8.1 PURPOSE

The main focus of updating the collection system model was to incorporate the flow monitoring and rainfall data that the City has collected since the last master plan update. This information will greatly improve the accuracy of the model, and will allow the City to evaluate the impacts that actual storms will have on the system.

Discussions with the City during the preliminary stages of the project resulted in the selection of the 10-year, 24-hour storm being used as the design storm for the collection system model. The analysis was conducted using XP-SWMM based on an updated version of the City’s existing InfoSWMM model. The overall objective for the analysis is to assist in planning and prioritizing future capital improvement plan (CIP) projects within the city’s service area. The hydraulic model was used to simulate flows within the collection system under existing, Tier I, Tier II and Tier III conditions. Under all planning scenarios, the 10-year, 24-hour storm event was routed through the model to estimate impacts to the collection system. The analysis identifies opportunities for optimizing the capacity in the existing system as well as identifying future CIP Projects. All future CIP projects will be divided into the three planning tiers as discussed previously in this report.

8.2 BACKGROUND OF THE EXISTING HYDRAULIC MODEL

Carollo Engineers previously updated the hydraulic model of the existing collection system as part of the 2007 Master Plan update. The model was developed using InfoSWMM software in the fully dynamic mode and only included pipes 15 inches and larger. Figure 8.1 shows the layout of the modeled pipes included in the model, and the proposed storage which was identified in the 2007 master plan update.

Flows input to the model in 2007 were developed using the City’s flow equation with a correction factor 0.75 applied which can be seen below. This equation utilizes the developable area in acres served by sanitary sewers as a basis for determining peak sanitary flows. The correction factor of 0.75 was applied so that the modeled peak flows matched the actual flows recorded at the various lift stations within the collection system.

\[
Q = ((0.01726 \times A^{0.8}) + (0.003 \times A)) \times 0.75
\]

Where,

\(Q\) = peak sanitary flow in cfs, \(A\) = developable land area in acres, 0.75 = flow reduction factor used in 2007 modeling effort

The flows calculated using this equation were conservative and as a result, the model predicted several surcharged conditions during the previous Tier II and III model runs. Thus
to reduce surcharge and flooding conditions, storage was modeled at several locations throughout the City. The proposed storage locations that were included in the 2007 model are shown on Figure 8.1 and summarized in Table 8.1. As seen in Table 8.1, about 42.0 million gallons of collection system storage was required to minimize surcharging. Based on the conservative nature of the previous modeling effort, the City elected to revise the model based upon the new flow monitoring and rainfall data to more accurately represent the existing and future flow conditions as discussed below.

Table 8.1 Summary of Proposed Storage from the 2007 Master Plan Wastewater Facilities Master Plan Update - 2014 City of Lincoln, Nebraska

<table>
<thead>
<tr>
<th>Basin</th>
<th>Tier I</th>
<th>Tier II</th>
<th>Tier III</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Salt Valley</td>
<td>2.0 MG</td>
<td>1.5 MG</td>
<td>6.0 MG (1)</td>
<td>9.5 MG</td>
</tr>
<tr>
<td>Upper SW</td>
<td>-</td>
<td>4.0 MG</td>
<td>3.0 MG</td>
<td>7.0 MG</td>
</tr>
<tr>
<td>Haines Branch</td>
<td>-</td>
<td>3.0 MG</td>
<td>3.0 MG</td>
<td>6.0 MG</td>
</tr>
<tr>
<td>West 'O' and Middle Creek</td>
<td>-</td>
<td>4.5 MG</td>
<td>2.0 MG</td>
<td>6.5 MG</td>
</tr>
<tr>
<td>Oak Creek</td>
<td>-</td>
<td>2.5 MG</td>
<td>7.5 MG</td>
<td>10.0 MG</td>
</tr>
<tr>
<td>Southeast</td>
<td>-</td>
<td>-</td>
<td>3.0 MG</td>
<td>3.0 MG</td>
</tr>
<tr>
<td>Totals</td>
<td>2.0 MG</td>
<td>15.5 MG</td>
<td>24.5 MG</td>
<td>42.0 MG</td>
</tr>
</tbody>
</table>

Notes: 1 – This 6.0 MG of storage is associated with additional storage in lieu of a new SW WWTF.
8.3 UPDATED HYDRAULIC MODELING METHODOLOGY

For this Master Plan update, the existing collection system model was updated with the new flow monitoring data, and land use planning information. The use of actual flow metering data and land use resulted in a more conventional approach to modeling collection systems and a more accurate model which better reflects the existing flow conditions. Additionally the flows in the updated model were split into dry weather and wet weather flows. Dry weather flows were estimated from the flow monitoring data and correlated with land use information. The wet weather flow parameters were determined from the flow monitoring data on days where precipitation was detected in the rain gauges. In general the model was updated as summarized below:

- The main sewer sheds were divided into smaller subsystems. Each inflow manhole was then associated with at least one sewer shed.
- Existing and projected dry weather flows for the sewer sheds were based on the flow monitoring data and the land use information resulting in unit flow factors for different land uses.
- Diurnal curves were developed from the flow monitoring data for each sewer shed.
- The new hydrographs were inserted into the model and replaced the flows that were estimated from the flow equation.
- Unique wet weather flow parameters were developed for each sewer shed based on the flow monitoring data. Other wet weather flow parameters such as impervious cover and slope were estimated from the City’s GIS database.
- The rain gauges provided data in hourly intervals for use in developing intensity, duration, and volume of storm events that occurred during the flow monitoring period. The rainfall data was also used to estimate the return periods of the storms.
- The 10-year, 24-hour storm was used for the modeling efforts as the design storm, which is typical in the Midwest for collection system evaluations.
- Based on the revised model, the need for storage was re-evaluated.

8.4 WASTEWATER AND RAINFALL MONITORING

This section presents the results of the City's flow monitoring program between 2005 and 2011. The flow monitoring was performed to accomplish the following:

- Gather and analyze rainfall and wastewater flows in order to calibrate and validate the collection system hydraulic model for existing dry and wet weather flows.
- Perform an inflow and infiltration analysis.
- Project future wastewater flows.
- Correlate projected flows, which are now based on land use and flow factors, with actual or "real world" collection system flows.
Flow projections for this modeling effort are based on land use information obtained from the City and unit flow factors developed from flow monitoring data. The current and future flow projections were used to perform the hydraulic assessment which is discussed later in this report. Figure 8.2 presents the flow monitoring and rain gauge locations.
FIGURE 8.2 – RAIN GAUGE AND FLOW METER LOCATIONS
WASTEWATER FACILITIES MASTER PLAN UPDATE – 2014
CITY OF LINCOLN, NEBRASKA
8.4.1 Rainfall Monitoring

Eight rain gauges were used for the hydraulic modeling effort. The rain gauges were utilized between 2005 and 2011 during the flow monitoring period. The locations of the rain gauges are shown on Figure 8.2 and summarized in Table 8.2. The purpose of the rainfall monitoring was to evaluate observed rainfall events for use in determining the relationship between rainfall and inflow for the collection system and to estimate the intensity, duration, and volume of storm events that occurred during the flow monitoring period. These values form part of the basis for analyzing the existing wastewater collection system capacity and projecting future system requirements. The rainfall data was also used to estimate the return periods of the recorded storm events.

Five significant storm events occurred during the monitoring period which are summarized in Table 8.3. Based on data from 1971 through 2010, the average yearly rainfall total was calculated to be 28.5 inches. The current rain gauge data indicated an average yearly rainfall total of 27.7 inches over the monitoring period, which is about one inch below the calculated historical average. It should be noted that the rain gauges provided data in hourly intervals and all rain gauges were used in this master plan update.

<table>
<thead>
<tr>
<th>Rain Gauge ID</th>
<th>Location</th>
<th>Northing</th>
<th>Easting</th>
</tr>
</thead>
<tbody>
<tr>
<td>254739</td>
<td>Highlands Golf Course</td>
<td>148697.7</td>
<td>2240505.8</td>
</tr>
<tr>
<td>254809</td>
<td>IANR (East Campus)</td>
<td>172036.7</td>
<td>211141.9</td>
</tr>
<tr>
<td>254759</td>
<td>901 N. 6th</td>
<td>158338.7</td>
<td>210664.6</td>
</tr>
<tr>
<td>254699</td>
<td>82nd &amp; Havelock (UNL Bldgs)</td>
<td>186387.5</td>
<td>220616.6</td>
</tr>
<tr>
<td>254749</td>
<td>Cotner &amp; A (Fire Sta.)</td>
<td>174136.2</td>
<td>200038.3</td>
</tr>
<tr>
<td>254729</td>
<td>82nd &amp; South (Fire Sta.)</td>
<td>186211.4</td>
<td>196936.0</td>
</tr>
<tr>
<td>254769</td>
<td>27th &amp; Old Cheney (Fire Sta.)</td>
<td>165974.7</td>
<td>184335.8</td>
</tr>
<tr>
<td>254719</td>
<td>Lincoln Country Club (20th &amp; Calvert)</td>
<td>163731.4</td>
<td>192783.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Date</th>
<th>Total Rainfall (inches)</th>
<th>Intensity (inches/hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/23/2006</td>
<td>1.131</td>
<td>0.062</td>
</tr>
<tr>
<td>5/5/2007</td>
<td>1.461</td>
<td>0.081</td>
</tr>
<tr>
<td>6/5/2008</td>
<td>0.991</td>
<td>0.058</td>
</tr>
<tr>
<td>6/11/2008</td>
<td>1.301</td>
<td>0.065</td>
</tr>
<tr>
<td>8/4/2009</td>
<td>1.211</td>
<td>0.064</td>
</tr>
</tbody>
</table>
8.4.2 Flow Monitoring

A total of 22 temporary flow meters were installed in the collection system to monitor and collect flow data. This flow monitoring data, coupled with the rain gauge information, was used to calibrate the collection system hydraulic model for both dry and wet weather flows, as well as perform an inflow and infiltration analysis.

The service area for the City was divided into twenty-two unique sewer basins based on the location of the flow meters. The purpose of dividing the City’s service area into unique basins was to ensure that the inflows and outflows from each basin were being isolated and recorded. This was accomplished by defining each basin using multiple flow meters.

8.4.3 Wastewater Flow Components

Typically, wastewater consists of three components: base sanitary flow (BSF), groundwater infiltration (GWI), and rainfall dependent infiltration and inflow (RDII). These components are shown graphically on Figure 8.3. BSF and GWI during dry weather constitute dry weather flow (DWF). BSF is generated from residential, commercial, industrial, and public sources that discharge into the wastewater collection system. BSF varies during the day in a diurnal pattern with the lowest flow generally occurring during the early morning hours when most people are asleep and businesses are closed, and the peak flow occurs in the mid-morning after people get ready for their days activities. GWI occurs when groundwater levels are above the inverts of the collection system pipes, and when the collection system has faulty joints or other defects that allow infiltration. Particularly the amount of GWI can be influenced by sewer pipes within close proximity to a body of water and can vary seasonally. RDII occurs during wet weather conditions and can enter the collection system in a variety of ways. A few examples of rainfall dependent infiltration are cracks in pipelines, misaligned joints, and root penetration. A few examples of rainfall dependent inflow are roof drain and downspout connections, leaky manhole covers, and illegal storm drain connections.
FIGURE 8.3 - WASTEWATER FLOW COMPONENTS

DWF: Dry Weather Flow
- GWI: Ground Water Infiltration
- BSF: Base Sanitary Flow

RDII: Rain Dependent Inflow and Infiltration
- Inflow
- Infiltration

Rain

Flow (mgd)
0.00 12:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00 00:00
0 2 4 6 8 10 12 14 16 18 20

Rain (inches)
0.00 0.50 1.00 1.50 2.00 2.50 3.00 3.50 4.00 4.50 5.00 5.50 6.00 6.50 7.00 7.50 8.00 8.50 9.00 9.50 10.00

FIGURE 8.3 - WASTEWATER FLOW COMPONENTS
WASTEWATER FACILITIES MASTER PLAN UPDATE – 2014
CITY OF LINCOLN, NEBRASKA
8.5 ANALYSIS OF FLOW MONITORING DATA

During the flow monitoring period, depth and velocity data were collected at each meter location in 15 minute intervals. The 15-minute data was then aggregated to hourly data for the dry and wet weather flow calibration and validation efforts. All analyses were performed using the U.S. EPA SSOAP Toolbox.

8.5.1 Dry Weather Flow Analysis

A dry weather flow analysis was performed for each flow meter basin based on the flow monitoring and rainfall data. To delineate dry weather flows, wastewater flow meter data was examined to identify periods of dry weather consisting of at least five days without a storm event. The selected dry weather flows were then decomposed into base sanitary flow and groundwater flows. The flow monitoring data for the days of January 15 through January 19, 2010 provided the most characteristic dry weather flow period for all flow meters during the flow monitoring period. The hourly data for these days were averaged to provide a typical 24-hour dry weather flow pattern at each meter. This hourly flow data was then used to validate the hydraulic model for average dry weather flow.

8.5.2 Groundwater Infiltration (GWI)

The groundwater table fluctuates over the wet weather season. This fluctuation is seen as a mounding effect in the flow monitoring data. Thus at different times during the wet weather season, groundwater infiltration will play a more significant role. It is important in the modeling process to calibrate to the highest groundwater mounding effect seen in the flow monitoring data. This ensures that the model is being calibrated to the worst case scenario and that the potential impact of groundwater infiltration is not underestimated. However, GWI is not directly related to a specific rainfall event. Rather it is a function of the collection system’s physical condition, and the proximity of the collection system pipelines to groundwater. For instance, gravity sewers constructed prior to the 1960’s were typically vitrified clay pipes. This type of sewer is highly susceptible to GWI because the joint material deteriorates rapidly which allows groundwater to enter the sewer system, and the amount of GWI entering the system increases the closer the collection system is to bodies of water.

Understanding GWI contributions lends insight into the seasonal variation observed in DWF. This seasonal variation in DWF is used to develop monthly values for input into hydraulic models for continuous simulation. It is also useful to estimate GWI for prioritizing areas with larger GWI values for more detailed sewer system evaluations and rehabilitation. The flow monitors provided data that was used to estimate GWI for the monitored basins. GWI was estimated from nighttime flow, which occurs between midnight and 2:00 am, which is termed the minimum nighttime flow approach. The nighttime flow represents a period of minimal sanitary flow; therefore, a high percentage of the nighttime flow can be attributed to groundwater infiltration. However, a portion of the nighttime minimum flow may
also be attributed to sanitary flow from 24-hour industrial/commercial operations, institutional flows, and/or some small amount of domestic flow. To differentiate the actual GWI and actual sanitary flow, a combination of the following two analyses can be used:

- A detailed survey of large nighttime industrial, commercial, or institutional water users, and/or
- An estimate, based on similar communities in the region, of the expected percentage of the minimum nighttime flow which may be attributable to domestic flow contributions.

The percentage of the minimum nighttime flow which may be attributable to domestic flow contributions is likely to be a small portion. In this analysis, 90 percent of the minimum nighttime flow was attributed to groundwater infiltration based on the characteristics from other communities in the Midwest. Table 8.4 shows the seasonal variation of GWI for the monitored basins as calculated using the method described above.

<table>
<thead>
<tr>
<th>Basin</th>
<th>Flow Meter Manhole Location</th>
<th>Spring(1)</th>
<th>Summer(1)</th>
<th>Fall(1)</th>
<th>Winter(1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Middle Creek</td>
<td>A4-19</td>
<td>0.059</td>
<td>0.054</td>
<td>0.052</td>
<td>0.042</td>
</tr>
<tr>
<td>West 'O'</td>
<td>A5-129</td>
<td>0.270</td>
<td>0.000(2)</td>
<td>0.270</td>
<td>0.270</td>
</tr>
<tr>
<td>Salt Valley</td>
<td>B1-166</td>
<td>0.180</td>
<td>0.170</td>
<td>0.170</td>
<td>0.140</td>
</tr>
<tr>
<td>Beals</td>
<td>B2-17</td>
<td>0.220</td>
<td>0.200</td>
<td>0.200</td>
<td>0.170</td>
</tr>
<tr>
<td>Antelope</td>
<td>B5-496</td>
<td>0.510</td>
<td>0.490</td>
<td>0.440</td>
<td>0.400</td>
</tr>
<tr>
<td>Oak Creek</td>
<td>A6-198</td>
<td>0.320</td>
<td>0.290</td>
<td>0.240</td>
<td>0.210</td>
</tr>
<tr>
<td>Little Salt</td>
<td>B6-269</td>
<td>0.080</td>
<td>0.080</td>
<td>0.050</td>
<td>0.040</td>
</tr>
<tr>
<td>Deadmans</td>
<td>B6-115</td>
<td>0.190</td>
<td>0.170</td>
<td>0.160</td>
<td>0.130</td>
</tr>
<tr>
<td><strong>Totals:</strong></td>
<td>-</td>
<td>1.829</td>
<td>1.454</td>
<td>1.582</td>
<td>1.402</td>
</tr>
</tbody>
</table>

Notes:
(1) Spring: March-May, Summer: June-August, Fall: September-November, Winter: December-February.
(2) Average for entire basin.

### 8.5.3 Inflow and Infiltration Analysis

The flow monitoring data was also evaluated to determine the optimal wet weather period to calibrate the hydraulic model. Five peak wet weather flow (PWWF) periods occurred during the monitoring. During these periods, the creeks running through the City’s service area can cause the groundwater table to rise, potentially causing large amounts of groundwater infiltration to enter into the collection system. A summary of the peak wet weather flow data recorded during the monitoring period is presented in Table 8.5. The peaking factor for
each individual flow meter is also presented in Table 8.5. Peaking factors are determined by dividing the peak wet weather flow by the average dry weather flow. Although the peaking factors give a relative indication of the quantity of inflow and infiltration entering the collection system upstream of a particular meter they do not show which pipes upstream of a meter are contributing the greatest volume of RDII. Analyzing the peaking factors and factors calculated from the volume of RDII from each basin will only yield an overall picture of the City’s collection system performance during rainfall events.

<table>
<thead>
<tr>
<th>Table 8.5 Wet Weather Flow Monitoring Summary by Flow Meter</th>
<th>Wastewater Facilities Master Plan Update - 2014</th>
<th>City of Lincoln, Nebraska</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meter I.D.</td>
<td>Basin</td>
<td>ADWF(2) (mgd)</td>
</tr>
<tr>
<td>B2-17</td>
<td>Beals</td>
<td>1.83</td>
</tr>
<tr>
<td>B5-496</td>
<td>Antelope</td>
<td>7.68</td>
</tr>
<tr>
<td>A6-198</td>
<td>Oak Creek</td>
<td>1.47</td>
</tr>
<tr>
<td>B6-269</td>
<td>Little Salt</td>
<td>0.55</td>
</tr>
<tr>
<td>B1-166</td>
<td>Salt Valley</td>
<td>0.73</td>
</tr>
<tr>
<td>B6-115</td>
<td>Dead Mans</td>
<td>1.15</td>
</tr>
<tr>
<td>A5-129</td>
<td>West O</td>
<td>0.53</td>
</tr>
<tr>
<td>A4-19</td>
<td>Middle Creek</td>
<td>0.86</td>
</tr>
</tbody>
</table>

Notes:
(1) PWWF = peak wet weather flow
(2) ADWF = Average Dry Weather Flow based on average

8.5.4 Estimation of Existing Base Sanitary Flow (BSF)

Base sanitary flow is sanitary flows generated from residential, commercial, industrial, public or institutional sources that discharge into the wastewater collection system. It may vary in magnitude throughout the day, but generally follows a predictable and repeatable diurnal pattern with peak flow usually occurring during the morning hours. Different methods are available for estimating base sanitary flow. Two methods that can be used to develop projected flows are the application of per capita flow factors or flow-per-acre factors. The per capita flow approach is generally based on population projections and a sanitary flow generation rate per person, whereas the flow-per-acre approach is based on the total areas, and determined flow factors for each unique land use type. Validation of either
estimation method is determined by comparing the results for estimated flows with the flows measured during the flow monitoring period. Based on data availability, the flow-per-acre approach was employed for this analysis.

Generally the flow per-per-acre approach involves calculating area for each existing major land use type within each subsystem (single family residential, multi-family residential, commercial, and industrial). The areas estimated for each land use category are then converted to a flow volume by applying a land use flow factor unique to each respective land use type. In this analysis, published flow rates were initially used and later refined through iterative techniques using the dry weather flow data observed during the flow monitoring period. The existing land use utilized in this analysis is shown in Figure 8.4. The existing land use was designations were identified by using the parcel and zoning maps as provided by the City. The flow-per-acre approach is outlined in more detail below.

**Step 1: Estimate Average Base Flow from Flow Monitoring Data.** The first step in calculating flow using this approach is to examine historic sanitary flow data compiled from the flow monitoring programs. Based on analysis of the collected flow monitoring data, average base sanitary flows were estimated for each monitored subsystem.

**Step 2: Estimate Acreage for each Land Use Type.** From the existing land use data, area (in acres) for each land use category was extracted for each subsystem. Within the current wastewater service area, developed land totals over 54,000 acres. Single family residential accounts for about 60 percent of the developed area in the service area.

**Step 3: Calculate Land Use Coefficient.** Land use coefficients were used to convert developed land use area to base flow equivalents. Initially, published land use coefficients were assigned to each land use category. The following equation was then used to estimate actual land use coefficients for each subsystem.

\[ \sum_{i} f_i = \sum_{i} (\mu_i \times A_i) = F_s \]

Where:
- \( F \) = observed flow in gpd in a subsystem
- \( \mu \) = land use coefficient in gpd/acre
- \( A \) = developed area in acres
- \( i \) = land use category
- \( n \) = total number of land use categories
- \( s \) = subsystem
- \( f \) = flow generated from land use category

For each land use category, the land use coefficient (\( \mu \)) was determined iteratively. Table 8.6 summarizes the calibrated land use coefficients determined for the subsystems.
Step 4: Estimate Subsystem-Wide Base Sanitary Flow. Using the land use coefficients calculated in Step 3, and as shown in Table 8.6, with developed acreages for each land use category, current flows were estimated for each subsystem.
### Table 8.6 Unit and Area Wastewater Flows

**Wastewater Facilities Master Plan Update - 2014**  
**City of Lincoln, NE**

<table>
<thead>
<tr>
<th>Land Use Type</th>
<th>Flow (gpd/ac)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential</td>
<td>950</td>
</tr>
<tr>
<td>Business/Commercial/Office</td>
<td>2,000</td>
</tr>
<tr>
<td>Industrial</td>
<td>400</td>
</tr>
<tr>
<td>Park/Agricultural</td>
<td>22</td>
</tr>
</tbody>
</table>

#### 8.5.5 Diurnal Curve

After the residential and commercial/industrial flows were determined, diurnal curves were created for all pipes tributary to a specific flow meter. The diurnal curves depict the time variation of the base sanitary flow throughout a 24-hour period. Usually, peaks in a diurnal curve will occur in the morning between 8 a.m. and 10 a.m., and again in the evening between 6 p.m. and 8 p.m. Using the flow data measured during the monitoring period, an average diurnal curve was developed for each flowmeter basin. Figures 8.5 and 8.6 show the average diurnal curve developed for flow monitoring sites B1-166 and B1-290, respectively.

To develop the dry weather diurnal curve, a period of five days was selected which had dry weather (no precipitation) and was preceded by a dry weather period of at least a few days. As stated previously, the dates selected for this analysis fell between January 15 and January 19, 2010. The dry weather flow pattern was based on metered flows occurring every 15 data, aggregated to hourly data over a 24-hour period.
FIGURE 8.5 – DIURNAL CURVES FOR FLOW MONITORING
LOCATION B1-166
WASTEWATER FACILITIES MASTER PLAN UPDATE – 2014
CITY OF LINCOLN, NEBRASKA
8.5.6 Tier I, Tier II and Tier III Wastewater Flow Projections

Once the land use flow factors were calibrated to match the flow monitoring data, they were used to project the future flows based on the Tier I, Tier II and Tier III growth conditions of the City. To calculate the future flows, the calibrated flow factors were applied to the area for future land use in the service area as derived from the City’s GIS database for each subsystem. Figures 8.7 through 8.10 show the dry weather flows derived for the Existing Tier I, Tier II, and Tier III growth conditions, respectively.

8.5.1 Extreme (200-yr) Storm Event Analysis

On the night of September 30, through October 1, 2014 the City of Lincoln, NE experienced an approximate 200-yr storm event. The total rainfall depth across the City varied from 4.5 to 6.5” during the storm event. Thus, the City tasked Carollo with running this same storm event through the updated hydraulic model to see how the model compares to the metered data collected during this event. A summary of this analysis and the results are included in Appendix C.
FIGURE 8.7 – EXISTING DRY WEATHER FLOWS

CITY OF LINCOLN, NEBRASKA

Legend

Existing Flows, mgd

- 0.00 - 1.00
- 1.01 - 2.00
- 2.01 - 3.00
- 3.01 - 4.00
- 4.01 - 5.00
- 5.01 - 6.00
- 6.01 - 7.00
- 7.01 - 8.00
- 8.01 - 9.00

# Flow, mgd

Therresa St. WWTP
Northeast WWTP

FIGURE 8.7 – EXISTING DRY WEATHER FLOWS
WASTEWATER FACILITIES MASTER PLAN UPDATE – 2014
CITY OF LINCOLN, NEBRASKA
FIGURE 8.8 – PROJECT TIER I DRY WEATHER FLOWS
WASTEWATER FACILITIES MASTER PLAN UPDATE – 2014
CITY OF LINCOLN, NEBRASKA

Legend

<table>
<thead>
<tr>
<th>Tier I Flows, mgd</th>
<th>0.00 - 1.00</th>
<th>1.01 - 2.00</th>
<th>2.01 - 3.00</th>
<th>3.01 - 4.00</th>
<th>4.01 - 5.00</th>
<th>5.01 - 6.00</th>
<th>6.01 - 7.00</th>
<th>7.01 - 8.00</th>
<th>8.01 - 9.00</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Green</td>
<td>Green</td>
<td>Green</td>
<td>Yellow</td>
<td>Yellow</td>
<td>Orange</td>
<td>Red</td>
<td>Purple</td>
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<td>Theresa St. WWTP</td>
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<td>Northeast WWTP</td>
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Flow, mgd
FIGURE 8.9 – PROJECT TIER II DRY WEATHER FLOWS
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Legend

Tier II Flows, mgd

- Green: 0.00 - 1.00
- Yellow: 1.01 - 2.00
- Green: 2.01 - 3.00
- Light Green: 3.01 - 4.00
- Dark Green: 4.01 - 5.00
- Orange: 5.01 - 6.00
- Light Orange: 6.01 - 7.00
- Yellow: 7.01 - 8.00
- Red: 8.01 - 9.00

Theresa St. WWTP
Northeast WWTP
Flow, mgd

FIGURE 8.9 – PROJECT TIER II DRY WEATHER FLOWS
WASTEWATER FACILITIES MASTER PLAN UPDATE – 2014
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FIGURE 8.10 – PROJECT TIER III DRY WEATHER FLOWS

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Legend

Tier III Flows, mgd

- 0.00 - 1.00
- 1.01 - 2.00
- 2.01 - 3.00
- 3.01 - 4.00
- 4.01 - 5.00
- 5.01 - 6.00
- 6.01 - 7.00
- 7.01 - 8.00
- 8.01 - 9.00

- # Flow, mgd

- Theresa St. WWTP
- Northeast WWTP

FIGURE 8.10 – PROJECT TIER III DRY WEATHER FLOWS
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8.6 MODEL UPDATE

The fundamental definition of a model is “a representation of a physical entity.” A collection system model is thus a simplified representation of the real collection system. The amount of simplification will define the applicability of the model in a given situation. In general, collection system models are used to assess the current level of performance for the collection system based on population and land use.

However, collection system models can also perform “what if” scenarios to project the performance of the collection system based on future developments, population and land use changes, and associated changes in collection system flow. Having a robust modeling capability to run these “what if” scenarios is a priority to the City. In order to achieve this objective, a model should constantly be updated to include new development, and new methodology.

The updates made to the existing model as part of the this updated are summarized below:

- Included newly constructed pipes.
- Sub divided each drainage basin into smaller sewer sheds meaning wastewater flows generated from the sewer sheds are routed through the collection system.
- Assigned diurnal curve to each sewer shed.
- Developed and coupled hydrologic model for the simulation of RDII.
- Decomposed and represented dry weather flow as GWI and BSF.
- Calibrated model to the new flow monitoring data.

8.6.1 Dry Weather Flow Calibration/Validation

Simulation runs under dry weather flow conditions were performed to verify the base flow generated. The validation was performed at each flow monitoring location using data from the updated monitoring program. The primary goal of the validation was to match the volume of flow generated in the model with the volume measured during the monitoring period. The secondary goal was to match the average dry weather flow pattern between the data sets.

GWI and BSF rates were added to each loading manhole (flow insertion point) and run through the model. The results of the dry weather flow calibration are shown graphically for B1-166 and B1-290 flow monitoring sites on Figures 8.11 and 8.12, respectively. The results show overall good agreement between the simulated results and the flow monitoring data.
FIGURE 8.11 – COMPARISON SIMULATED DRY WEATHER FLOW AND OBSERVED FLOW AT B1-166
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FIGURE 8.12 – COMPARISON SIMULATED DRY WEATHER FLOW AND OBSERVED FLOW AT B1-290
WASTEWATER FACILITIES MASTER PLAN UPDATE – 2014
CITY OF LINCOLN, NEBRASKA
8.6.2 Wet Weather Flow Model Simulation

Overall, the wet weather calibration involved simulating the observed rainfall data in the model and comparing the model results with the observed flow monitoring data. The wet weather flow calibration begins with the development of a runoff model to estimate RDII. The SWMM software was used to develop this runoff model to simulate the response of the sanitary collection system to the sanitary, groundwater, and rainfall derived inflows. Once constructed and calibrated, the runoff model was used to project flows under wet weather conditions for existing conditions.

The SWMM model is primarily intended to simulate collection systems that involve stormwater, i.e. drainage systems and combined sewer systems. However, SWMM can also model separate sanitary systems with a high degree of reliability. If the interest is in the quantity of RDII, special procedures have to be used. One procedure is to estimate the RDII separately and input it to the model. Another procedure that has been used successfully, and was used in this study, is to use the SWMM runoff module to simulate the RDII flow component. In either case, RDII estimation has to be based on flow measurements during wet weather. The RDII hydrograph development is explained below.

Analysis of RDII requires a method to relate sewer flows to rainfall. Methods in use are documented in the Water Environment Research Foundation project report Sanitary Sewer Overflow Flow Prediction Technologies, Project 97-CTS-8, April 1999. The Rainfall-Flow Regression Method and true hydrologic method (using SWMM runoff module) are two commonly methods often considered. The report notes that for prediction of peak flows under actual conditions (prolonged wet periods or multiple events), true hydrologic methods are preferred.

The Rainfall-Flow Regression method estimates RDII based upon a relationship developed using multiple linear regressions to associate rainfall summed over various antecedent periods to observed RDII flow. Due to the available data quality and quantity, the Rainfall-Flow Regression Method was not considered in this analysis. Therefore the true hydrologic method was used. Once calibrated, the model can be used with any arbitrary rainfall condition, a long-term local rainfall record, or design storms to simulate the RDII and total flows that would be expected at every hour of that rainfall record. With this method, there is increased confidence that the response of the system is accurately estimated. This confidence, however, is measured on the ability of the model to predict peak flows beyond the range of rainfall conditions experienced in the monitoring periods. Confidence is increased with longer monitoring and a greater variation in rainfall events during the monitoring period.

Simulating RDII using SWMM runoff requires the specification of subbasin characteristics that result in correct RDII. These subbasin characteristics do not have any physical significance, but they allow simulation of RDII using runoff calculation formulations. The parameters specified are described below:
**Subbasin Area, (A).** The surface area of the sewer shed area tributary to the inflow point in the model. Subbasin areas were generated in GIS and imported into the model.

**Percent Impervious, C (dimensionless).** For most cases, the volume of RDII is proportional to the rainfall depth. If $V$ is the volume of RDII, then

$$V = CA(D - D_s)$$

where

- $V =$RDII volume, ft$^3$
- $C =$ percent impervious (equivalent to runoff coefficient)
- $D =$rainfall depth, ft
- $D_s =$Depression storage, ft
- $A =$ subbasin area, ft$^2$

The value of $C$ was determined by analysis of flow measurement data. After separating the rainfall-induced flow for a number of storms, RDII volumes were calculated and plotted versus rainfall depth. The slope of the correlation line gives an estimate of $C$.

The parameter $C$ is used to represent the ratio of RDII volume in feet to rainfall depth in feet. The $C$-value is dependent upon the condition of the sewer and the density of development. A sanitary sewer systems in good condition will have $C$-values of less than 0.01. Areas with lower densities of development will have lower densities of sewer, and thus, will have lower $C$-values. Based on the monitoring, all the flow monitoring sites have $C$-values greater than 0.01.

**Subbasin Slope, S (dimensionless).** This parameter acts together with the subbasin width, discussed below. The actual subbasin slope was determined from the City’s contour data.

**Subbasin Width, W (ft).** The width is determined to match the peak RDII flow during several storms. This can be done by simulating the storm events using the runoff module and adjusting the subbasin width until the correct peak flow is obtained. The initial value was estimated from subbasin area and the longest flow path.
FIGURE 8.13 – FLOW METER BASIN R-FACTORS
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8.6.3 Wet Weather Flow Validation

The ultimate goal of the wet weather flow validation was for the modeled data to match the storm peaks from the 100-year storm event. The 24-hour 100-year storm depth for Lincoln is approximately 6.73 inches. Figure 8.14 compares simulated flows based on the City's design equation (with the 0.75 correction factor) and the 100-year 24-hour flows. The figure shows flow in the 60-inch pipe between manholes B1-3 and B1-2. The results show that two methods are in agreement regarding peak flow. However, the flow volumes generated by these methods are significantly different.

Based on the fact that the peak flows from the two methods are approximately the same, it can be concluded that City's design equation (with a correction factor of 0.75) can be used to size new infrastructure based on peak flows for planning purposes. However, for more detailed design, the dynamic model utilizing the updated flows should be used.
FIGURE 8.14 – FLOW COMPARISONS

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Updated Model  Old Model
8.7 WET WEATHER FLOW – DESIGN STORM SIMULATION

After achieving reasonable agreement between the flows simulated from the hydrologic model and the City design flow equation, the updated model was used to simulate the 10-year, 24-hour design storm. A total rainfall depth of 5.5 inches was distributed to the hours in the day using the SCS Type II rainfall distribution. The simulation results for selected basins are summarized below for existing, Tier I, Tier II and Tier III conditions. The basins included below are the only basins in which storage was proposed for future flow conditions based on the revised model, and the Stevens Creek and Havelock basin due to the new diversion pipe. For all other basins, the results based on the revised model indicated there were no SSO’s, and the minimum required free board of 3 feet was maintained during the design storm.

8.7.1 Salt Valley Basin

Model simulations of all planning horizons predict that a minimum of 3 feet of freeboard will be maintained, and that no SSOs will occur in the Salt Valley trunk sewer system under dry weather conditions. However, the results show that overflows are expected to occur under the 10-year, 24-hour storm event for Tier III conditions only. Under Tier III conditions, a two to three hour surcharge condition occurred in the Salt Valley Trunk Sewer between manhole B1-5 on the lower end (near 1st St and Old Cheney Rd) to manhole B3-675 at the upper end (near 2nd St and ‘A’ St) of Salt Valley Relief Trunk Sewer Phase V project near Old Cheney and Salt Creek. In this sewer, 13,198 feet of pipe exceeded a d/D ratio of 1.2 indicating the line is undersized for the Tier III flow condition. However, the freeboard or the elevation difference between the simulated water surface and the crest of the manholes in the surcharged pipes is larger than 3 feet. To prevent this surcharging, a 1.0 MG storage facility will likely still be needed under Tier III conditions in the Upper SE Salt basin (upstream of Salt Valley basin) to prevent the occurrence of overflows. This is based on several of the manholes in the basin having less than 3 feet of freeboard during the design storm event. Although there is storage proposed for upstream of the Salt Valley basin, it is recommended that these manholes be continuously monitored when growth reaches the Tier III planning horizon to confirm if storage is needed.

8.7.2 West O Basin

Model simulations for dry weather flows across all planning horizons predict that the minimum requirement of 3 feet of freeboard will be maintained, and that no SSOs will occur in the West O basin trunk sewer system. However, during the design wet weather storm for Tier III conditions, the d/D increases to 1.3 in a few pipes reducing the freeboard to less than 3 feet in these manholes. Thus, a new storage basin of 1.5 MG is proposed for Tier III conditions to increase the minimum freeboard to greater than 3 feet for the design storm. Again, this is much less than the total storage of 5.0 MG proposed in 2007 but the manholes in question should be monitored to confirm if storage is still required as the Tier III growth horizon is reached.
8.7.3 Oak Creek Basin

Model simulations of all planning horizons predict that a minimum of 3 feet of freeboard will be maintained, and that no SSOs will occur in the Oak Creek trunk sewer system under dry weather conditions. However, during the design storm for Tier III conditions, short duration SSOs are predicted to occur. Thus, a new storage basin of 2.0 MG is proposed for Tier III conditions to minimize these SSOs and increase the minimum freeboard to greater than 3 feet for the design storm. Again, this is much less than the total storage of 10.0 MG proposed in 2007 but the manholes in question should be monitored to confirm if storage is still required as the Tier III growth horizon is reached.

8.7.4 Stevens Creek Basin

Included in the 2007 work for the Steven Creek sewer system was a 10-inch force main and a lift station. The lift station is needed at this location to serve the Tier I area located just south of Cornhusker Highway between Interstate 80 and Stevens Creek. This area is very flat and to provide sewer service a lift station is required to raise the wastewater high enough to flow by gravity into the existing Stevens Creek Basin Trunk Sewer at Junction Structure E-6 (manhole E8-1). Due to the need for a lift station, this site was a good candidate for a future storage facility to manage peak flows in the Stevens Creek Basin Trunk Sewer and the NE WWTF which was reevaluated as part of the modeling effort. Although previous analysis using the City’s flow equation identified the site of the lift station as a potential storage facility, the current modeling effort indicated that the Stevens Creek sewer system has adequate capacity to convey the design storm flow under all planning tiers. Thus, no storage facility is needed at the lift station. This area known as Study Area No. 3 is currently being analyzed by the City under a separate project to identify and further define sanitary sewer options to serve the area. This analysis has been included in Appendix D as part of this Master Plan Update.

8.7.5 Havelock Basin

The Havelock and surrounding drainage basins are shown schematically in Figure 8.15 where it can be seen that the Deadmans Run sewer system and Havelock Basin sewer system currently drain to the Northeast WWTF, while the East Campus Basin sewer system drains to the Theresa Street WWTF. It should be noted that the flows in the Deadmans Run trunk sewer upstream from manhole C6-194 (south of N. 40th St and Adams St) can be diverted to the Theresa Street WWTF. Model simulations of the existing system indicated that no SSO’s occurred during the peak wet weather flows under the design storm event but some pipeline segments had minor surcharging with d/D values ranging from 0.49 to 1.1.

To eliminate the intermittent peak flow surcharging conditions, increased conveyance is required in the sewers between manholes D9-73 and D9-77. This pipe segment is relatively flat and there is a potential for overflow for storms greater than design storm event. To achieve an increase in capacity, flow will be diverted upstream of manhole E7-9 through a
new 2,000 feet, 15-inch sewer to Stevens Creek interceptor at manhole SC_JSE-1 which is located near 84th Street and Fletcher Avenue. This proposed new diversion sewer between manholes E7-9 and SC_JSE-1 is also illustrated in Figure 8.15. The improvement would reduce the surcharging in the pipe segment between D9-73 and D9-77 which is illustrated in Figure 8.16 where the hydrographs of the 2007 analysis, current analysis without flow diversion at E7-9, and current analysis with flow diversion at E7-9 just upstream of D9-68 are shown graphically. This Regent Heights diversion sewer has several benefits including removing flow from the Deadmans Trunk Sewer which in turn provides additional capacity for development in the Northeast Salt Creek Basin, and directs flow into the Stevens Creek Basin Trunk Sewer which currently is not being utilized. This flow will aid in flushing solids out of the Trunk Sewer that may accumulate due to the back water effects of the system.

8.7.6 Beals Slough Basin

The majority of the Beals Slough basin is located within the City limits and currently served by the existing collection system. However, due to growth that is expected to occur in the south east portion of the basin, parallel and upsized sewers were evaluated as part of a separate TM entitled *Timing of Beals Slough Improvements*. This document has been included in Appendix E as part of this Master Plan Update.
FIGURE 8.16 – COMPARISON OF HYDROGRAPHS
UPSTREAM OF MANHOLE D9-68
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