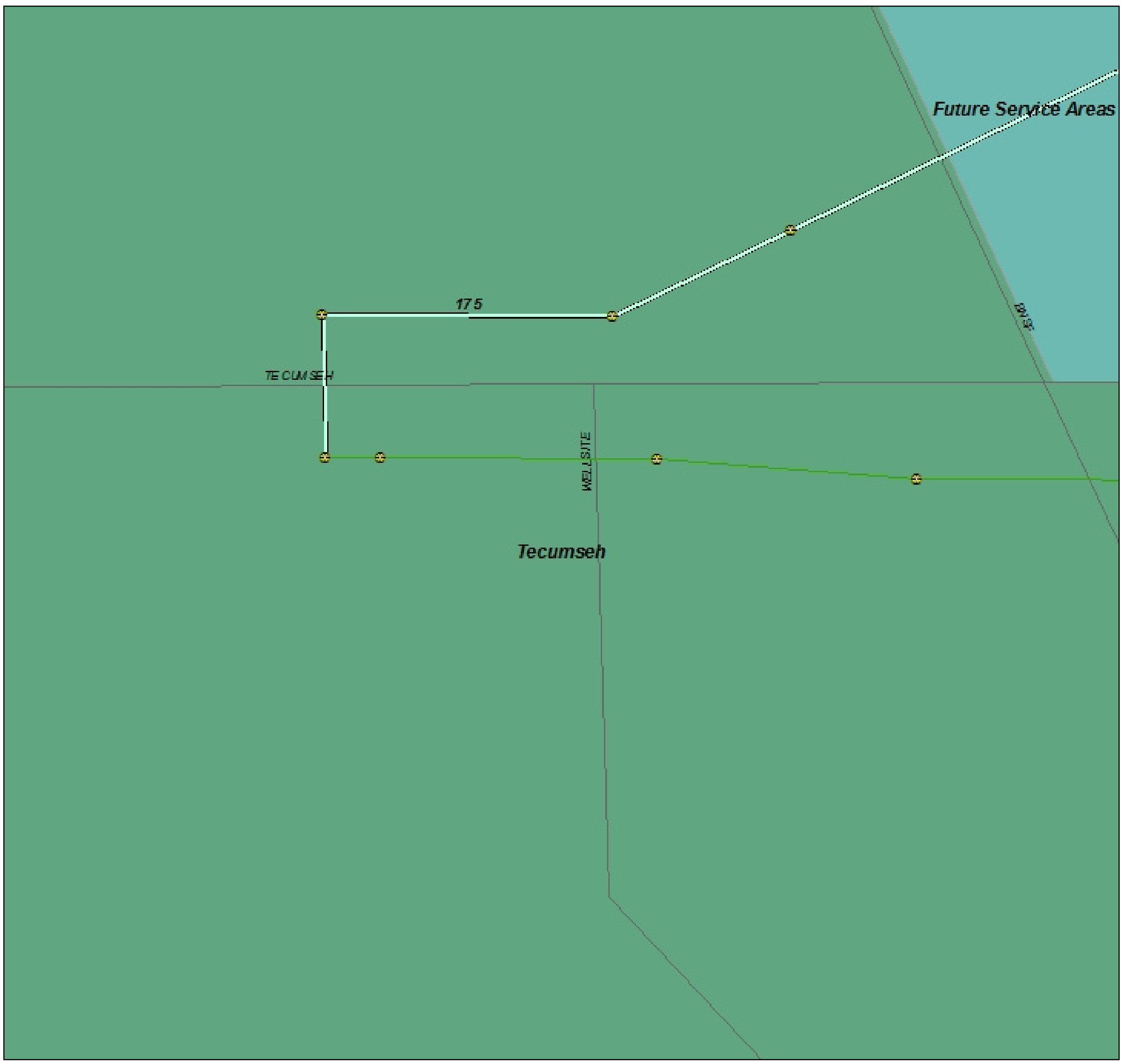
## Tecumseh Sewer Basin

As illustrated on **Figure 3-4**, one node was identified in the Tecumseh Sewer Basin as having less than 1 foot of freeboard in the manhole. No SSOs occurred under the FBO 2030 design storm scenario.

Approximately 723 LF (0.14 miles) of sewer improvements, shown on **Figure 4-3** and below on **Figure 4-10**, are identified in order to eliminate the identified deficiency at the node to meet the required design criteria previously described.

**Figure 4-10. Tecumseh Sewer Basin Proposed Piping Improvements**



The individual pipe segments are listed in **Table 4-7**. The information presented includes the pipe ID (with corresponding upstream manhole), Branch ID (corresponding to **Figure 4-10**) that identifies the grouping, the pipe asset identification number, the existing diameter, the proposed diameter, the peak velocity, the peak flow, and the peak water depth. Peak values represent the maximum value observed during the design storm event in the model analysis assuming the piping improvements have been applied.

**Table 4-7. Tecumseh Sewer Basin Proposed Piping Improvements**

| Pipe ID | Asset ID | Branch ID | Pipe Material | L (ft) | Existing Dia (in) | Proposed Dia (in) | Peak After Improvements | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Velocity (ft/s) | Flow (mgd) | Water Depth (ft) |
| 101028.1 | 10102875013 | 175 | UNKN | 100.6 | 12 | 18 | 4.59 | 2.66 | 8.19 |
| 75013.1 | 75013075017 | 175 | UNKN | 205 | 12 | 18 | 4.79 | 2.64 | 6.31 |
| 75017.1 | 75017075015 | 175 | UNKN | 140 | 12 | 18 | 4.91 | 2.63 | 5.30 |
| 75015.1 | 7501575016 | 175 | UNKN | 278 | 12 | 18 | 5.66 | 2.61 | 4.76 |

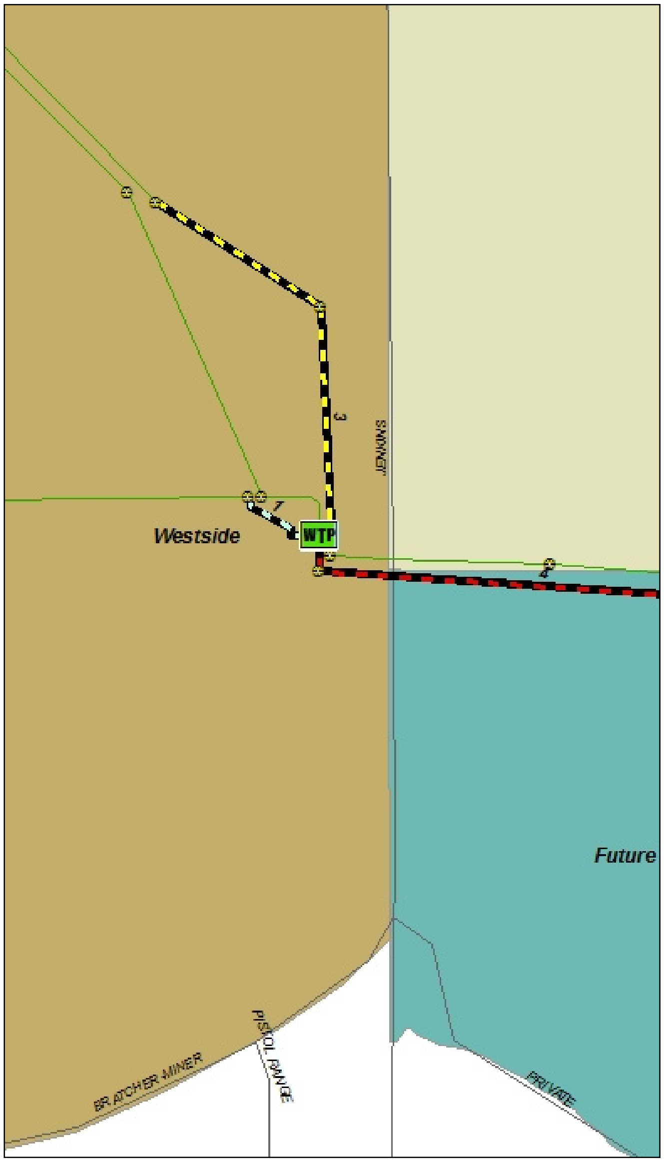
UNKN – information is unknown

## Westside Sewer Basin

As illustrated on **Figure 3-4**, a total of 2 nodes are identified in the Westside Sewer Basin as having less than 1 foot of freeboard in the manhole. Both of these nodes are predicted to have an SSO resulting in an approximate overflow quantity of 21,218 gallons under the design storm and FBO 2030 condition.

Approximately 1,537 LF (0.3 mi) of sewer improvements shown on **Figure 4-3** and below on **Figure 4-11** are identified in order to eliminate the identified nodes and SSO locations to meet the required design criteria previously described.

**Figure 4-11. Westside Sewer Basin Proposed Piping Improvements**




The individual pipe segments are listed in **Table 4-8**. The information presented includes the pipe ID (with corresponding upstream manhole), Branch ID (corresponding to **Figure 4-11**) that identifies the grouping, the pipe asset identification number, the existing diameter, the proposed diameter, the peak velocity, the peak flow, and the peak water depth. Peak values represent the maximum value observed during the design storm event in the model analysis assuming the piping improvements have been applied.

**Table 4-8. Westside Sewer Basin Proposed Piping Improvements**

| Pipe ID | Asset ID | Branch ID | Pipe Material | L (ft) | Existing Dia (in) | Proposed Dia (in) | Peak After Improvements | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Velocity (ft/s) | Flow (mgd) | Water Depth (ft) |
| 328013.1 | 328013328056 | 4 | DIP | 54.6 | 48 | 54 | 7.5 | 33.6 | 5.1 |
| 328051.1 | 328051328052 | 3 | PVC | 302.3 | 42 | 48 | 4.3 | 21.0 | 5.4 |
| 328052.1 | 328052328653 | 3 | PVC | 342.8 | 42 | 48 | 6.7 | 24.6 | 5.3 |
| 328055.1 | 328055328056 | 1 | DIP | 142.3 | 24 | 36 | 10.7 | 25.0 | 11.1 |
| 329024.1 | 329024328013 | 4 | FRP | 695.4 | 48 | 54 | 5.9 | 32.6 | 5.5 |

UNKN – information is unknown

## York Sewer Basin

As illustrated on **Figure 3-4**, a total of 5 nodes are identified in the York Sewer Basin as having less than 1 foot of freeboard in the manhole. Three of these nodes are predicted to have an SSO resulting in an approximate overflow quantity of 141,560 gallons under the design storm and FBO 2030 condition.

Approximately 5,810 LF (1.1 mi) of sewer improvements shown on **Figure 4-3** and below in **Figure 4-12** are identified in order to eliminate the identified nodes and SSO locations to meet the required design criteria previously described.

The individual pipe segments are listed in **Table 4-9**. The information presented includes the map ID (corresponding to **Figure 4-12**), the pipe asset identification number, the existing diameter, the proposed diameter, the peak velocity, the peak flow, and the peak water depth. Peak values represent the maximum value observed during the storm event in the model analysis assuming the piping improvements have been applied. In general, 12- inch diameter pipe needs to be upsized to 18-inch pipe to convey the FBO 2030 design storm flow. Some 8-inch pipe needs to be upsized to 18-inches to convey the flow.

The York Lift Station located in the York Sewer Basin was modeled as inactive, meaning this lift station is no longer in service.

**Figure 4-12. York Sewer Basin Proposed Piping Improvements**

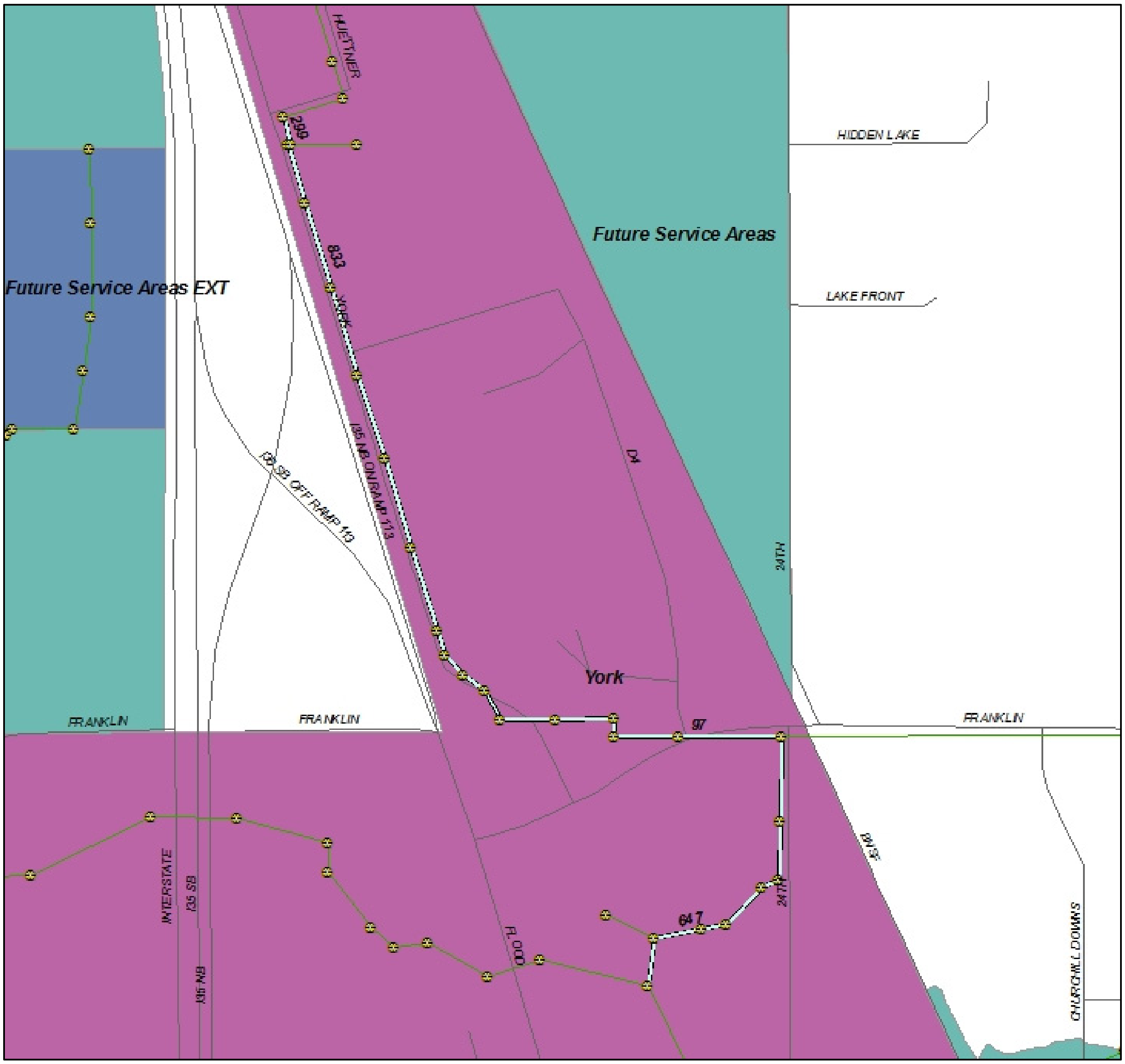


**Table 4-9. York Sewer Basin Proposed Piping Improvements**

| Pipe ID | Asset ID | Branch ID | Pipe Material | L (ft) | Existing Dia (in) | Proposed Dia (in) | Peak After Improvements | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Velocity (ft/s) | Flow (mgd) | Water Depth (ft) |
| 44010.1 | 044010044011 | 97 | PVC | 255 | 18 | 18 | 2.4 | 0.9 | 11.8 |
| 44011.1 | 044011070001 | 97 | PVC | 259 | 12 | 18 | 3.3 | 1.0 | 12.9 |
| 70001.1 | 070001070034 | 97 | VCP | 85 | 8 | 18 | 2.9 | 1.0 | 12.9 |
| 70031.1 | 070031070032 | 97 | VCP | 461 | 8 | 18 | 3.5 | 1.0 | 11.2 |
| 70032.1 | 070032070033 | 97 | VCP | 386 | 8 | 18 | 4.9 | 1.2 | 4.4 |
| 70033.1 | 070033070006 | 97 | VCP | 263 | 8 | 18 | 6.7 | 1.4 | 0.6 |
| 70034.1 | 070034070031 | 97 | VCP | 294 | 8 | 18 | 3.1 | 1.0 | 9.8 |
| 42010.1 | 042010042011 | 299 | PVC | 128 | 12 | 18 | -1.6 | -1.0 | 8.5 |
| 70035.1 | 070035070036 | 647 | PVC | 217 | 12 | 18 | 2.8 | 1.5 | 1.3 |
| 70051.1 | 070051070052 | 647 | PVC | 85 | 12 | 18 | 3.8 | 1.5 | 0.7 |
| 70052.1 | 070052070053 | 647 | PVC | 229 | 12 | 18 | 4.2 | 1.5 | 0.6 |
| 70053.1 | 070053070054 | 647 | PVC | 112 | 12 | 18 | 4.9 | 1.5 | 0.6 |
| 70054.1 | 070054070035 | 647 | PVC | 218 | 12 | 18 | 5.1 | 1.5 | 0.6 |
| 42008.1 | 042008044001 | 833 | PVC | 278 | 12 | 18 | 3.2 | 1.1 | 8.9 |
| 42011.1 | 042011042008 | 833 | PVC | 17 | 12 | 18 | 2.1 | 1.1 | 8.7 |
| 44001.1 | 044001044002 | 833 | PVC | 399 | 12 | 18 | 2.6 | 1.0 | 9.9 |
| 44002.1 | 044002044003 | 833 | PVC | 408 | 12 | 18 | 3.3 | 0.8 | 10.6 |
| 44003.1 | 044003044004 | 833 | PVC | 400 | 12 | 18 | 1.3 | 0.7 | 11.6 |
| 44004.1 | 044004044005 | 833 | PVC | 418 | 12 | 18 | 2.9 | 1.3 | 11.0 |
| 44005.1 | 044005044006 | 833 | PVC | 395 | 12 | 18 | 2.6 | 1.3 | 10.8 |
| 44006.1 | 044006044007 | 833 | PVC | 110 | 12 | 18 | 2.5 | 1.3 | 10.8 |
| 44007.1 | 044007044008 | 833 | PVC | 127 | 12 | 18 | 2.4 | 1.3 | 10.7 |
| 44008.1 | 044008044009 | 833 | PVC | 118 | 12 | 18 | 3.5 | 1.0 | 10.7 |
| 44009.1 | 044009044010 | 833 | PVC | 149 | 18 | 18 | 2.6 | 1.0 | 11.1 |

UNKN – information is unknown

### 4.3.2 Lift Station and Force Main Evaluation

A summary of the modeled lift station conditions is shown in **Table 4-10**.

**Table 4-10. Existing Lift Station Model Conditions**

| Lift Station Name | Lift Station ID | Pump 1 Operating Point[2] (mgd @ TDH, ft) | On Level 1 (ft)[1] | On Level 2 (ft)[3] | On Level 3 (ft)[3] |
|---|---|---|---|---|---|
| Alameda Park | 215701-Wetwell | 0.12 @ 39.3 | 1170.97 | 1171.09 | - |
| Ashton Grove | 111700-Wetwell | 1.01 @ 72 | 1111.60 | 1111.90 | - |
| Eagle Cliff | 357700-Wetwell | 0.26 @ 62 | 1076.60 | 1078.00 | - |
| East Ridge | 289700-Wetwell | 0.43 @ 43 | 1142.50 | 1144.50 | 1146.50 |
| Hall Park North | 154001-Wetwell | 0.87 @ 125 | 1123.50 | 1124.50 | - |
| Hall Park South | 168004-Wetwell | 0.65 @ 136.8 | 1139.00 | 1140.50 | - |
| Lift Station D | 79700-Wetwell | 6.0 @ 281 | See Note 4 | - | - |
| Park Hill | 107038-Wetwell | 0.21 @ 120 | 1153.50 | 1154.50 | - |
| Post Oak | 360701-Wetwell | 1.01 @ 90 | 1121.00 | 1122.00 | - |
| Royal Oaks | 216700-Wetwell | 0.87 @ 84.5 | 1139.75 | 1142.90 | - |
| Siena Springs | 264700-Wetwell | 0.21 @ 35 | 1179.50 | 1180.50 | - |
| Summit Lakes | 248701-Wetwell | 0.25 @ 101 | 1172.70 | 1173.70 | - |
| Summit Valley | 301700-Wetwell | 1.12 @ 72 | 1112.02 | 1112.50 | - |
| Sutton Place | 165700-Wetwell | 0.30 @ 76 | 1167.00 | 1167.00 | - |
| Vo-Tech | 55700-Wetwell | 0.21 @ 50 | 1138.14 | 1138.64 | - |
| Westside | 328700-Wetwell | 8.96 @ 41.3 | 1080.50 | 1081.20 | 1081.20 |

[1]Below Level 1, the pump rate is zero.
[2]Between Level 1 and Level 2, the pump rate is Flow Rate 1. Operating point is the flow rate which pump operates at in the model and may differ from the pump's actual design point.
[3]Above Level 2 and Level 3, Lag 1 (and Lag 2, if available) are operating
[4]Lift Station D operates at a maximum of 6 mgd and maintains a set wet well level.
[5]Inactive lift stations are considered abandoned and were not included in the modeled condition.

The lift station capacity was evaluated for the design storm and FBO 2030 loading condition against the existing station firm and total pumping capacity. Firm pumping capacity is defined as having all pumps but one available for operation.

**Table 4-11** summarizes the lift station capacity evaluation in terms of peak 2-hour flow and average design flow. Lift stations are typically designed for the peak 2-hour flow condition; however ODEQ requires only the effective volume of the wet well to be based on design average flow and a filling time not to exceed 30 minutes unless the facility is designed to provide flow equalization.

**Table 4-11. Existing Lift Station and Force Main Capacity Evaluation**

| Lift Station Name | Existing Capacity | | Peak 2-Hour Flow[2] | | | | Average Design Flow[3] | | Force Main | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Total (mgd) | Firm[1] (mgd) | Predicted (mgd) | Remaining Firm Capacity (mgd) | Remaining Total Capacity (mgd) | Water Depth from Wetwell Rim (ft) | Predicted (mgd) | Remaining Firm Capacity (mgd) | Existing Diameter (in) | Velocity at P2H Flow (ft/sec) | Velocity at Avg. Design Flow (ft/sec) |
| Alameda Park | 0.24 | 0.12 | 0.11 | 0.01 | 0.13 | -19.5 | 0.03 | 0.09 | 4 | 2.7 | 2.3 |
| Ashton Grove | 2.02 | 1.01 | 0.56 | 0.45 | 1.46 | -23.2 | 0.19 | 0.82 | 12 | 3.5 | 2.7 |
| Eagle Cliff | 0.52 | 0.26 | 0.07 | 0.19 | 0.45 | -11.5 | 0.04 | 0.22 | 6 | 3.2 | 2.3 |
| East Ridge | 2.34 | 0.78 | 0.61 | 0.17 | 1.73 | -13.0 | 0.14 | 0.64 | 8 | **9.9** | 5.4 |
| Hall Park North | 1.74 | 0.87 | 1.47 | **-0.60** | 0.27 | -7.7 | 0.40 | 0.47 | 8 | 6.3 | 4.6 |
| Hall Park South | 1.30 | 0.65 | 2.33 | **-1.68** | **-1.03** | -11.3 | 0.61 | 0.04 | 12 | 4.7 | 3.2 |
| Lift Station D | 6.00 | 6.00 | 21.14 | -15.14[4] | -15.14[4] | -13.5 | 4.88 | 1.12 | 16 | 6.6 | 5.4 |
| Park Hill | 0.42 | 0.21 | 0.08 | 0.13 | 0.34 | -10.4 | 0.04 | 0.17 | 4 | 4.0 | 3.1 |
| Post Oak | 2.02 | 1.01 | 0.58 | 0.43 | 1.44 | -10.6 | 0.18 | 0.83 | 12 | 3.3 | 2.2 |
| Royal Oaks | 1.74 | 0.87 | 1.52 | **-0.65** | 0.22 | -8.0 | 0.39 | 0.48 | 12 | 3.4 | 2.2 |
| Siena Springs | 0.42 | 0.21 | 0.18 | 0.03 | 0.24 | -8.9 | 0.04 | 0.17 | 8 | 4.9 | 2.7 |
| Summit Lakes | 0.50 | 0.25 | 0.15 | 0.10 | 0.35 | -10.8 | 0.04 | 0.21 | 4 | 4.3 | 3.7 |
| Summit Valley | 2.24 | 1.12 | 0.44 | 0.68 | 1.80 | -14.7 | 0.22 | 0.90 | 12 | 3.0 | 2.2 |
| Sutton Place | 0.59 | 0.30 | 0.62 | **-0.33** | **-0.03** | -7.0 | 0.03 | 0.26 | 6 | 5.2 | 4.6 |
| Vo-Tech | 0.42 | 0.21 | 0.72 | **-0.51** | **-0.30** | -16.4 | 0.09 | 0.12 | 10 | 3.0 | 1.6 |
| Westside | 26.88 | 17.92 | 20.14 | **-2.22** | 6.74 | -15.8 | 6.28 | 11.64 | 24 | **11.9** | 4.4 |

[1]Firm Capacity based on Operating Point listed in Table 4-10

[2]P2HF is the predicted peak two-hour flow under the design storm and FBO 2030 condition

[3]Average Design Flow is the predicted average flow under the design storm and FBO 2030 condition

[4]While the firm pumping capacity of Lift Station D is only 6 mgd, sufficient equalization storage is available to handle the excess peak two-hour flow and is integral to the design of the station and equalization storage. Thus, the facility as a whole currently has the capacity to handle P2H FBO 2030 flows.

## Lift Station Capacity Evaluation

**FBO 2030 Average Design Flow:** All of the lift stations in the City's system currently have the firm capacity to handle FBO 2030 average design flows.

**Firm Capacity at FBO 2030 Peak 2-hour Flow:** Six lift stations (Hall Park North, Hall Park South, Royal Oaks, Sutton Place, Vo-Tech, and Westside) are predicted to receive FBO 2030 P2H flows in excess of their firm pumping capacities (see bolded values in **Table 4-11**). However, as shown in the table, none of the lift station wetwells overflow under P2H flows. The upstream gravity piping water levels rise slightly to handle this additional flow without creating overflows in the upstream manholes. As a results, the firm capacity for all of the lift stations is adequate for FBO 2030 P2H flows.

**Total Capacity at FBO 2030 Peak 2-hour Flow:** Three lift stations (Hall Park South, Sutton Place, and Vo-Tech) are predicted to receive FBO 2030 P2H flows in excess of their total pumping capacities (see bolded values in **Table 4-11**). However, as discussed above, the upstream gravity piping has the capacity to store the additional flow and the existing lift station capacities are adequate.

## Force Main Evaluation

**FBO 2030 Average Design Flow:** For FBO 2030 average design flows, the velocities in all of the force mains are below 6 feet per second, which is less than the stated evaluation criterion maximum of 7 feet per second.

**FBO 2030 Peak 2-hour Flow:** For FBO 2030 P2H flows, the force mains at two lift stations (East Ridge and Westside) experience velocities higher than the evaluation criteria (see bolded values in **Table 4-11**). In both cases, however, the velocities are not high enough to create excessive headloss in the system, and the pumps are able to convey the necessary flow. It is not recommended that these two force mains be upsized. However, the City many want to consider paralleling the Westside force main in the future to provide additional reliability since it is a critical facility at the existing Norman WRF.

## Recommendations

Based on this evaluation, there are no lift station improvements necessary to meet the FBO 2030 flows. Also, all of the existing force mains are adequately sized to convey these flows, so no force main improvements are needed.

The analysis identified two lift stations that could be eliminated to simplify system operations:

- The Siena Springs Lift Station could be eliminated by consolidating flow with the East Ridge Lift Station as proposed by the developer of the Stone Lake Subdivision. The East Ridge Lift Station has the remaining capacity to handle this additional flow.
- The Vo-Tech Lift Station could be eliminated by constructing a gravity sewer to the Little River Interceptor.

### 4.3.3 Estimated Construction Costs

A summary of the estimated capital costs for the recommended improvements to the existing collection system shown on **Figure 4-3** is listed in **Table 4-12**. For the purpose of this planning level cost estimate, it is assumed that the gravity sewers are being replaced with the proposed diameters rather than installing a parallel sewer. Evaluating whether to construct parallel sewers versus replacement is typically performed during the design phase of an individual project to determine the most cost-effective approach.

**Table 4-12. Estimated Gravity System Improvement Costs for the One-Plant Scenario**

| Sewer Basin | Length of Pipe (ft) | Length of Pipe (mi) | Estimated Construction Cost | Sewer Shed |
|---|---|---|---|---|
| LS D | 14,583 | 2.8 | $2,898,912 | North |
| Tecumseh | 724 | 0.2 | $132,079 | North |
| York | 5,810 | 1.1 | $1,060,426 | North |
| **Subtotal North** | **21,117** | **4.0** | **$4,091,417** | |
| Bishop | 24,223 | 4.6 | $5,929,482 | South |
| Brookhaven | 6,607 | 1.3 | $1,031,627 | South |
| Imhoff | 1,748 | 0.3 | $247,990 | South |
| Normandy | 11,352 | 2.2 | $2,106,138 | South |
| Westside | 1,537 | 0.3 | $608,788 | South |
| **Subtotal South** | **45,469** | **8.6** | **$9,924,025** | |
| **Total** | **66,586** | **12.6** | **$14,015,442** | |

### 4.3.4 Norman WRF Design Flow

Predicted wastewater flows occurring under a 5-year, 4-hour design storm event using the FBO 2030 loading condition, with the improvements in place, were determined as shown in **Figure 4-1**. The wastewater flows were summed to determine the expected flow to the Norman WRF. The summary of these flows is included in **Table 4-13**.

**Table 4-13. Predicted Norman WRF Wastewater Flows for One-Plant Scenario**

| Parameter | Description | Historical Data | Predicted for One-Plant Scenario, FBO 2030 |
|---|---|---|---|
| Flow (mgd) | Average Dry Weather[1] | 10.5 | 18.6 |
| | Annual Average[2] | 11.1 | 18.3[6] |
| | Annual Average Plus Planning Capacity[3] | 11.7 | 19.2 |
| | Maximum Month[4] | 12.6 | - |
| | Maximum Month + Planning Capacity | 13.2 | - |
| | Maximum Day[5] | 21.3 | 33.5 |
| | Peak 2-hour | - | 70.8 |

[1]Review of wastewater treatment plant influent record date from May 2010 to September 2010, dry weather only, to coincide with the flow monitoring period.

[2]Review of wastewater treatment plant influent record data from October 2009 to September 2010. This is an annual average of all monthly data and includes dry and wet weather flow events.

[3]Planning capacity = 5%.

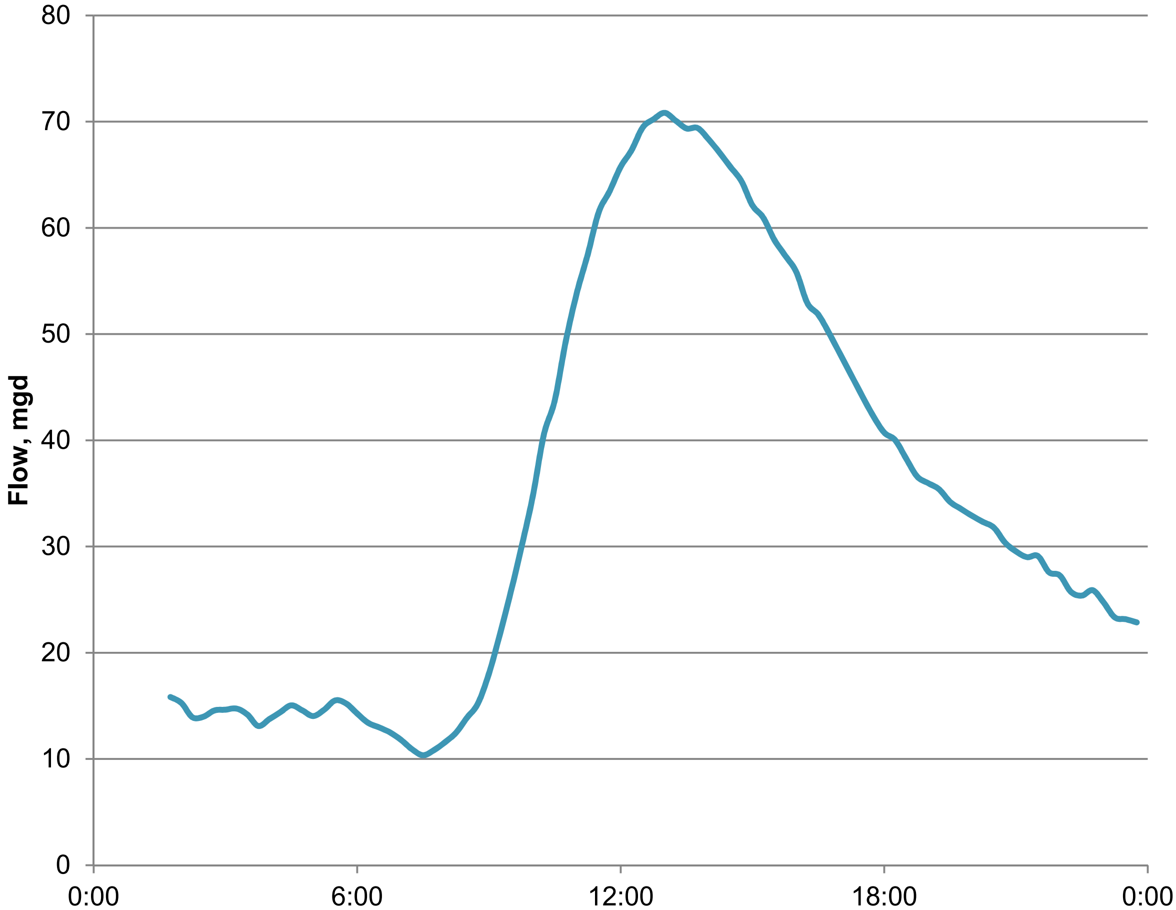
[4]Review of monthly flow volume totals from October 2009 to September 2010. The maximum month flow was identified and divided by 30 days.

[5]Review of daily wastewater plant influent record data from October 2009 to September 2010.

[6]Predicted annual average = predicted average dry weather/0.98

The predicted wet weather flow curve is shown on **Figure 4-13**.

**Figure 4-13. Predicted Norman WRF Flow Curve for One-Plant Scenario for Peak 2-Hour FBO 2030**



## 4.4 Two-Plant Scenario

This section describes the evaluation with the system flow at a FBO 2030 loading condition under a 5-year, 4-hour storm event being conveyed as follows:

- All North Basin flow conveyed from Lift Station D to a proposed North WRF.
- All South Basin flow conveyed to the existing Norman WRF.

### 4.4.1 Pipeline System Evaluation

With the 6 mgd maximum flow from Lift Station D contributing flow to the South Sewer Shed removed, the model was run to predict capacity and SSO issues (as defined in Chapter 3) during a 5-year, 4-hour design storm event when the FBO 2030 loading is applied to the existing sewer system.

Fifty (50) total locations were identified as not meeting the design capacity criteria. These locations are shown on **Figure 4-14** and listed in **Table 4-14**. Of the 50 locations, 23 manholes are identified as SSO locations resulting in a predicted overflow volume of approximately 0.45 million gallons (MG). Twenty-seven (27) locations are identified as having less than one foot of freeboard available.

The model output data for the design storm and FBO 2030 loading condition for all conduits (pipe segments) after all the improvements described herein are implemented are included in **Appendix 4E**.

Very little difference exists between the One-Plant Scenario data (**Table 3-3**) and the Two-Plant Scenario data shown in **Table 4-14**. In the Two-Plant Scenario, the 6 mgd from Lift Station D is redirected to the proposed North WRF and not discharged into the south sewer shed via the Bishop Sewer Basin (as currently performed).

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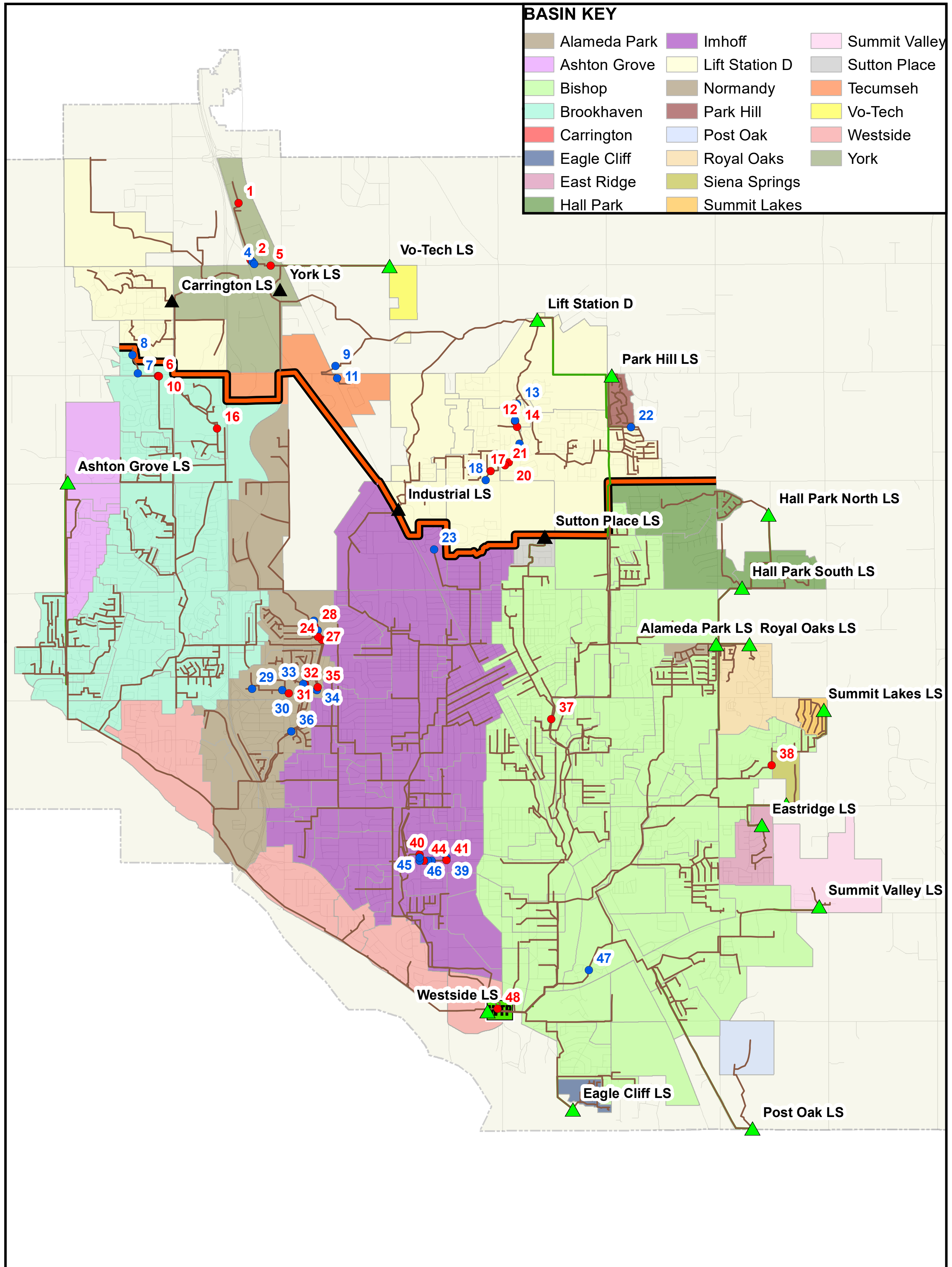
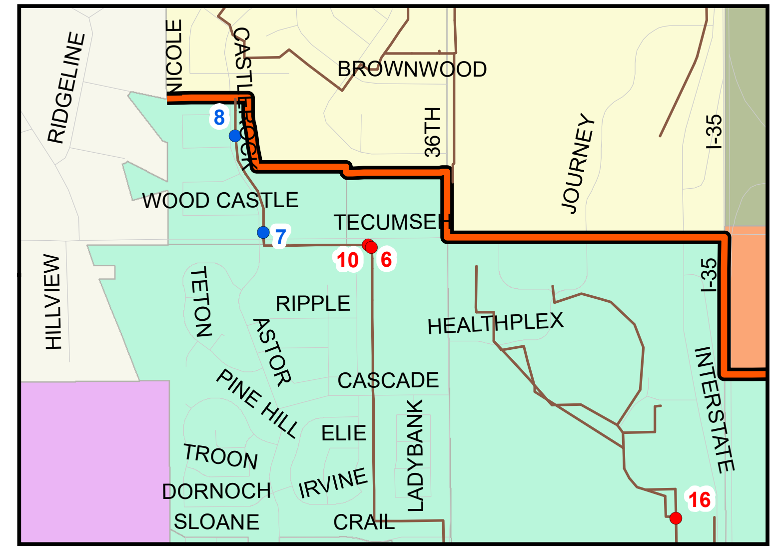
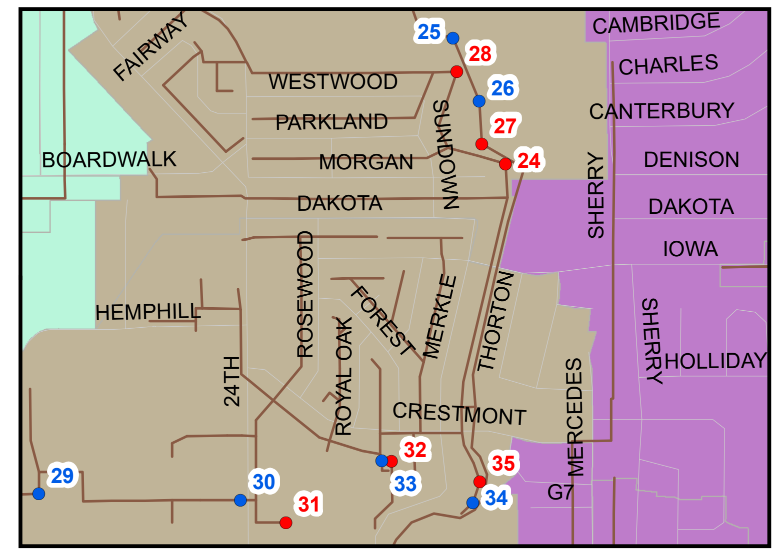



PATH: C:\USERS\TWHITFIE\DOCUMENTS_WORKING DOCUMENTS_PROJECTS_ACTIVE\NORMAN_NORMAN_GIS\MAP_DOCS\MXD\2017_FIGURE_RECREATION\FIGURE4-14.MXD - USER: TWHITFIE - DATE: 11/1/2017

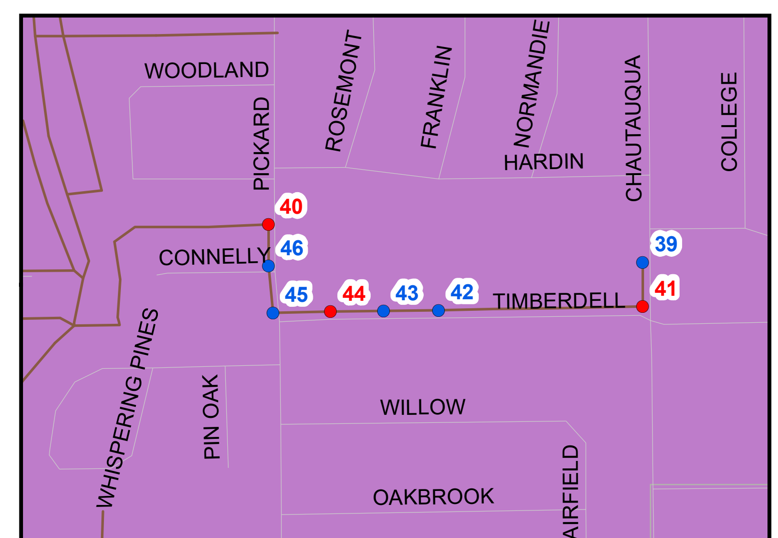
INSET 1




INSET 2

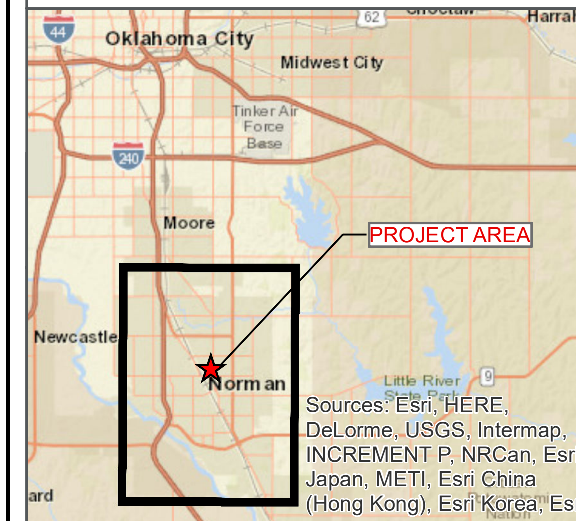



INSET 3




# CITY OF NORMAN FLOW MONITORING AND MODELING

CAPACITY ISSUE LOCATIONS
TWO PLANT SCENARIO
FBO 2030 5-YEAR 4-HOUR STORM
FLOW CONDITION




Sources: Esri, HERE, DeLorme, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri

## MODELED SYSTEM LEGEND

- Inactive Lift Station
- Active Lift Station
- FB
- SSO
- Conduit
- WTP Outfall
- Force Main
- Flow Division Line
- City Boundary
- Street Centerlines

SOURCE: CITY OF NORMAN INFRASTRUCTURE AS OF APRIL 2012

INSET MAPS NOT TO SCALE

N

NORMAN UTILITIES AUTHORITY

HDR ENGINEERING, INC.
17111 PRESTON ROAD, SUITE 300
DALLAS, TEXAS 75248-1232
(972) 960-4400

FIGURE 4-14

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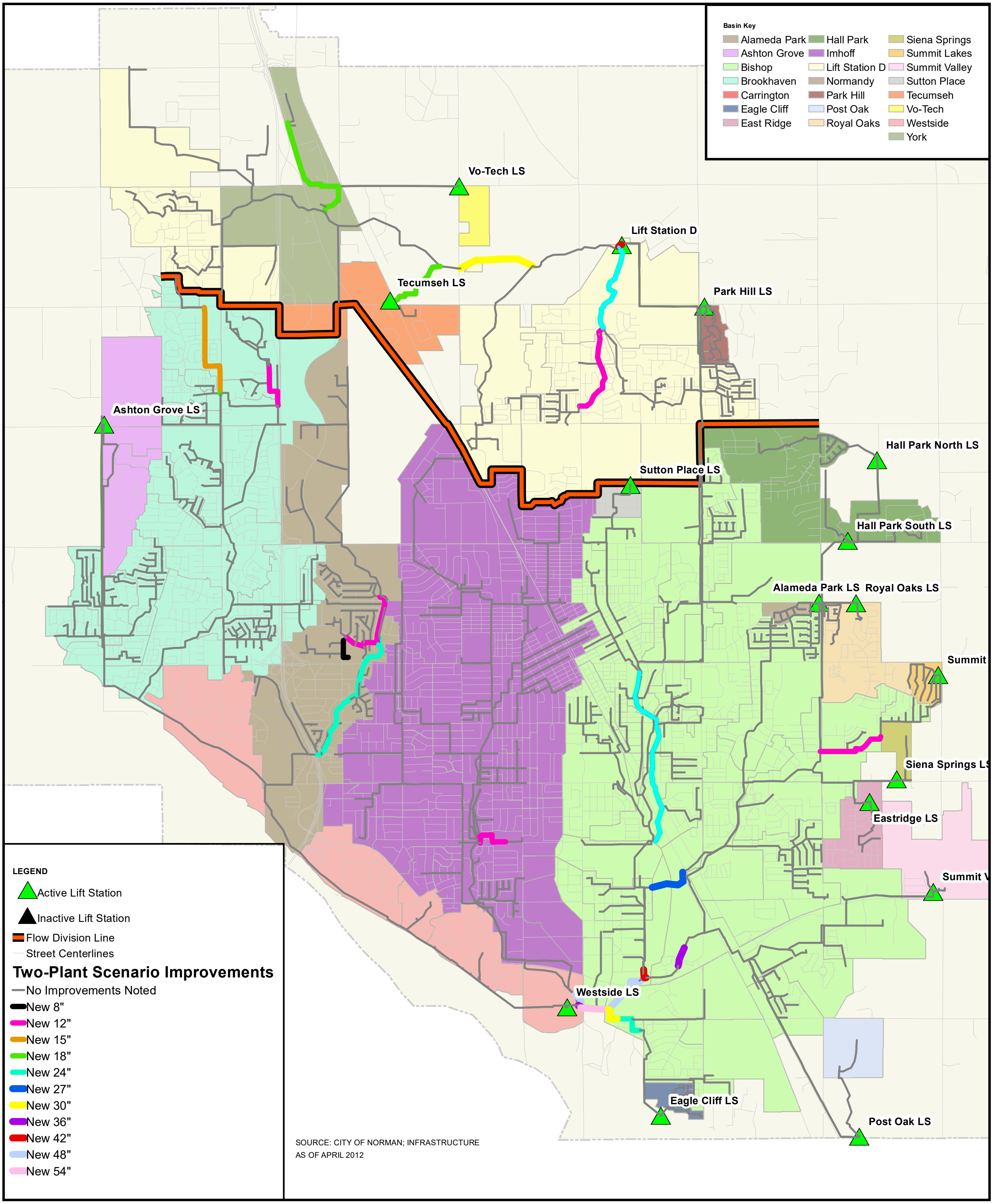
### Table 4-14. Design Storm Capacity Issues at FBO 2030 – Two-Plant Scenario

| Map ID | Sewer Basin | Manhole ID | Rim Elevation | Maximum Water Level Elevation | Freeboard to Manhole Rim (ft) | SSO (Y or N) | Estimated Overflow (gallons) |
|---|---|---|---|---|---|---|---|
| 37 | Bishop | 243056 | 1141.00 | 1141.00 | 0.00 | Y | 6,600 |
| 38 | Bishop | 263037 | 1208.88 | 1208.88 | 0.00 | Y | 45,400 |
| 47 | Bishop | 321075 | 1116.67 | 1116.67 | -0.75 | N | 0 |
| | | | | | | **Subtotal Bishop** | **52,000** |
| 6 | Brookhaven | 72002 | 1188.24 | 1188.24 | 0.00 | Y | 42,100 |
| 7 | Brookhaven | 72008 | 1192.55 | 1192.55 | -0.91 | N | 0 |
| 8 | Brookhaven | 72024 | 1192.78 | 1192.78 | -0.22 | N | 0 |
| 10 | Brookhaven | 98065 | 1188.19 | 1188.19 | 0.00 | Y | 100 |
| 16 | Brookhaven | 113008 | 1180.03 | 1180.03 | 0.00 | Y | 9,600 |
| | | | | | | **Subtotal Brookhaven** | **51,800** |
| 23 | Imhoff | 162002 | 1191.41 | 1191.41 | -0.59 | N | 0 |
| 39 | Imhoff | 283037 | 1152.00 | 1152.00 | 0.00 | N | 0 |
| 40 | Imhoff | 283118 | 1144.00 | 1144.00 | 0.00 | Y | 12,200 |
| 41 | Imhoff | 283125 | 1151.24 | 1151.24 | 0.00 | Y | 1000 |
| 42 | Imhoff | 283128 | 1147.80 | 1147.80 | -0.28 | N | 0 |
| 43 | Imhoff | 283129 | 1146.91 | 1146.91 | -0.52 | N | 0 |
| 44 | Imhoff | 283130 | 1146.00 | 1146.00 | 0.00 | Y | 1,500 |
| 45 | Imhoff | 283131 | 1145.28 | 1145.28 | -0.45 | N | 0 |
| 46 | Imhoff | 283132 | 1144.63 | 1144.63 | -0.76 | N | 0 |
| | | | | | | **Subtotal Imhoff** | **14,700** |
| 9 | Lift Station D | 76008 | 1143.34 | 1143.34 | -0.66 | N | 0 |
| 12 | Lift Station D | 105012 | 1134.00 | 1134.00 | 0.00 | Y | 8,200 |
| 13 | Lift Station D | 105014 | 1127.13 | 1127.13 | -0.87 | N | 0 |
| 14 | Lift Station D | 105039 | 1136.50 | 1136.50 | 0.00 | Y | 7,700 |
| 15 | Lift Station D | 105043 | 1133.62 | 1133.62 | -0.88 | N | 0 |
| 17 | Lift Station D | 118008 | 1158.86 | 1158.86 | 0.00 | Y | 1,600 |
| 18 | Lift Station D | 118015 | 1162.12 | 1162.12 | -0.63 | N | 0 |
| 19 | Lift Station D | 119085 | 1142.10 | 1142.10 | -0.36 | N | 0 |
| 20 | Lift Station D | 119092 | 1148.42 | 1148.42 | 0.00 | Y | 20,500 |
| 21 | Lift Station D | 119093 | 1150.19 | 1150.19 | 0.00 | Y | 9,400 |
| | | | | | | **Subtotal Lift Station D** | **47,400** |
| 24 | Normandy | 192020 | 1147.10 | 1147.10 | 0.00 | Y | 23,800 |
| 25 | Normandy | 192029 | 1151.86 | 1151.86 | -0.48 | N | 0 |

| Map ID | Sewer Basin | Manhole ID | Rim Elevation | Maximum Water Level Elevation | Freeboard to Manhole Rim (ft) | SSO (Y or N) | Estimated Overflow (gallons) |
|---|---|---|---|---|---|---|---|
| 26 | Normandy | 192030 | 1149.77 | 1149.77 | -0.39 | N | 0 |
| 27 | Normandy | 192031 | 1148.29 | 1148.29 | 0.00 | Y | 3,000 |
| 28 | Normandy | 192033 | 1151.73 | 1151.73 | 0.00 | Y | 100 |
| 29 | Normandy | 207003 | 1162.36 | 1162.36 | -0.85 | N | 0 |
| 30 | Normandy | 207061 | 1158.90 | 1158.90 | -0.15 | N | 0 |
| 31 | Normandy | 208022 | 1157.13 | 1157.13 | 0.00 | Y | 22,500 |
| 32 | Normandy | 208044 | 1143.00 | 1143.00 | 0.00 | Y | 60,300 |
| 33 | Normandy | 208045 | 1143.44 | 1143.44 | -0.47 | N | 0 |
| 34 | Normandy | 208072 | 1141.10 | 1141.10 | -0.90 | N | 0 |
| 35 | Normandy | 208079 | 1141.06 | 1141.06 | 0.00 | Y | 9,200 |
| 36 | Normandy | 239042 | 1135.25 | 1135.25 | -0.85 | N | 0 |
| | | | | | | **Subtotal Normandy** | **118,900** |
| 22 | Park Hill | 121012 | 1210.71 | 1210.71 | -0.94 | N | 0 |
| | | | | | | **Subtotal Park Hill** | **0** |
| 11 | Tecumseh | 102008 | 1153.26 | 1153.26 | -0.47 | N | 0 |
| | | | | | | **Subtotal Tecumseh** | **0** |
| 48 | Westside | 328055 | 1124.60 | 1124.60 | 0.00 | Y | 21,200 |
| | | | | | | **Subtotal Westside** | **21,200** |
| 1 | York | 42006 | 1168.12 | 1168.12 | 0.00 | Y | 78,600 |
| 2 | York | 44008 | 1166.11 | 1166.11 | 0.00 | Y | 18,300 |
| 3 | York | 44009 | 1166.05 | 1166.05 | -0.94 | N | 0 |
| 4 | York | 44010 | 1166.03 | 1166.03 | -0.67 | N | 0 |
| 5 | York | 70031 | 1160.00 | 1160.00 | 0.00 | Y | 44,700 |
| | | | | | | **Subtotal York** | **141,600** |
| | | | | | | **Total North Sewer Shed** | **189,000** |
| | | | | | | **Total South Sewer Shed** | **258,600** |
| | | | | | | **Total All Basins** | **447,600** |

The locations of the proposed pipeline improvements to eliminate these 50 capacity excursions are shown collectively on **Figure 4-15.** Like the One-Plant Scenario, it is assumed all gravity pipeline improvements are replacement lines for this planning level evaluation.

Of the approximately 66,586 LF of identified gravity sewer improvements under the One-Plant Scenario, about 4,500 LF would not need improvement under the Two-Plant Scenario due to the decrease in flow loading. A summary of the difference in gravity piping lengths per sewer basin is given in **Table 4-15**.




**CITY OF NORMAN**
**FLOW MONITORING AND MODELING**
RECOMMENDED PIPING IMPROVEMENTS
TWO PLANT SCENARIO
FBO 2030, 5-YEAR 4-HOUR STORM
FLOW CONDITION

N
0 0.5 1 Miles

NORMAN UTILITIES AUTHORITY

HDR ENGINEERING, INC.
17111 PRESTON RD, SUITE 300
DALLAS, TX 75248-1232
972-960-4400

**FIGURE 4-15**

PATH: C:\USERS\TWHITFIE\DOCUMENTS_WORKING DOCUMENTS_PROJECTS_ACTIVE\NORMAN_NORMAN_GIS\MAP_DOCS\MXD\2017_FIGURE_RECREATION\FIGURE4-16.MXD - USER: TWHITFIE - DATE: 11/1/2017

### Table 4-15. Comparison of Gravity Piping Improvements Between One-Plant and Two-Plant Scenarios

| Sewer Basin | One-Plant Scenario Length of Pipe (ft) | Two-Plant Scenario Length of Pipe (ft) | Length Difference (ft) |
|---|---|---|---|
| Bishop | 24,223 | 19,700 | 4,524 |
| Brookhaven | 6,607 | 6,607 | 0 |
| Imhoff | 1,748 | 1,748 | 0 |
| LS D | 14,584 | 14,584 | 0 |
| Normandy | 11,352 | 11,352 | 0 |
| Tecumseh | 724 | 724 | 0 |
| Westside | 1,538 | 1,538 | 0 |
| York | 5,810 | 5,810 | 0 |
| **Total** | **66,586** | **62,062** | **4,524** |

As listed in **Table 4-15**, piping improvements needed for the One-Plant Scenario but not the Two-Plant Scenario are all located within the Bishop Sewer Basin. **Figure 4-16** highlights the location of the pipe segments that differ between the two scenarios. The two figures below illustrate improvements recommended for the One-Plant Scenario only (Segments 80, 609, 660 and 689).

### Figure 4-16. Gravity Piping Improvements in Bishop Creek Basin Not Needed for Two-Plant Scenario


Note: All gravity improvements identified previously for the One-Plant Scenario are also needed for the Two-Plant Scenario except for the improvements shown in this figure.

### 4.4.2 Lift Station and Force Main Evaluation

As with the majority of the piping improvements, redirecting the North Sewer Shed flow to a proposed North WRF has no affect on the lift station and force main evaluation conducted previously for the One-Plant Scenario. No force main or lift station improvements are needed for either scenario.

### 4.4.3 Estimated Construction Costs

A summary of the estimated capital costs for the recommended improvements to the existing gravity collection system are included below in **Table 4-16**.

**Table 4-16. Estimated Gravity System Improvement Costs for the Two-Plant Scenario[1]**

| Basin | Length of Pipe (ft) | Length of Pipe (mi) | Estimated Cost | Sewer Shed |
|---|---|---|---|---|
| LS D | 14,583 | 2.8 | $2,844,517 | North |
| Tecumseh | 724 | 0.2 | $132,079 | North |
| York | 5,810 | 1.1 | $1,060,426 | North |
| **Subtotal North** | **21,117** | **4.0** | **$4,037,022** | |
| Bishop | 19,700 | 3.7 | $4,639,965 | South |
| Brookhaven | 6,607 | 1.3 | $1,031,627 | South |
| Imhoff | 1,748 | 0.3 | $247,990 | South |
| Normandy | 11,352 | 2.2 | $2,106,138 | South |
| Westside | 1,537 | 0.3 | $608,788 | South |
| **Subtotal South** | **40,945** | **7.8** | **$8,634,508** | |
| **Total** | **62,062** | **11.8** | **$12,671,530** | |

[1]The estimated costs are the same as the One-Plant Scenario listed in Table 4-12, except for the Bishop Sewer Basin.

**Table 4-17** compares the estimated improvements costs for the One-Plant Scenario versus the Two-Plant Scenario.

**Table 4-17. Difference in Improvement Costs Between One-Plant and Two-Plant Scenarios**

| Improvement Type | One-Plant Scenario Estimated Cost | Two-Plant Scenario Estimated Cost | Difference |
|---|---|---|---|
| Gravity Pipe Improvements | $14,015,442 | $12,671,530 | $1,343,912 |
| **Total** | **$14,015,442** | **$12,671,530** | **$1,343,912** |

### 4.4.4 Norman WRF and North WRF Design Flows

Predicted wastewater flows occurring under a 5-year, 4-hour design storm event using the FBO 2030 loading condition were determined for the South Sewer Shed flowing to the existing Norman WRF and the North Sewer Shed flowing to a proposed North WRF. The summary of these flows is included in **Table 4-18**.

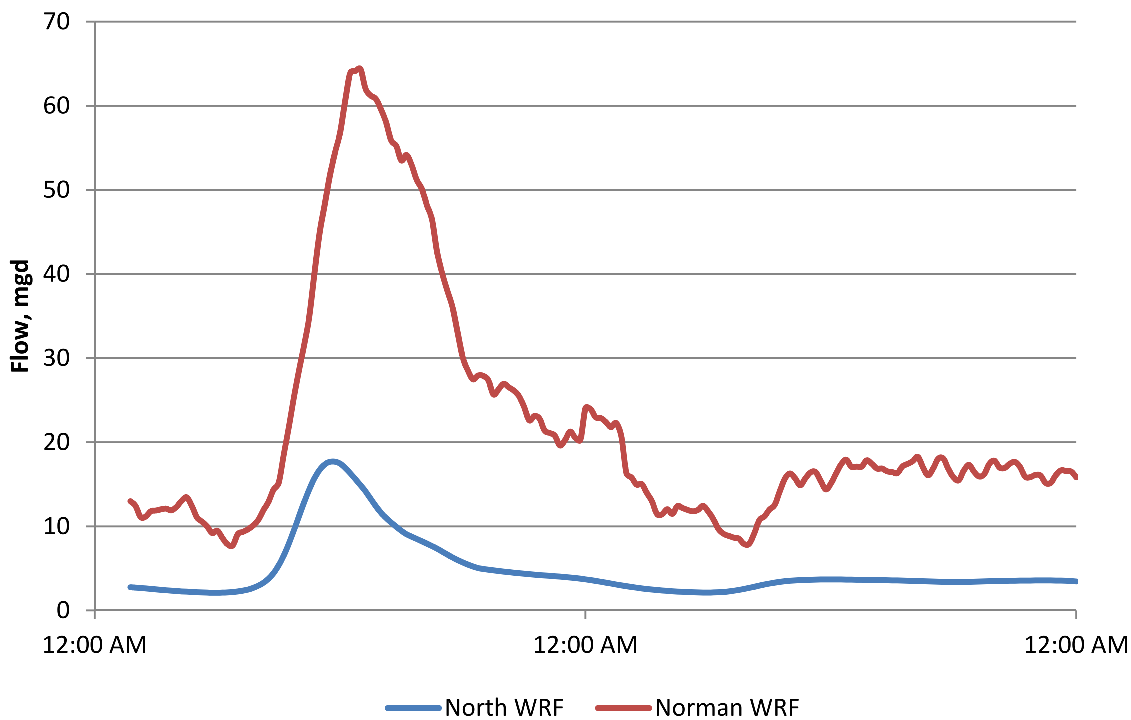
Table 4-18. Predicted WRF Wastewater for Two-Plant Scenario

| Parameter | Description | Predicted for Two-Plant Scenario, FBO 2030 | |
|---|---|---|---|
| | | Norman WRF | North WRF |
| Flow (mgd) | Average Dry Weather | 17.4 | 3.2 |
| | Annual Average[1] | 17.1 | 3.1 |
| | Annual Average Plus Planning Capacity[2] | 17.9 | 3.3 |
| | Maximum Day[3] | 28.2 | 6.4 |
| | Peak 2-hour Flow | 64.4 | 17.7 |

[1]Annual average = predicted average dry weather flow x 0.98
[2]Planning Capacity = 5%
[3]Maximum predicted wet weather flow over a 24-hour period

The predicted wet weather flow curve is shown on **Figure 4-17**.

Figure 4-17. Predicted Norman WRF and North WRF Flow Curves for Two-Plant Scenario for Peak 2-Hour FBO 2030



# 5 Evaluation Results Summary

## 5.1 Effectiveness of Recent Improvements

After incorporating updated flow monitoring data and calibrating the updated model, it was determined that the collection system improvements that Norman has implemented since the previous master plan was completed in 2001 have been very effective in reducing inflow and infiltration (I/I). The bulk of improvements occurred in the Bishop Creek basin, and I/I appears to have been reduced by up to 72%. The estimated percent reduction in I/I for all basins analyzed is shown in **Table 3-4**.

## 5.2 Recommended Future Collection System Improvements for Full Build-Out 2030 Flows

A total of 16.0 miles of sewer improvements (gravity and force main) have been identified under the One-Plant Scenario, whereas a total of 15.1 miles of sewer improvements have been identified under the Two-Plant Scenario. In essence, the collection system and lift station improvements are largely independent of whether or not a second WRF is implemented. This is mainly due to the significant improvements the City has already made in the Bishop Creek basin to carry northside flows to the existing WRF.

The improvements recommended to maintain the design criteria given the FBO 2030 loading under the 5-year, 4-hour wet weather event are summarized in **Table 5-1**.

**Table 5-1. Summary Comparison of One-Plant and Two-Plant Scenario Evaluation Results**

| Parameter | Value | One-Plant Scenario | Two-Plant Scenario |
|---|---|---|---|
| Annual Average Flow | mgd | 18.0 | 14.8 to Norman WRF<br>3.2 to North WRF |
| Maximum Month Flow | mgd | 21.6 | 17.8 to Norman WRF<br>3.8 to North WRF |
| Maximum Day Flow | mgd | 36.0 | 29.4 to Norman WRF<br>6.6 to North WRF |
| Peak 2-hour Flow | mgd | 81.9 | 64.1 to Norman WRF<br>17.8 North WRF |
| Number of predicted locations that have less than one foot of freeboard | - | 54 | 50 |
| Number of predicted SSO locations | - | 22 | 23 |
| Estimated quantity of overflows | MG | 0.42 | 0.45 |
| Recommended length of gravity sewer improvements | mi | 12.6 | 11.8 |
| Number of lift stations that do not meet evaluation criteria | - | 0 | 0 |
| Estimated construction cost for gravity sewer improvements | millions | $14.0 | $12.7 |

The recommended improvements for the One-Plant and Two-Plant Scenarios are shown on **Figures 4-3 and 4-15**, respectively.

To ultimately determine whether or not a North WRF should be constructed in the future, more information is needed than that included in this project. Costs for a new North WRF, as well as costs to expand the existing Norman WRF to meet the One-Plant or Two-Plant Scenario predicted influent flow must be considered. Costs and benefits for WRF effluent, both direct non-potable and indirect potable, also need to be considered. There are other economic and non-economic factors that will need to be considered as well to ultimately reach the decision that is best for the City.