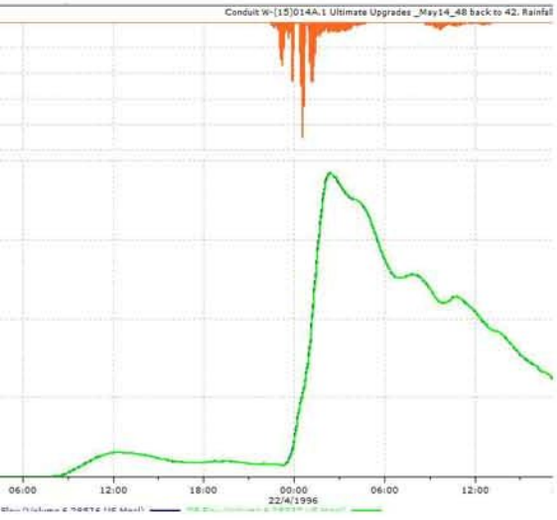
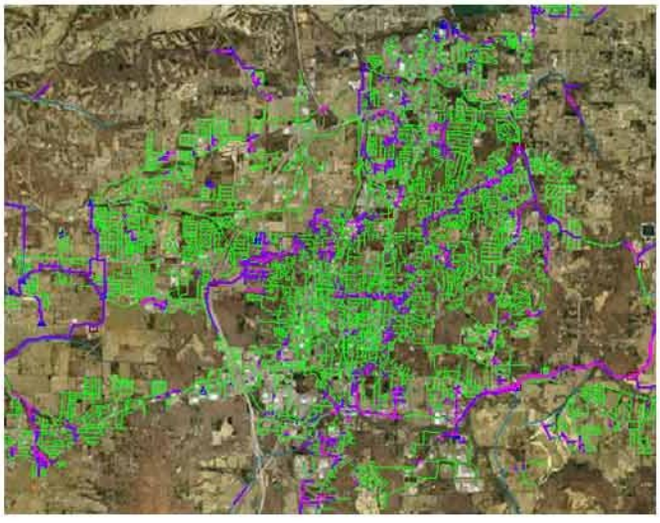
October 2014

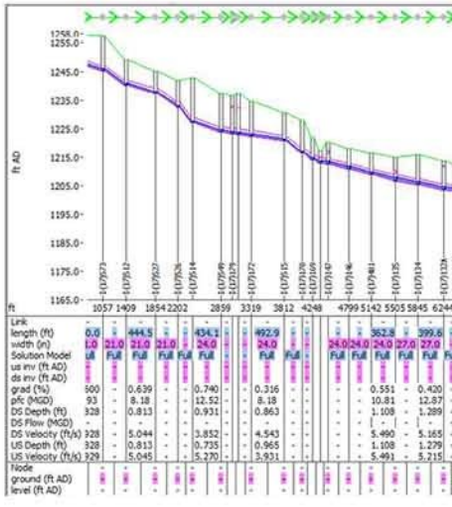
City of Fayetteville, Arkansas Wastewater Collection System 2014 Master Plan Update

Final Report

City of Fayetteville







prepared by

rjngroup

The Choice for Collection System Solutions

4170 West MLK Blvd., Suite 907 • Fayetteville, Arkansas 72704 • 479.300.6308 • www.rjn.com

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October 7, 2014

Mr. Don Marr
City of Fayetteville
113 W Mountain Street
Fayetteville, AR 72701

Subject: City of Fayetteville, Arkansas
Wastewater Master Plan Update

Dear Mr. Marr:

In accordance with the May 15, 2012 Engineering Agreement, RJN Group, Inc. is pleased to submit this final report for the above referenced project.

This report provides information on the development of the hydraulic model update including flow data and the application of this information to the analysis, a brief discussion of population projections, model build and calibration, modeling results, and a projected capital improvement program.

We appreciate the opportunity to work with the City of Fayetteville and the excellent cooperation from the City's staff throughout the project. We look forward to working with the City of Fayetteville in the future. Should you have any questions, please call.

Respectfully Submitted,

RJN GROUP, INC.

Hugh M Kelso

Hugh Kelso
Vice President

Shannon W. Jones

Shannon Jones, P.E.
Project Manager

HMK/SWJ/2595
Enclosure

2014 WASTEWATER MASTERPLAN UPDATE

October 2014

City of Fayetteville

Fayetteville, Arkansas

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 11053
SHANNON W. JONES
10-07-14

I hereby certify that this report was prepared under my direct supervision and that I am a duly registered Professional Engineer under the laws of the State of Arkansas.

Date: 10/07/2014 Registration No.: 11053

CERTIFICATE OF AUTHORIZATION
RJN GROUP, INC.
No. 1523
ARKANSAS

EXECUTIVE SUMMARY

This report presents the results of the Wastewater Collection System Model and Master Plan Update that was authorized by the City of Fayetteville under an agreement dated May 15, 2012.

BACKGROUND AND OBJECTIVES

The wastewater system contains over 540 mile of main sewer and two WWTPs and serves over 80,000 customers.

In 1996 the City of Fayetteville entered into a contract with CH2MHILL and RJN Group, Inc. to complete a Collection System Master Plan and Facility Plan for the wastewater system in the City. The Master Plan resulted in a recommended plan to eliminate wet weather overflows and bring the system into compliance with USEPA and ADEQ regulations. The Plan included construction of a new WWTP in the Illinois River Watershed, upgrading the Noland WWTP in the White River watershed as well as construction of new pipelines and lift stations to convey wastewater flow to the two WWTPs. The Plan also recommended the abandonment of several lift stations within the City. The recommendations in the Plan were implemented through the Wastewater System Improvement Project (WSIP) which was completed by 2009.

The changes made to the system as part of the WSIP resulted in the hydraulic model developed in 1996 to now be outdated and does not provide the City the ability to evaluate the impact of new developments on the capacity of the collection system and treatment system. This Master Plan Update Project updated the City's hydraulic model to reflect the current conditions and also evaluated the impact of future growth within the City's Service Area. The hydraulic model will be provided to the City as part of the deliverables of the project.

PROJECT ADMINISTRATION AND MANAGEMENT

This task involved project administration, data management, and workshops conducted with City personnel.

Project Administration included completing a final schedule of work activities, meeting with City of Fayetteville staff to update previous investigative work, and coordinating upcoming tasks.

Data Management involved review of existing information provided by Fayetteville including GIS maps, facility record drawings, pump curves, reports, zoning requirements, and all other pertinent information.

RJN conducted three workshops to establish methodologies for developing the Master Plan document. These workshops were a valuable tool in gathering additional information from key City of Fayetteville personnel and to keep all stakeholders abreast of the progress and solutions derived during the study. The three workshops were conducted over a period of six months and were held at milestones of the project. These included model development, calibration and future development requirements.

FLOW MONITORING

RJN Group, Inc. performed a system-wide flow monitoring program during the fall of 2012 and again in the spring of 2013. The objective of the flow monitoring was to collect dry and wet-weather flows for model calibration and to determine I/I quantities for each metered basin. The original scope of the services consisted of a 60-day flow monitoring period beginning on May 5, 2012 to capture both dry weather flows and a minimum of three to five significant rain events across the entire service area. However due to a lack of large significant rain events across the entire system and a poor outlook for rain in the next 30 days the meters were pulled on July 2, 2012. An amendment was approved authorizing an additional 60 days of flow monitoring during a period with a greater possibility for significant rain events. The second 60-day flow monitoring period began on January 22, 2013. Flow meter and rain gauges were installed in the same locations as the original monitoring period. This second flow monitoring period had a much higher number of significant rainfall events than the initial monitoring period.

RJN installed thirty seven (37) gravity flow meters and ten (10) rain gauges throughout the study area. Of the 37 flow meters 18 were permanent meters owned by the City and 19 were temporary meters owned by RJN.

POPULATION AND FLOW PROJECTIONS

Based on meetings with the City of Fayetteville, RJN identified areas of potential future growth within the City of Fayetteville Planning Area. This included evaluating available land currently underdeveloped, and considering the future land use as indicated by the City's GIS. These areas were consistent with the Water Master Plan dated November 2011.

DRY-WEATHER FLOWS

A flow rate of 60 gallons per capita per day was used for the future growth areas. This was the average usage rate determined during model calibration. This rate is exclusive of permanent groundwater infiltration and represents only the sewer usage of individual customers.

WET-WEATHER FLOWS

Following the dry-weather flow projections, the subcatchments were programmed to generate wet weather flows. During wet-weather calibration, the system response to RDII was identified and the hydraulic model was calibrated to re-create the observed response within each flow meter basin. The model utilizes a series of coefficients and area distributions to generate wet weather flows as described in Chapter 4. By using this methodology, the

coefficients can be applied to other subcatchments and their wet-weather response will scale with the subcatchment size.

It is anticipated that rainfall derived infiltration/inflow (RDII) rates would be low for future growth areas, assuming good construction practices in new sanitary sewers. The model calibration revealed that existing basin I15-2 had a low RDII rate resulting in only a small response to rainfall. The runoff parameters from existing basin I15-2 were applied to the future growth basins.

This method provides a more accurate model as the rainfall response more closely matches how the system is behaving rather than using a fixed wet weather peaking factor.

SUMMARY OF FLOWS

Table A lists the summary of flows for both dry weather and wet weather.

Table A
DRY AND WET WEATHER CAPACITY FOR BOTH WASTEWATER TREATMENT PLANTS

		Existing	**2030**	**Build-out**
Model Population		78,618	108,570	304,086
		Flow (mgd)		
Average Dry (24 hrs)	Noland	3.92	6.81	14.00
	Westside	5.26	6.15	12.74
	Total	**9.18**	**12.96**	**26.74**
Peak Wet (24 hrs)	Noland	20.39	25.48	41.53
	Westside	29.89	31.70	39.89
	Total	**50.28**	**57.18**	**81.42**

HYDRAULIC MODEL UPDATE

The existing hydraulic model contained a hydraulic network for line segments constructed prior to 1996 in the SWMM XP model. The SWMM model data was imported into the InfoWorks CS model software by Innovyze and was updated to include sewer segments constructed since 1996. For all sewer segments 10-inches in diameter and larger, as well as critical 6-inch and 8-inch diameter sewers, invert and rim elevations were obtained from the existing model or from existing record drawings. For the remaining non critical 6-inch and 8-inch diameter lines, the rim and invert elevations were estimated utilizing the topographical data provided to RJN by the City. Record Drawings of all pump stations and pump curves were reviewed and input into InfoWorks CS. Sub-Basins and catchments were reviewed and some were redefined for better representation in the model. Population and land use data were evaluated and input to reflect current conditions.

MODEL CALIBRATION

This task involved calibrating the updated model for dry and wet-weather conditions using flow data collected during the monitoring task. The following tasks were included in this phase of the project:

- Select dry-weather period from flow data
- Develop dry-weather flow unit equivalents
- Calibrate model to dry-weather weekday and weekend conditions
- Select four storm events with varying rainfall intensities that ideally did not result in overflows in the collection system
- Calibrate model based on selected storm events

HYDRAULIC CAPACITY ANALYSIS

This task included performing a system analysis for existing dry weather conditions. Normal wastewater production and dry weather infiltration flow components versus available pipe capacity was analyzed to determine if any potential dry weather hydraulic capacity problems existed.

RJN consulted with the City and agreed to use the same design storm for the WSIP (5-year/24-hour storm event with 2-year/60-minute peak intensity) for the update.

DESIGN STORM

A 5-year/24-hour storm event with 2-year/60-minute peak intensity, with a peak intensity of approximately 1.80 inches/hour and total rainfall depth of 5.26 inches, shown in Figure 1 was selected as the design storm. The system response to the design storm was assessed to determine the collection system's capacity to transport peak wet weather flow.

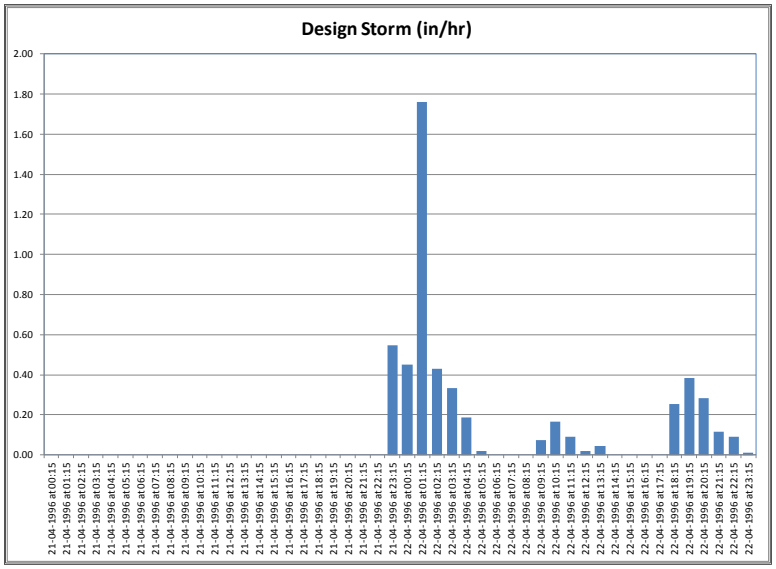


Figure 1

DESIGN CRITERIA

The design criteria for the Fayetteville sewer network is to convey all flows while maintaining a level of surcharge not to exceed three feet below manhole rim elevations.

CAPITAL IMPROVEMENT PLAN

IMMEDIATE PIPELINE IMPROVEMENTS

The required pipeline improvements include localized improvements to convey flow through the collection system with no overflows. The primary target areas are in the vicinity of Ramsey and Overcrest. These areas are the remaining known overflow locations that were not eliminated as part of the WSIP. The estimated construction cost of the recommended immediate capacity improvements is $2.68 million dollars.

2030 IMPROVEMENTS

The following improvements are recommended to eliminate model predicted overflows and to reduce surcharge in the existing sanitary sewer system through the projected 2030 population conditions.

2030 PIPELINE IMPROVEMENTS

The required pipeline improvements include localized improvements to convey flow through the collection system with no overflows as well as new sewer line to convey flow from new development. The estimated construction cost of the recommended capacity improvements through 2030 is $13.69 million dollars. The improvements are shown graphically in Figure 6.3.

2030 PUMP STATION IMPROVEMENTS

As pipeline improvements are constructed, several pump station will be abandoned as listed below.

Copper Creek II
Stonebridge Meadows
Stonebridge Meadows II
Stonebridge Meadows V
Crescent Lake
Meadows
Dot Tipton
Masters

As pipeline improvements are constructed, several pump stations will also need to be upgraded as listed below. The estimated construction cost of the recommended pump station improvements for Ultimate Build-out is $3.81million dollars.

Stonewood
Airport North
Greenland
Industrial Park
Owl Creek

ULTIMATE BUILDOUT

The following improvements are recommended to eliminate model predicted overflows and to reduce surcharge in the existing sanitary sewer system under future Ultimate Build-out conditions.

ULTIMATE BUILDOUT PIPELINE IMPROVEMENTS

The required pipeline improvements include localized improvements to convey flow through the collection system with no overflows as well as new sewer line to convey flow from new development. The estimated construction cost of the recommended capacity improvements for Ultimate Build-out is $57.08 million dollars. The improvements are shown graphically in Figure 6.3.

ULTIMATE BUILDOUT PUMP STATION IMPROVEMENTS

As pipeline improvements are constructed, another pump station will be abandoned as listed below.

McDonald

As development progresses into the Ultimate Build-out areas, the terrain within the City of Fayetteville sewer collection system will require approximately seven new sewer pump stations and upgrades to another nine pump stations. The estimated construction cost of the recommended pump station improvements for Ultimate Build-out is $14.60 million dollars.

IMPROVEMENT SUMMARY

The estimated total capital cost to implement the recommended improvements includes engineering, land acquisition, and contingencies which are estimated to be 30% of the construction cost. A summary of the estimated capital cost is given in Table B.

Table B

SUMMARY OF CAPITAL IMPROVEMENTS

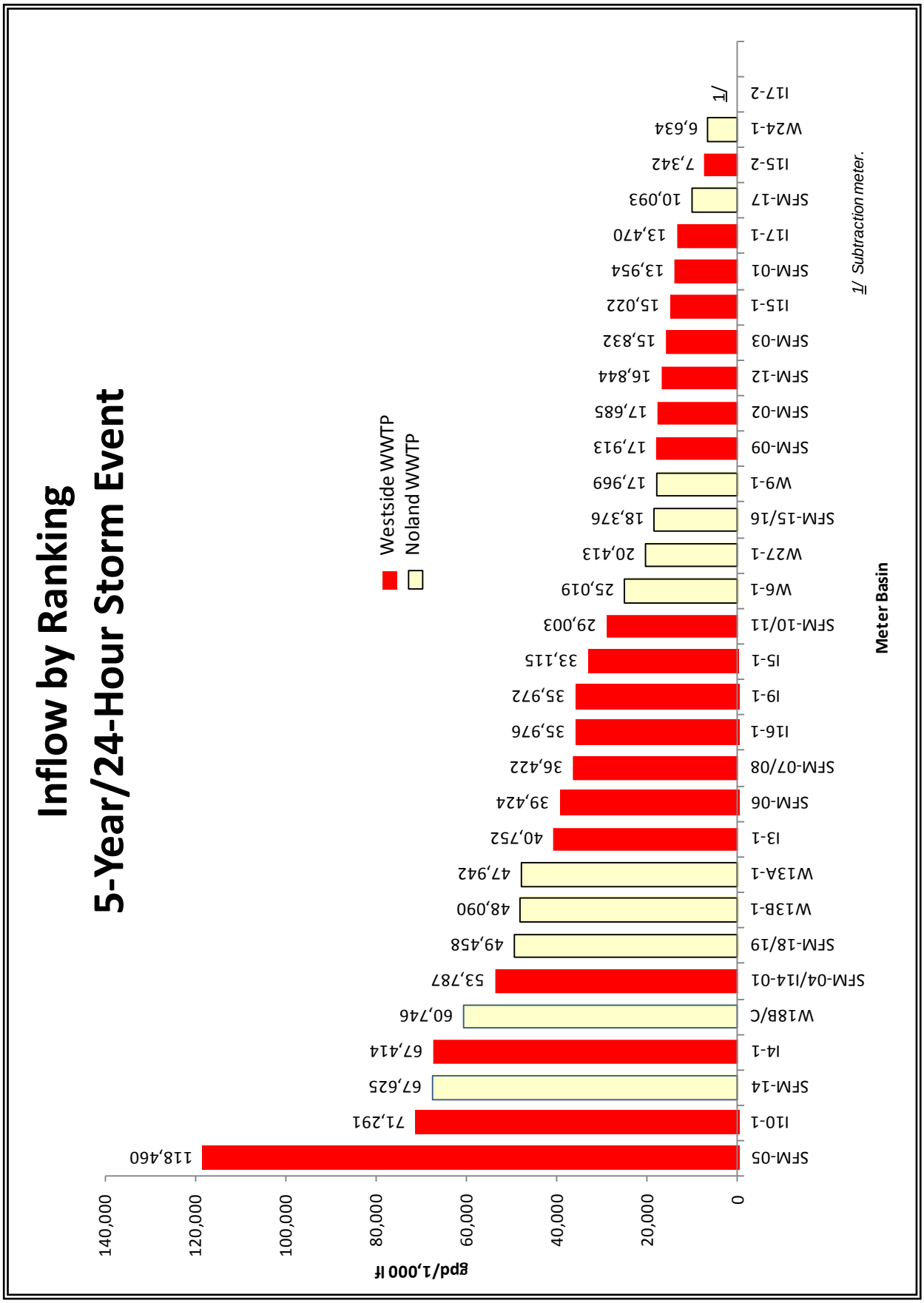
Item	Description	Estimated Capital Improvement Cost ($ Million)[1/]
Pipeline Improvements		
Immediate Improvements		3.48
Improvements Through Year 2030		22.75
Improvements Through Ultimate Build-out		93.19

1/ Estimated Capital Cost is given in 2014 dollars. No cost escalation is included for future years.

FUTURE SSES RECOMMENDATIONS

A significant part of the City's core sewer collection system bounded by I-49 to the west, Clear Creek to the north, Hwy 265 to the east, and Hwy 16 to the south has had SSES and rehab project performed in the 1990s. Some of the areas have not been inspected in over 20 years.

Inflow rankings are shown in Figure 2. The EPA maximum recommended inflow rate is 10,000 gallon per day per 1,000 linear feet of sewer line. It is recommend that the City continue SSES work on the basins identified in Figure 2 beginning with the highest inflow ranked basin (SFM-05) and continuing through Basin SMF-17. Multiple basins can be evaluated in a given year. The approximate maximum sewer line footage that can be inspected annually- is 150,000 linear feet.


Inflow by Ranking
5-Year/24-Hour Storm Event
Westside WWTP
Noland WWTP
gpd/1,000 lf
140,000
120,000
100,000
80,000
60,000
40,000
20,000
0
SFM-05 118,460
I10-1 71,291
SFM-14 67,625
I4-1 67,414
W18B/C 60,746
SFM-04/I14-01 53,787
SFM-18/19 49,458
W13B-1 48,090
W13A-1 47,942
I3-1 40,752
SFM-06 39,424
SFM-07/08 36,422
I16-1 35,976
I9-1 35,972
I5-1 33,115
SFM-10/11 29,003
W6-1 25,019
W27-1 20,413
SFM-15/16 18,376
W9-1 17,969
SFM-09 17,913
SFM-02 17,685
SFM-12 16,844
SFM-03 15,832
I15-1 15,022
SFM-01 13,954
I17-1 13,470
SFM-17 10,093
I15-2 7,342
W24-1 6,634
I17-2 1/
Meter Basin
1/ Subtraction meter.

ADDITIONAL RECOMMENDATIONS

36-inch Sewer Line – Happy Hollow to Noland WWTP

The existing 36-inch diameter ductile iron sewer line that runs from the City of Fayetteville Compost Facility to the Noland WWTP was constructed in the 1960s. A new parallel 42-inch diameter sewer line was constructed in 2008. Both of the sewer lines are necessary to carry the projected sewer flows through ultimate build-out.

The 36-inch ductile iron pipe was made with a polyethylene liner to reduce the effects of the highly corrosive sewer environment. Over the years, portions of the liner have shown up at the screening facility at the Noland WWTP. While the 36-inch ductile iron pipe was used as a force main from the Happy Hollow PS, the corrosion risk was low as long as the pipe was full. Now that the pipe has been converted back to a gravity sewer line and is not running full, the risk of H2S buildup and bacteria converting the gas to sulfuric acid is significantly higher. Sewer maintenance crews have identified holes within the pipe at two different locations within the last year. It is recommended that the City perform a structural integrity inspection of the existing 36-inch ductile iron sewer line.

The estimated cost for inspection services is $117,000. The estimated cost to line 26,500 feet of 36-inch DIP with cured-in-place-pipe could range from $5.96 Million to $8.22 Million depending on the condition of the pipe. For comparison purposes, the parallel 42-inch sewer that was recently installed was constructed for $10.7 Million.

Fox Hunter Sewer Lines

The parallel sewer lines that start at the top of the hill in the vicinity of the Barrington Park subdivision and follows Fox Hunter down to the Noland WWTP have also been identified as a maintenance issue. These lines are identified in the Ultimate Build-out planning as needing upsized. It is recommended that the sewer lines be further evaluated.

Budget prices were not calculated for this project because the length of sewer pipe as well as the diameter will not be determined until an evaluation has been performed.

Rain Gauges

Rain gauges help attribute rainfall to sewer subcatchments and can assist with wet weather flow managements by giving “real-time” rainfall measurements throughout the City. The City’s rain gauges were purchases over 15-years ago. The average life cycle for rain gauges equipment is 10-years. It is recommended that the City purchase new rain gauges and connect them to the Water/Wastewater SCADA system. Approximately 10 rain gauges will be required to cover the City’s wastewater collection system area.

The estimated cost to purchase rain gauges and install telemetry is $17,350.

Flow Meter Telemetry

The City purchased seventeen (17) flow meters to be permanently installed a few years ago. The field technician with the Water/Sewer Department has to visit each site routinely to download data and perform diagnostics checks. Adding telemetry to the existing flow meters would reduce the frequency of site visits and would provide for remote downloading of the flow data and system diagnostics. This has proven well with the SCADA system for monitoring the sewer pump stations. It is recommended that the City pursue installing telemetry on the permanent flow meters.

The estimated cost to install telemetry on the permanent flow meters is $51,000. There will be an ongoing monthly charge depending on the cell service provider for each site.

Model Upkeep

The City has made a significant investment in both money and time to build and calibrate the wastewater model. The model will need to be kept up to date by including infrastructure improvements and checking performance with the City's permanent flow meters. Ideally, the model would be kept up to date as new development is approved for construction and verified as construction is completed. This is a daunting task without a full-time assigned employee. Realistically, the model should be updated with all sewer improvements on an annual basis. The model should be ran with the new improvements added and compared to the data obtained from the permanent flow meters to verify that the model is tracking with the real world flow data.

It is recommended that the wastewater model be re-calibrated approximately every 5-7 years depending on the results of the annual updates. The re-calibration of the model will include the installation of temporary flow meters to supplement the City's permanent flow meter network. Sub-Basins and catchments will be reviewed and updated. Population and land use data will also be evaluated and updated to reflect current conditions.

The estimate annual cost to retain a consultant to upkeep the model is $25,000. This will involve adding new sewer lines to the model as a result of development, sewer rehabilitation, and sewer line replacement.

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CHAPTER 1

INTRODUCTION

This report presents the results of the Wastewater Collection System Model and Master Plan Update that the City of Fayetteville retained RJN Group, Inc. to perform.

BACKGROUND AND OBJECTIVES

The wastewater system contains over 540 mile of main sewer and two WWTPs and serves over 80,000 customers.

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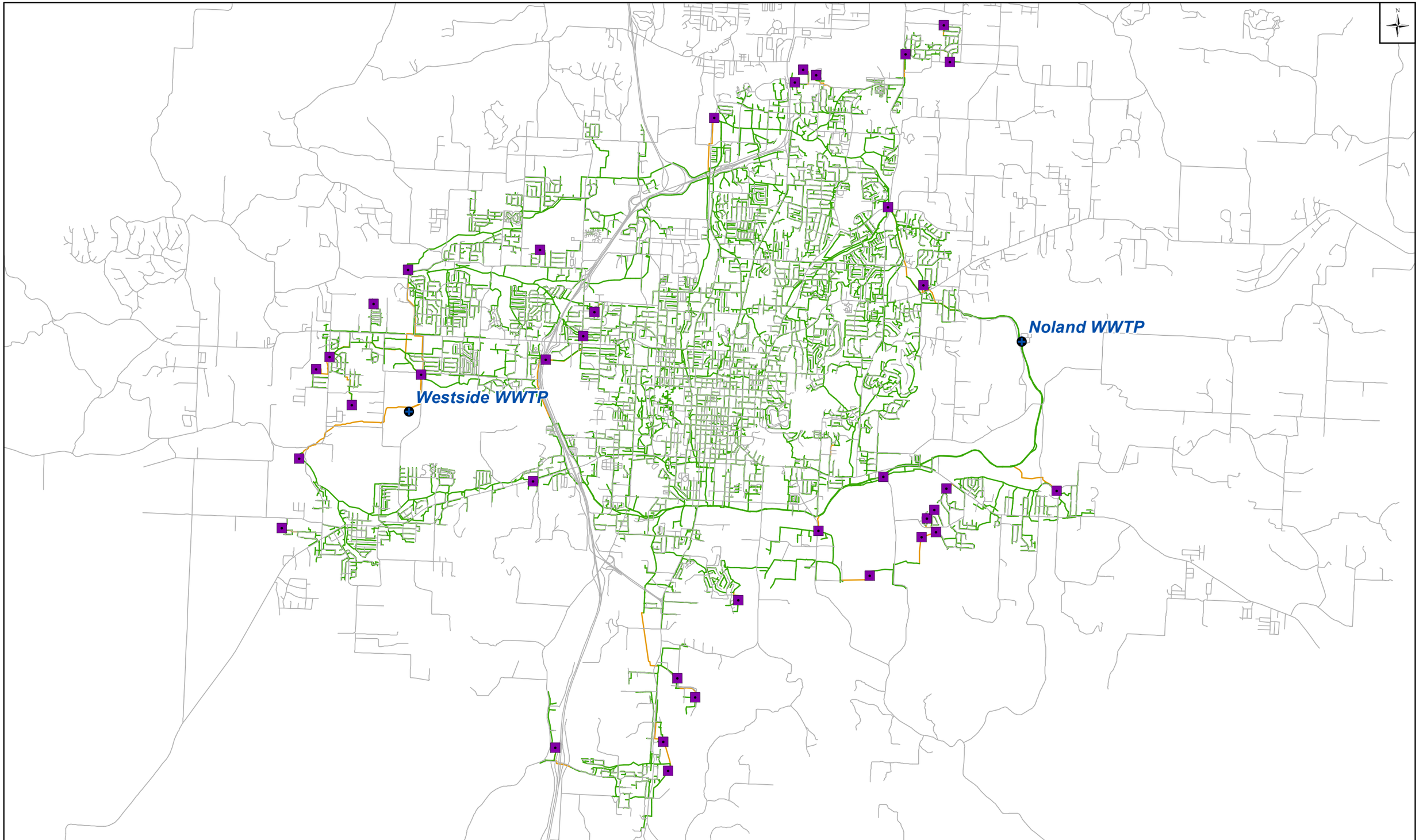
The changes made to the system as part of the WSIP resulted in the hydraulic model developed in 1996 to now be outdated and does not provide the City the ability to evaluate the impact of new developments on the capacity of the collection system and treatment system. This Master Plan Update Project will update the City's hydraulic model to reflect the current conditions and will also evaluate the impact of future growth within the City's Service Area. The hydraulic model will be provided to the City as part of the deliverables of the project.

A system overview map is shown in Figure 1-1 on page 1-2.



rjngroup
The Choice for Collection System Solutions

Treatment Plants
Lift Stations
Gravity Mains
Force Mains
Streets

CITY OF Fayetteville ARKANSAS

System Overview Map
City of Fayetteville Master Plan
Figure 1.1

SCOPE OF MASTER PLAN DEVELOPMENT

RJN Group, Inc. was authorized by the City of Fayetteville under an agreement dated May 15, 2012 to provide a Wastewater Collection System Model and Master Plan Update.

The scope of the project and a brief description of each task are discussed below:

PROJECT ADMINISTRATION AND MANAGEMENT

This task involved project administration, data management, and workshops conducted with City personnel.

Project Administration included completing a final schedule of work activities, meeting with City of Fayetteville staff to update previous investigative work, and coordinating upcoming tasks.

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FLOW MONITORING

RJN reviewed the collection system GIS maps and operational information for the collection system to select strategic flow monitoring locations. The City's existing permanent flow meters were used and supplemented with nineteen (19) temporary flow meters owned by RJN. In addition, ten (10) rain gauges were installed. The location of the flow meters and rain gauges are shown in Exhibit 1. The flow monitoring occurred during the fall of 2012 and again in the spring of 2013 due to insufficient rainfall events occurring during the 2012 monitoring period.

DEVELOPMENT OF FUTURE FLOWS

Based on meetings with the City of Fayetteville, RJN identified areas of potential future growth within the City of Fayetteville Planning Area. This included evaluating available land currently underdeveloped, and considering the future land use as indicated by the City's GIS. These areas were consistent with the Water Master Plan dated November 2011. The future growth areas are shown in Figure 3-3 on page 3-6.

HYDRAULIC MODEL UPDATE

The existing hydraulic model contained a hydraulic network for line segments constructed prior to 1996 in the SWMM XP model. The SWMM model data was imported into the InfoWorks CS model software by Innovyze and was updated to include sewer segments

constructed since 1996. For all sewer segments 10-inches in diameter and larger, as well as critical 6-inch and 8-inch diameter sewers, invert and rim elevations were obtained from the existing model or from existing record drawings. For the remaining non critical 6-inch and 8-inch diameter lines, the rim and invert elevations were estimated utilizing the topographical data provided to RJN by the City. Record Drawings of all pump stations and pump curves were reviewed and input into InfoWorks. Sub-Basins and catchments were reviewed and some were redefined for better representation in the model. Population and land use data were evaluated and input to reflect current conditions.

MODEL CALIBRATION

This task involved calibrating the updated model for dry and wet-weather conditions using flow data collected during the monitoring task. The following tasks were included in this phase of the project:

- Select dry-weather period from flow data
- Develop dry-weather flow unit equivalents
- Calibrate model to dry-weather weekday and weekend conditions
- Select four storm events with varying rainfall intensities that ideally did not result in overflows in the collection system
- Calibrate model based on selected storm events

HYDRAULIC CAPACITY ANALYSIS

This task included performing a system analysis for existing dry weather conditions. Normal wastewater production and dry weather infiltration flow components versus available pipe capacity was analyzed to determine if any potential dry weather hydraulic capacity problems existed. RJN consulted with the City and agreed to use the same design storm for the WSIP (5-year/24-hour storm event with 2-year/60-minute peak intensity) for the update. Using the selected design storm, the following tasks were included in this phase:

- System hydraulic analysis under existing population conditions
- System hydraulic analysis under a 2030 population
- System hydraulic analysis under ultimate buildout
- Estimate average daily flow, peak hour flow, and peak day flows for each WWTP for the two population scenarios

MASTER PLAN UPDATE

This task included evaluating the existing system performance and developing necessary improvement alternatives to transport the projected wastewater flows. The following tasks were included in this phase:

- Prioritization of each monitored area for future SSES and rehabilitation efforts

- Necessary capacity improvements for full conveyance to the WWTPs
- Peak flow management
- Peak wastewater flows to each WWTP
- Develop Master Plan Update Capital Improvement Plan
- Delivery of the InfoWorks CS model

REPORTING

This task included the creation of draft and final engineering reports summarizing the results of all previous tasks. The reporting describes work performed during various tasks, procedures and methodologies used, alternatives evaluated and required improvement plan, recommended maintenance plan as well as cost estimates.

The following sections of this report include the results of each task. Appendices to this report include documentation for the Master Plan Update including flow monitoring data, model data, recommended capacity improvements, and accompanying cost estimates.

CHAPTER 2

FLOW MONITORING

This chapter presents the flow monitoring data and how it was used in the model development. It provides a summary of the flow monitoring data for calibration of the model, inflow / infiltration (I/I) analysis, and development of dry-weather flow.

FLOW MONITORING SUMMARY

RJN Group, Inc. performed a system-wide flow monitoring program during the fall of 2012 and again in the spring of 2013. The objective of the flow monitoring was to collect dry and wet-weather flows for model calibration and to determine I/I quantities for each metered basin. The original scope of the services consisted of a 60-day flow monitoring period beginning on May 5, 2012 to capture both dry weather flows and a minimum of three to five significant rain events across the entire service area. However due to a lack of large significant rain events across the entire system and a poor outlook for rain in the next 30 days the meters were pulled on July 2, 2012. An amendment was approved authorizing an additional 60 days of flow monitoring during a period with a greater possibility for significant rain events. The second 60-day flow monitoring period began on January 22, 2013. Flow meter and rain gauges were installed in the same locations as the original monitoring period. This second flow monitoring period had a much higher number of significant rainfall events than the initial monitoring period.

RJN installed thirty seven (37) gravity flow meters and ten (10) rain gauges throughout the study area. Of the 37 flow meters 18 were permanent meters owned by the City and 19 were temporary meters owned by RJN. The gravity flow monitoring locations are given in Table 2-A and shown on Exhibit 1. Rain gauge location, recorded totals, and intensities are listed in Table 2-B. An area map denoting the locations of the flow meters and rain gauges, plus site sheets depicting each location are shown in the Appendix. A basin flow diagram indicating direction of flow from one basin to another is shown on Figure 2-1.

AVERAGE DAILY DRY-WEATHER FLOW

Flow data collected during dry-weather/low-groundwater periods was analyzed to determine the average daily dry-weather flow for each of the thirty (30) basins. The dry-weather period selected for this analysis was from March 1, 2013 through March 7, 2013. The analysis determined that the average daily dry-weather flow during the monitoring period was approximately 8.249 mgd. It should be noted that during the monitoring period, this week was the best suited for dry-weather analysis.

A summary of average daily dry-weather flow by basin is given in Table 2-C and is shown graphically on page 2-11. A basin flow diagram giving average daily dry-weather flow is shown in Figure 2-2. Hydrographs of the dry-weather flow overlaid with wet-weather periods for each basin are also included in Appendix A at the end of this report.

Table 2-A (Cont.)

FLOW METER LOCATIONS

Meter Number	WWTP Tributary Area	Manhole Number	Address	Diameter (in)
Permanent Meters				
SFM-01	Westside WWTP	I-(23) 001	571 N Doublesprings Road	21
SFM-02	Westside WWTP	I-(15) 003	2081 CR 877	24
SFM-03	Westside WWTP	Unknown	4143 Fire Fly Catch	48
SFM-04	Westside WWTP	Unknown	Remote	24
SFM-05	Westside WWTP	Unknown	450 Drake	36
SFM-06	Westside WWTP	Unknown	756 W North Street	24
SFM-07	Westside WWTP	I-(01) 111	3600 N Gregg	15
SFM-08	Westside WWTP	I-(01) 0008	3600 N Gregg	15
SFM-09	Westside WWTP	Unknown	3770 N Mall Avenue	30
SFM-10	Westside WWTP	I-(06) 001	Old Wire Road & Crossover Road	12
SFM-11	Westside WWTP	I-(06) 001	2911 N Crossover	18
SFM-12	Westside WWTP	I-(16) 003	3016 Summer Shade Drive	18
SFM-14	Noland WWTP	W-(43) 004	Noland South	14
SFM-15	Noland WWTP	Unknown	Noland @ WWTP	43
SFM-16	Noland WWTP	Unknown	1400 Fox Hunter Drive	37
SFM-17	Noland WWTP	Unknown	Molly Wagon Lift Station	18
SFM-18	Noland WWTP	W-(15) 005B	Enterprise @ Armstrong	30
SFM-19	Noland WWTP	Unknown	Enterprise @ Armstrong	43
Temporary Meters				
I3-1	Westside WWTP	I-(03) 001	Gregg St & Appleby Street	15
I4-1	Westside WWTP	I-(04) 001	2025 Greenacres Road	15
I5-1	Westside WWTP	I-(17) 137	3452 Old Missouri Road	18
I9-1	Westside WWTP	I-(09) 001	1502 E Overcrest Street	12
I10-1	Westside WWTP	I-(12) 002	598 Frisco Avenue	17
I14-1	Noland WWTP	W-(07) 027	Shiloh Road	21
I15-1	Westside WWTP	I-(15B) 102	630 N Double Springs Road	12
I15-2	Westside WWTP	I-(15B) 129	480 Broyles Avenue	18
I16-1	Westside WWTP	I-(16) 121	2706 Golden Oaks	12
I17-1	Westside WWTP	I-(17) 046	1465 E Joyce Boulevard	10
I17-2	Westside WWTP	Unknown	Sweetbriar Park	24
W6-1	Noland WWTP	W-(06) 052	923 W Cato Springs	24
W9-1	Noland WWTP	W-(09) 013	2428 Huntsville	12

Table 2-A (Cont.)

FLOW METER LOCATIONS

Meter Number	WWTP Tributary Area	Manhole Number	Address	Diameter (in)
W13A-1	Noland WWTP	W-(13A) 002	1330 Martin Luther King	12
W13B-1	Noland WWTP	W-(15) 081S	1151 Indian Trail	24
W18B-1	Noland WWTP	W-(18B/C) 504	1299 S Block Avenue	30
W18C-1	Noland WWTP	W-(18B/C) 185R	1299 S Block Avenue	18
W24-1	Noland WWTP	W-(24B) 073	Harvey Dowell Road & Mally Wagnon Road	12
W27-1	Noland WWTP	W-(15X) 301	1936 S Armstrong Road	24

Table 2-B

RAINFALL SUMMARY

Date	FRG-01 1400 N Fox Hunter Rd Noland WWTP Total Daily Rainfall (in)	FRG-01 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-02 3917 S McCollum Lift Station #16 Total Daily Rainfall (in)	FRG-02 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-03 3021 N Old Wire Rd Lift Station #6 Total Daily Rainfall (in)	FRG-03 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-04 3896 N Gregg Lift Station #5 Total Daily Rainfall (in)	FRG-04 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-05 764 W North St St James Missionary Baptist Church Total Daily Rainfall (in)	FRG-05 Peak 60-Minute Rainfall Intensity (in/hr)
2012										
May 13	1/	1/	1/	1/	1/	1/	1/	1/	1/	1/
19	1/	1/	1/	1/	1/	1/	0.21	0.16	1/	1/
29	0.38	0.27	0.29	0.19	0.37	0.3	0.36	0.30	0.30	0.13
30	1/	1/	1/	1/	1/	1/	1/	1/	1/	1/
31	0.41	0.41	0.33	0.33	0.49	0.49	0.41	0.41	0.50	0.05
June 3	1.01	0.91	1.11	0.65	1.55	1.53	1.21	1.20	1.39	1.39
4	0.41	0.20	0.61	0.18	0.37	0.15	0.42	0.19	0.41	0.17
21	0.23	0.13	0.26	0.15	0.23	0.13	0.37	0.22	0.20	0.11
Subtotal	2.44		2.60		3.01		2.98		2.80	
2013										
Jan. 29	2.26	1.13	1/	1/	2.55	1.22	2.49	0.99	2.49	0.99
Feb. 7	0.31	0.21	0.29	0.17	0.29	0.18	0.20	0.14	0.20	0.14
10	0.45	0.18	0.36	0.15	0.64	0.28	0.37	0.18	0.37	0.18
12	0.29	0.10	0.35	0.11	0.30	0.09	0.23	0.07	0.23	0.07
13	0.02	0.02	0.06	0.04	0.01	0.01	0.02	0.01	0.02	0.01
18	0.49	0.35	0.29	0.22	0.19	0.10	0.19	0.13	0.19	0.13
20	0.17	0.13	0.29	0.19	0.18	0.15	0.20	0.11	0.20	0.11

1/ No rainfall recorded.

Table 2-B

RAINFALL SUMMARY

Date	FRG-01 1400 N Fox Hunter Rd Noland WWTP Total Daily Rainfall (in)	FRG-01 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-02 3917 S McCollum Lift Station #16 Total Daily Rainfall (in)	FRG-02 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-03 3021 N Old Wire Rd Lift Station #6 Total Daily Rainfall (in)	FRG-03 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-04 3896 N Gregg Lift Station #5 Total Daily Rainfall (in)	FRG-04 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-05 764 W North St St James Missionary Baptist Church Total Daily Rainfall (in)	FRG-05 Peak 60-Minute Rainfall Intensity (in/hr)
21	0.29	0.15	0.27	0.19	0.23	0.12	0.48	0.42	0.48	0.42
25	0.49	0.21	0.45	0.20	0.37	0.13	0.39	0.15	0.39	0.15
March 9	0.57	0.19	0.55	0.23	0.72	0.25	0.62	0.21	0.76	0.28
10	1.07	0.24	1.07	0.16	0.99	0.19	1.06	0.16	1.04	0.19
21	0.33	0.12	0.31	0.09	0.32	0.11	0.28	0.09	0.32	0.15
22	0.38	0.18	0.24	0.15	0.51	0.29	0.49	0.26	0.18	0.08
29	1/	1/	0.72	0.62	1/	1/	0.88	0.32	1/	1/
30	1/	1/	0.83	0.28	1/	1/	0.65	0.28	1/	1/
April 2	1/	1/	0.58	0.16	1/	1/	0.31	0.10	1/	1/
8	1/	1/	1/	1/	1/	1/	1/	1/	1/	1/
Subtotal	7.12		6.66		7.30		8.86		6.87	
Total	**9.56**		**9.26**		**10.31**		**11.84**		**9.67**	

1/ No rainfall recorded.

Table 2-B (Cont.)

RAINFALL SUMMARY

Date	FRG-06 1555 E Mission Blvd Root Elementary Total Daily Rainfall (in)	FRG-06 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-07 123 N Block Avenue Buckley, McLemore & Hudson Attorneys at Law Total Daily Rainfall (in)	FRG-07 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-08 2065 N Sunshine Rd Lift Station #7 Total Daily Rainfall (in)	FRG-08 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-09 1 N Broyles Rd Westside WWTP Total Daily Rainfall (in)	FRG-09 Peak 60-Minute Rainfall Intensity (in/hr)	FRG-10 310 W Main Citgo Total Daily Rainfall (in)	FRG-10 Peak 60-Minute Rainfall Intensity (in/hr)
2012										
May 13	0.12	0.11	1/	1/	1/	1/	1/	1/	1/	1/
19	1/	1/	1/	1/	1/	1/	1/	1/	1/	1/
29	0.46	0.31	0.45	0.18	0.46	0.27	0.36	0.18	0.13	0.1
30	1/	1/	1/	1/	1/	1/	0.14	0.14	0.22	0.20
31	1/	1/	0.58	0.58	0.56	0.56	0.48	0.48	0.44	0.44
June 3	1/	1/	1.01	1.01	1.70	1.69	1.21	1.20	1.04	0.72
4	1/	1/	0.50	0.17	0.67	0.25	0.69	0.21	0.52	0.16
21	1/	1/	0.19	0.11	0.52	0.36	0.47	0.34	0.31	0.17
Subtotal	0.58		2.73		3.91		3.35		2.66	
2013										
Jan. 29	1/	1/	2.79	1.15	2.79	1.23	2.32	0.98	2.33	0.93
Feb. 7	0.28	0.19	0.44	0.22	0.31	0.22	1/	1/	0.23	0.17
10	0.55	0.28	0.55	0.29	0.36	0.17	0.41	0.20	0.44	0.19
12	0.15	0.10	0.35	0.11	0.27	0.08	0.25	0.08	0.28	0.08
13	0.19	0.14	0.07	0.04	0.02	0.01	0.02	0.02	0.02	0.01

1 *No rainfall recorded.*

Table 2-B (Cont.)

RAINFALL SUMMARY

	FRG-06		FRG-07		FRG-08		FRG-09		FRG-10	
	1555 E Mission Blvd		123 N Block Avenue		2065 N Sunshine Rd		1 N Broyles Rd		310 W Main	
	Root Elementary		Buckley, McLemore & Hudson Attorneys at Law		Lift Station #7		Westside WWTP		Citgo	
Date	**Total Daily Rainfall (in)**	**Peak 60-Minute Rainfall Intensity (in/hr)**	**Total Daily Rainfall (in)**	**Peak 60-Minute Rainfall Intensity (in/hr)**	**Total Daily Rainfall (in)**	**Peak 60-Minute Rainfall Intensity (in/hr)**	**Total Daily Rainfall (in)**	**Peak 60-Minute Rainfall Intensity (in/hr)**	**Total Daily Rainfall (in)**	**Peak 60-Minute Rainfall Intensity (in/hr)**
Feb.18	0.26	0.18	0.31	0.3	0.28	0.22	0.18	0.15	0.18	0.15
20	0.17	0.12	0.23	0.14	0.36	0.17	0.27	0.14	0.32	0.15
21	0.65	0.51	0.60	0.47	0.40	0.18	0.26	0.21	0.29	0.21
25	0.41	0.17	0.54	0.21	0.41	0.15	0.34	0.14	0.52	0.22
March 9	0.73	0.26	1/	1/	0.95	0.29	0.95	0.27	0.96	0.24
10	0.99	0.18	1/	1/	1.08	0.17	1.13	0.20	1.02	0.18
21	0.33	0.13	1/	1/	0.37	0.13	0.31	0.13	0.29	0.16
22	0.26	0.12	1/	1/	0.24	0.11	0.13	0.09	0.24	0.11
29	0.66	0.27	1/	1/	1/	1/	1.46	1.12	1/	1/
30	0.90	0.57	1/	1/	1/	1/	0.71	0.31	1/	1/
April 2	0.51	0.11	1/	1/	1/	1/	0.60	0.14	1/	1/
8	0.14	0.14	1/	1/	1/	1/	0.38	0.32	1/	1/
Subtotal	7.18		5.88		7.84		9.72		7.12	
Total	**7.76**		**8.61**		**11.75**		**13.07**		**9.78**	

1/ No rainfall recorded.

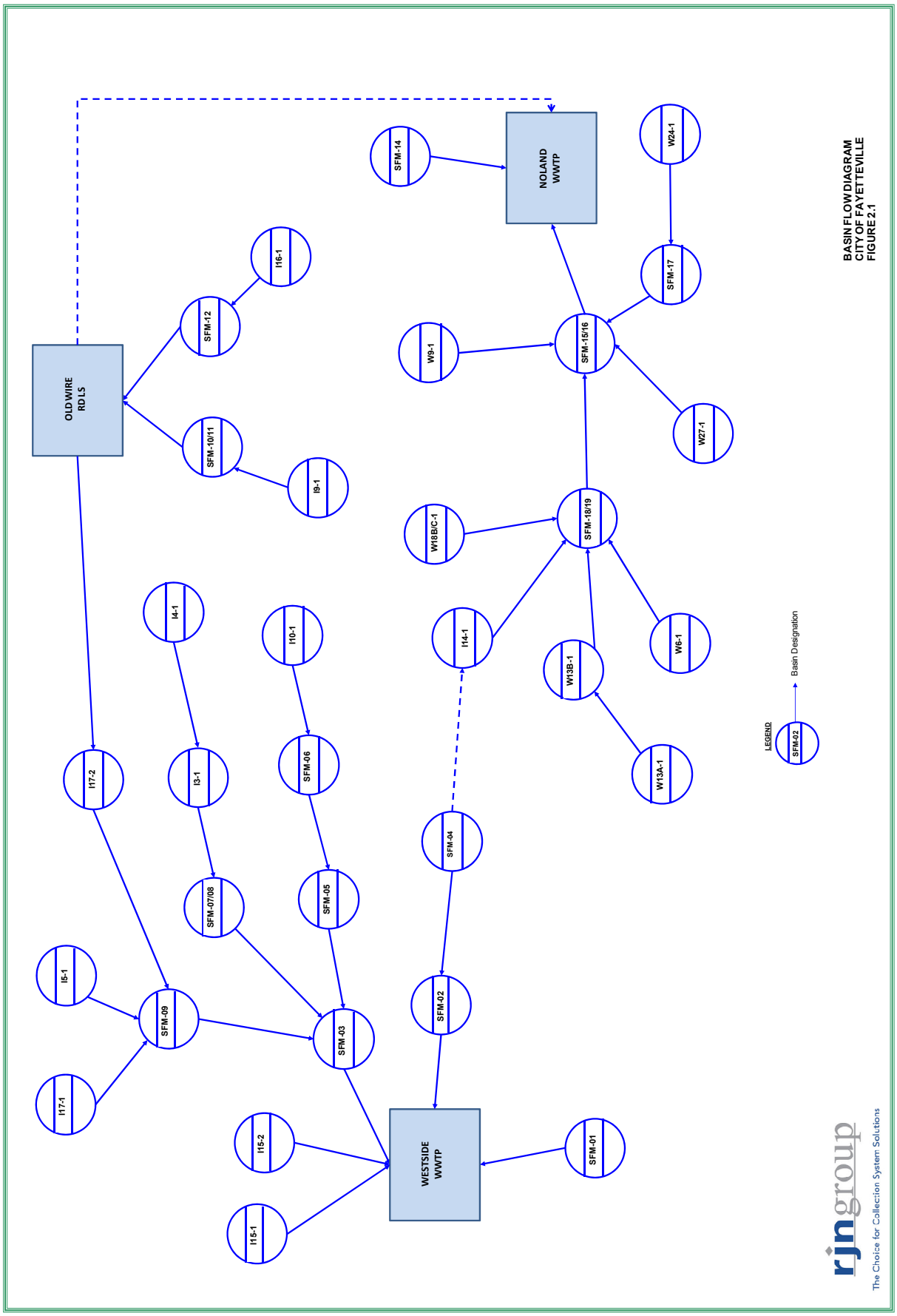
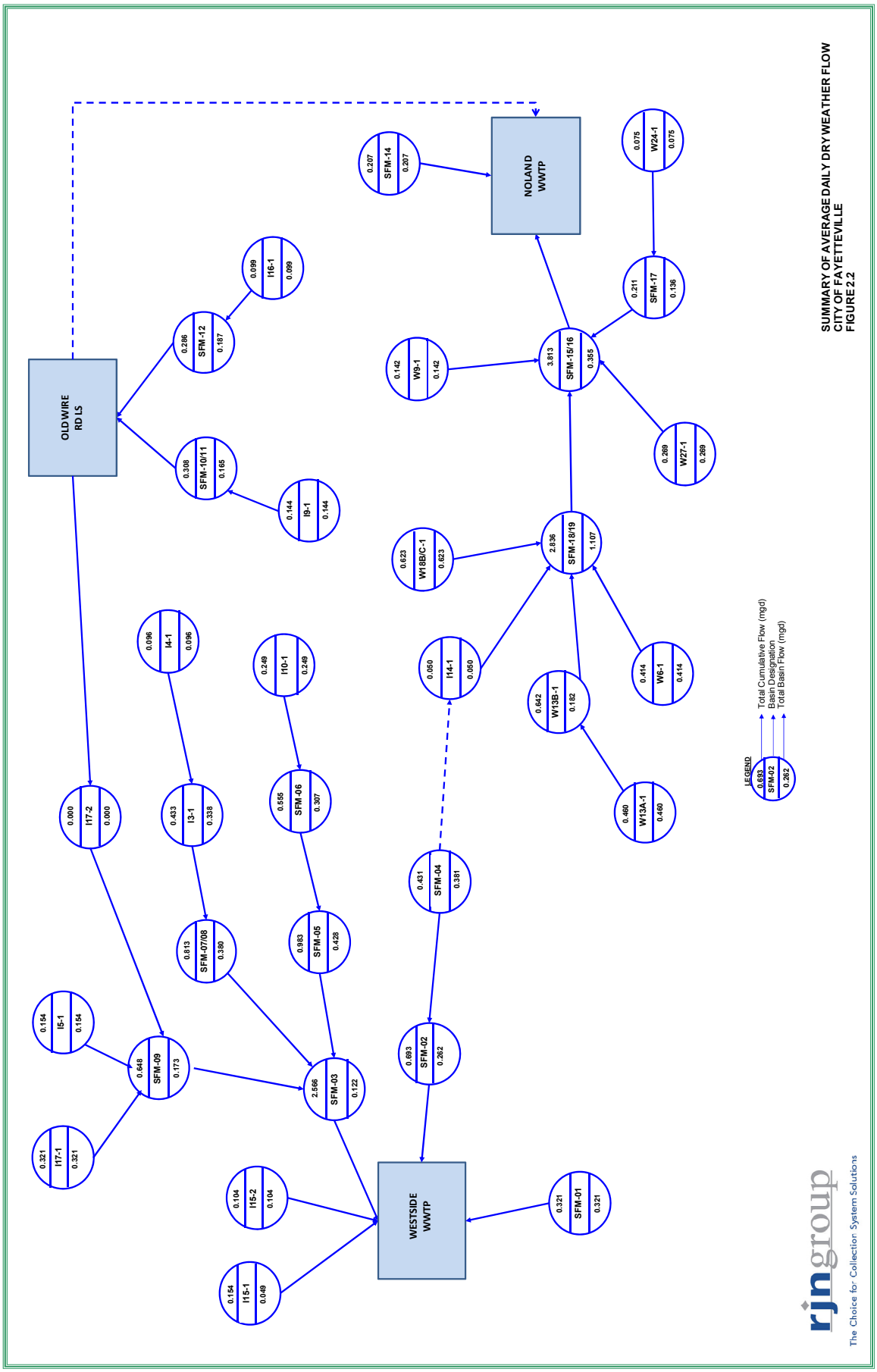

OLD WIRE RD LS
NOLAND WWTP
WESTSIDE WWTP
SFM-14
W24-1
SFM-17
I16-1
SFM-12
W9-1
SFM-15/16
W27-1
SFM-10/11
I9-1
W18B/C-1
SFM-18/19
I4-1
I10-1
I14-1
W13B-1
W6-1
I17-2
I3-1
SFM-06
W13A-1
SFM-04
SFM-07/08
SFM-05
I5-1
SFM-09
SFM-03
SFM-02
I17-1
I15-2
SFM-01
I15-1
LEGEND
SFM-02
Basin Designation
BASIN FLOW DIAGRAM
CITY OF FAYETTEVILLE
FIGURE 2.1
rjngroup
The Choice for Collection System Solutions

Table 2-C

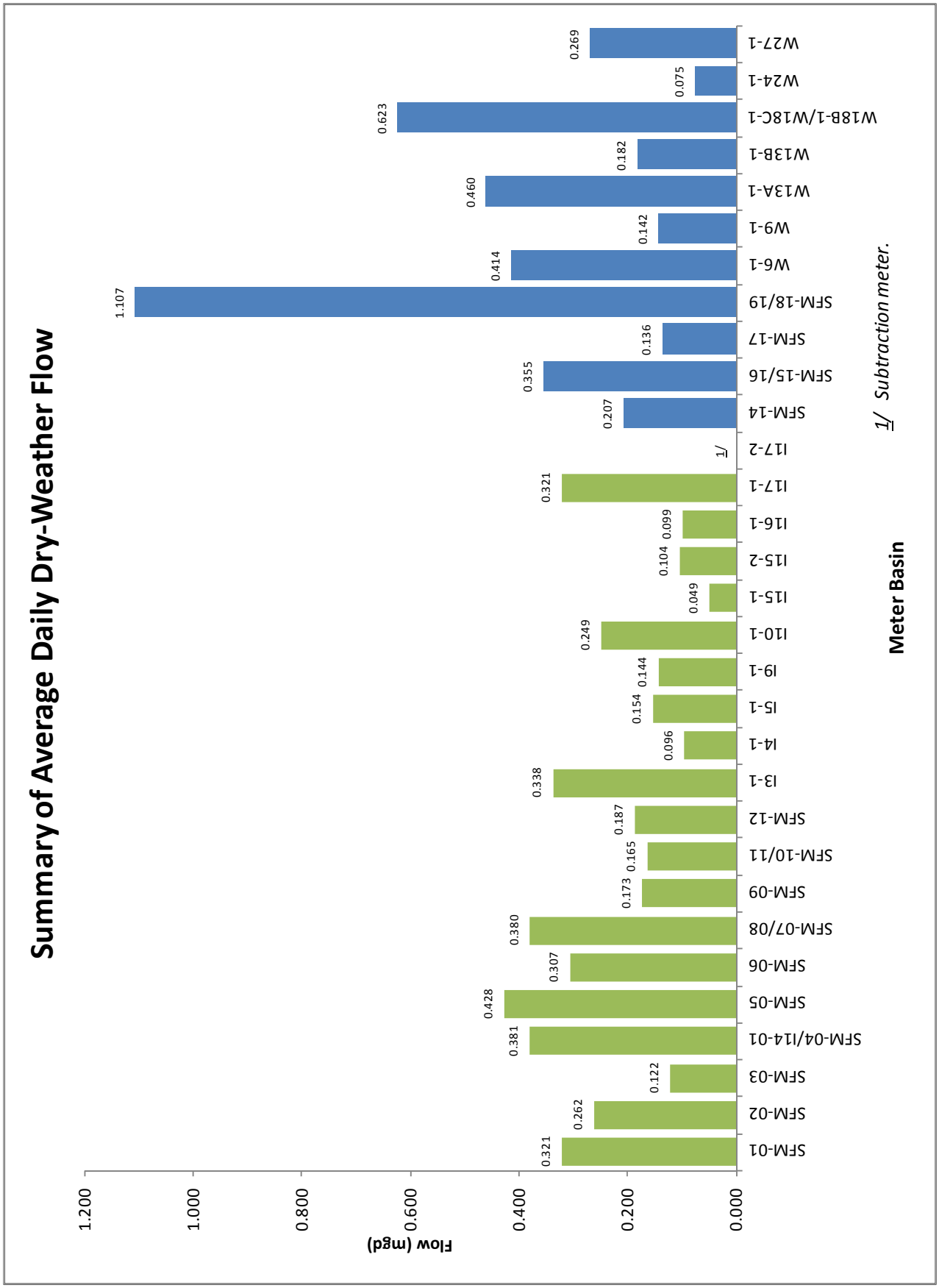
SUMMARY OF AVERAGE DAILY DRY-WEATHER FLOW

Meter Basin	Meter Type	Cumulative Average Daily Dry-Weather Flow (mgd)	Basin Average Daily Dry-Weather Flow (mgd)
Westside WWTP			
SFM-01	Permanent	0.321	0.321
SFM-02	Permanent	0.693	0.262
SFM-03	Permanent	2.566	0.122
SFM-04/I14-01	Permanent	0.431	0.381
SFM-05	Permanent	0.983	0.428
SFM-06	Permanent	0.555	0.307
SFM-07/08	Permanent	0.813	0.380
SFM-09	Permanent	0.648	0.173
SFM-10/11	Permanent	0.308	0.165
SFM-12	Permanent	0.286	0.187
I3-1	Temporary	0.433	0.338
I4-1	Temporary	0.096	0.096
I5-1	Temporary	0.154	0.154
I9-1	Temporary	0.144	0.144
I10-1	Temporary	0.249	0.249
I15-1	Temporary	0.154	0.049
I15-2	Temporary	0.104	0.104
I16-1	Temporary	0.099	0.099
I17-1	Temporary	0.321	0.321
I17-2	Temporary	0.000	1/
Subtotal			4.278
Noland WWTP			
SFM-14	Permanent	0.207	0.207
SFM-15/16	Permanent	3.813	0.355
SFM-17	Permanent	0.211	0.136
SFM-18/19	Permanent	2.836	1.107
W6-1	Temporary	0.414	0.414
W9-1	Temporary	0.142	0.142
W13A-1	Temporary	0.460	0.460
W13B-1	Temporary	0.642	0.182
W18B-1/W18C-1	Temporary	0.623	0.623
W24-1	Temporary	0.075	0.075
W27-1	Temporary	0.269	0.269
Subtotal			3.970
Total			**8.249**

1/ Subtraction meter.



SUMMARY OF AVERAGE DAILY DRY WEATHER FLOW
CITY OF FAYETTEVILLE
FIGURE 2.2


Summary of Average Daily Dry-Weather Flow
Flow (mgd)
1.200
1.000
0.800
0.600
0.400
0.200
0.000
SFM-01
SFM-02
SFM-03
SFM-04/I14-01
SFM-05
SFM-06
SFM-07/08
SFM-09
SFM-10/11
SFM-12
I3-1
I4-1
I5-1
I9-1
I10-1
I15-1
I15-2
I16-1
I17-1
I17-2
SFM-14
SFM-15/16
SFM-17
SFM-18/19
W6-1
W9-1
W13A-1
W13B-1
W18B-1/W18C-1
W24-1
W27-1
0.321
0.262
0.122
0.381
0.428
0.307
0.380
0.173
0.165
0.187
0.338
0.096
0.154
0.144
0.249
0.049
0.104
0.099
0.321
1/
0.207
0.355
0.136
1.107
0.414
0.142
0.460
0.182
0.623
0.075
0.269
Meter Basin
1/ Subtraction meter.

AVERAGE DAILY DRY-WEATHER FLOW PEAKING FACTOR

Wastewater flow during dry-weather periods will vary during the day in response to water consumption. By examining the diurnal curves for each monitored drainage basin, a peaking factor was determined. The peaking factor is the ratio of the peak hourly flow rate and the average daily flow. The average peaking factor was 1.75. Peaking factors varied from a 1.42 to 2.79 and are given for each basin in Table 2-D and is shown graphically on page 2-14.

INFILTRATION CONDITIONS

Infiltration may enter the system through pipe joints, sewer line defects (including main sewer lines and building sewer lines), and defective manhole walls, benches, and pipe seals. There are two types of infiltration that can be determined during a study, permanent infiltration and peak infiltration. Permanent infiltration is defined as extraneous flow that enters the sewer system through the ground during periods of dry-weather and low-groundwater. Peak infiltration is defined as the maximum extraneous flow that enters the sanitary sewer system during high-groundwater conditions after the inflow effects of a rain event have ended. Peak infiltration was used to evaluate the effects of infiltration on the sewer system.

PEAK INFILTRATION

Determining peak infiltration requires analysis of flow data obtained during dry-weather/high-groundwater conditions. Care must be exercised in the analysis to exclude days that are too close to rainfall events. This is necessary to avoid including residual inflow (rainfall induced infiltration) that may lead to an over-estimation of peak infiltration. Generally, periods following significant rainfall, excluding the day immediately following a rain event, are used for determining peak infiltration. Due to minimal amounts of rainfall and the region being in a drought condition during the study, it was determined that groundwater conditions were less than favorable to determine peak infiltration conditions during the monitoring period, although infiltration rates for the period was estimated

For this study, each basin is compared relative to the others by expressing the measured infiltration rate in units of gallons per day/inch-diameter mile (gpd/idm) of pipe. It was determined that one (1) basin exhibited significant infiltration with rates in excess of 5,000 gpd/idm. The system resulted in a total peak infiltration rate of 7.145 mgd. A basin flow diagram showing "peak" infiltration is shown on Figure 2-3. Table 2-E shows the results of the analysis and shown graphically on page 2-17. It should be noted that peak infiltration rates would be expected to be greater during a normal rainfall period versus the drought conditions that occurred during this monitoring period.

Table 2-D

DRY-WEATHER FLOW PEAKING FACTORS

Meter Number	Meter Type	Cumulative Average Daily Dry-Weather Flow (mgd)	Peak Hourly Flow Rate (mgd)	Dry-Weather Flow Peaking Factor
Westside WWTP				
SFM-01	Permanent	0.321	0.549	1.71
SFM-02	Permanent	0.693	0.993	1.43
SFM-03	Permanent	2.566	3.788	1.48
SFM-04	Permanent	0.381	0.518	1.36
SFM-05	Permanent	0.983	1.622	1.65
SFM-06	Permanent	0.555	1.158	2.08
SFM-07	Permanent	0.804	1.153	1.43
SFM-08	Permanent	0.009	0.017	1.83
SFM-09	Permanent	0.648	0.973	1.50
SFM-10	Permanent	0.308	0.534	1.73
SFM-11	Permanent	0.000	0.000	1/
SFM-12	Permanent	0.286	0.486	1.70
I3-1	Temporary	0.433	0.622	1.43
I4-1	Temporary	0.096	0.178	1.87
I5-1	Temporary	0.154	0.286	1.86
I9-1	Temporary	0.144	0.267	1.86
I10-1	Temporary	0.249	0.598	2.40
I14-01	Temporary	0.050	0.138	2.79
I15-1	Temporary	0.154	0.338	2.20
I15-2	Temporary	0.104	0.234	2.24
I16-1	Temporary	0.099	0.164	1.66
I17-1	Temporary	0.321	0.583	1.82
I17-2	Temporary	0.000	0.000	1/
Subtotal				1.81 (average)
Noland WWTP				
SFM-14	Permanent	0.207	0.569	2.74
SFM-15	Permanent	3.650	5.185	1.42
SFM-16	Permanent	0.162	0.298	1.83
SFM-17	Permanent	0.211	0.304	1.44
SFM-18	Permanent	0.000	0.000	1/
SFM-19	Permanent	2.836	4.188	1.48
W6-1	Temporary	0.414	0.597	1.44
W9-1	Temporary	0.142	0.231	1.62
W13A-1	Temporary	0.460	0.783	1.70
W13B-1	Temporary	0.642	1.012	1.58
W18B-1	Temporary	0.344	0.507	1.47
W18C-1	Temporary	0.279	0.397	1.42
W24-1	Temporary	0.075	0.156	2.07
W27-1	Temporary	0.269	0.462	1.72
Subtotal				1.69
Total				**1.75** (Average)

1/ Relief line.

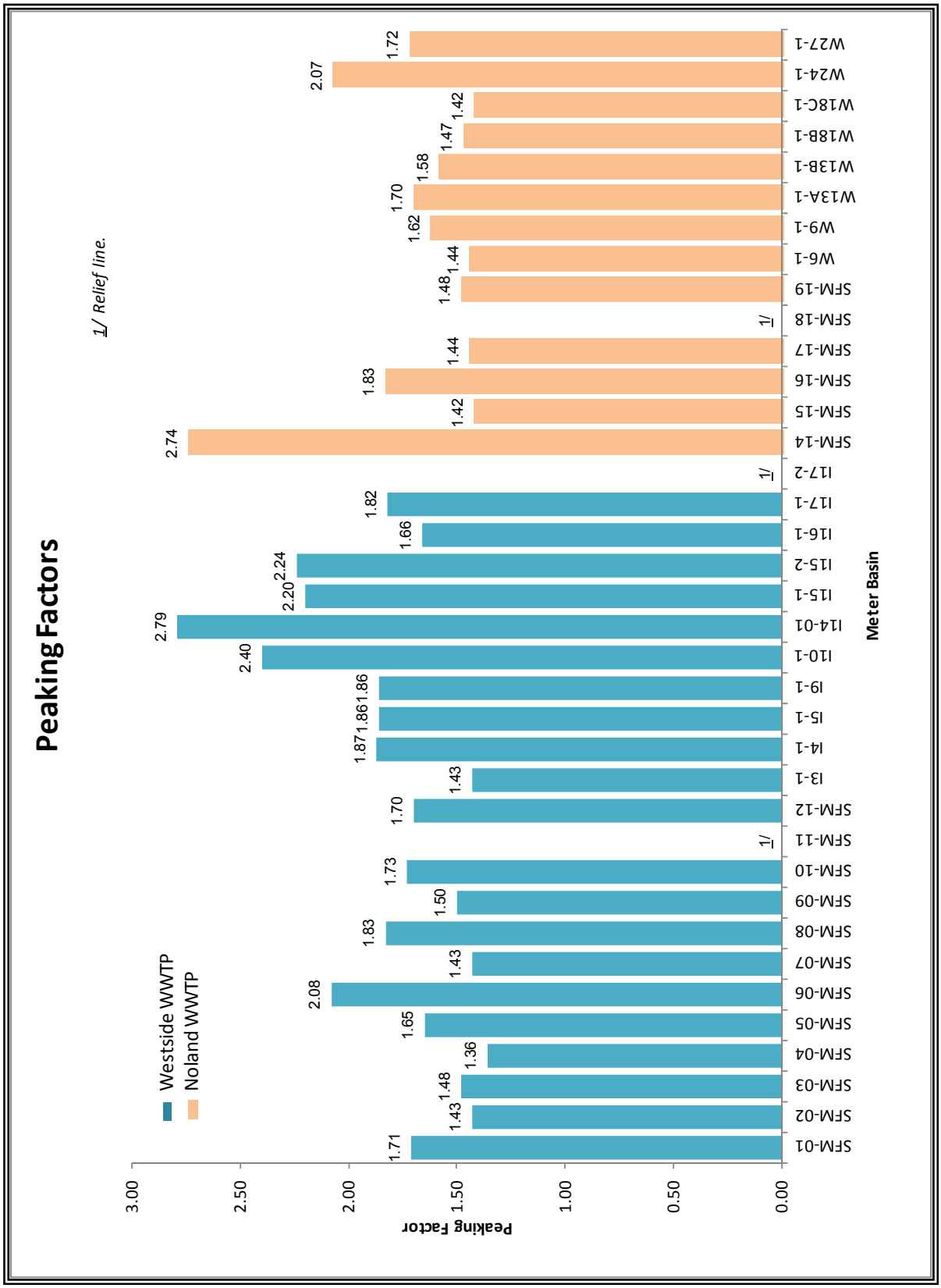
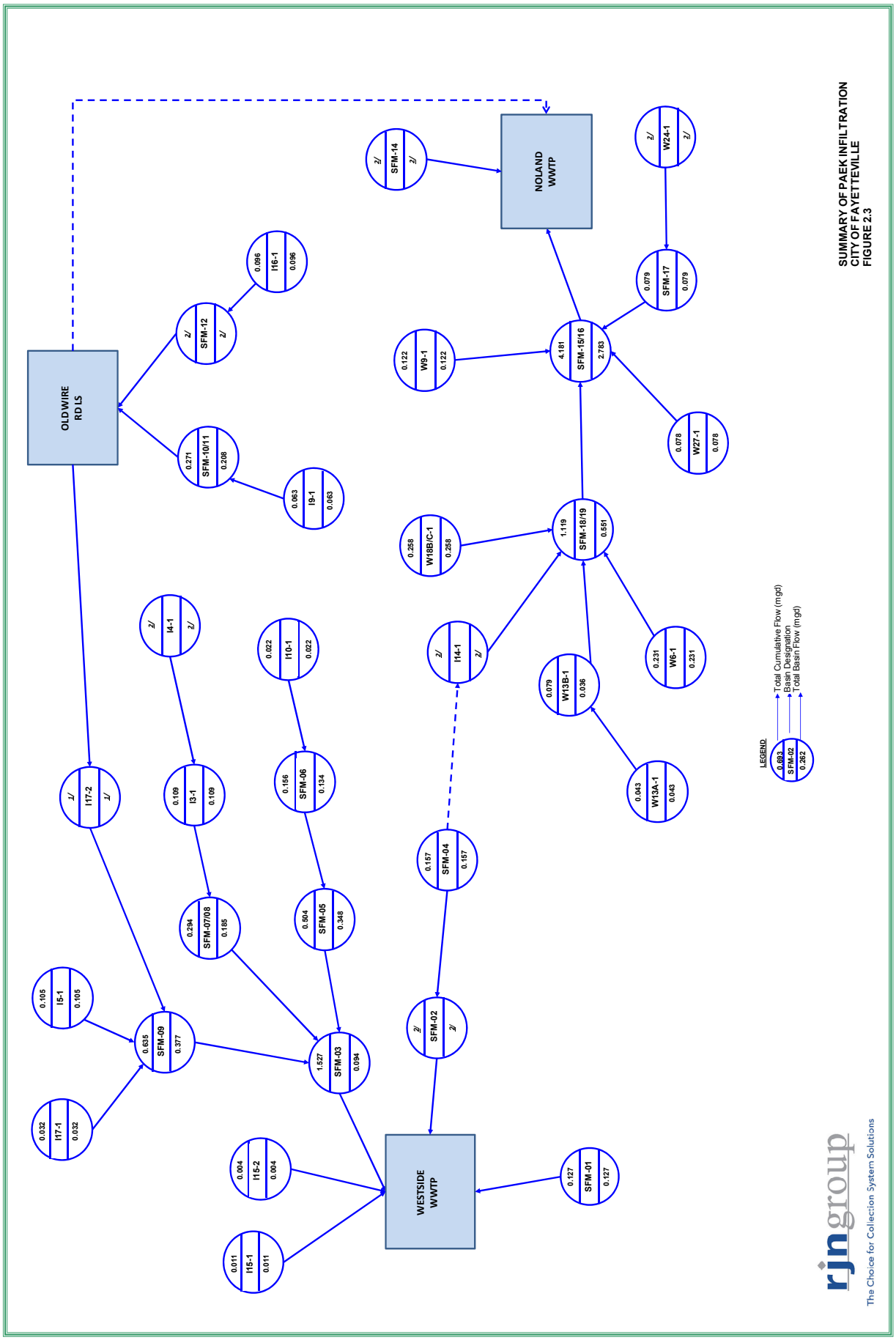

Peaking Factors
1/ Relief line.
Westside WWTP
Noland WWTP
Peaking Factor
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Meter Basin
SFM-01
SFM-02
SFM-03
SFM-04
SFM-05
SFM-06
SFM-07
SFM-08
SFM-09
SFM-10
SFM-11
SFM-12
I3-1
I4-1
I5-1
I9-1
I10-1
I14-01
I15-1
I15-2
I16-1
I17-1
I17-2
SFM-14
SFM-15
SFM-16
SFM-17
SFM-18
SFM-19
W6-1
W9-1
W13A-1
W13B-1
W18B-1
W18C-1
W24-1
W27-1
1.71
1.43
1.48
1.36
1.65
2.08
1.43
1.83
1.50
1.73
1.70
1.43
1.87
1.86
1.86
2.40
2.79
2.20
2.24
1.66
1.82
2.74
1.42
1.83
1.44
1.48
1.44
1.62
1.70
1.58
1.47
1.42
2.07
1.72

Table 2-E

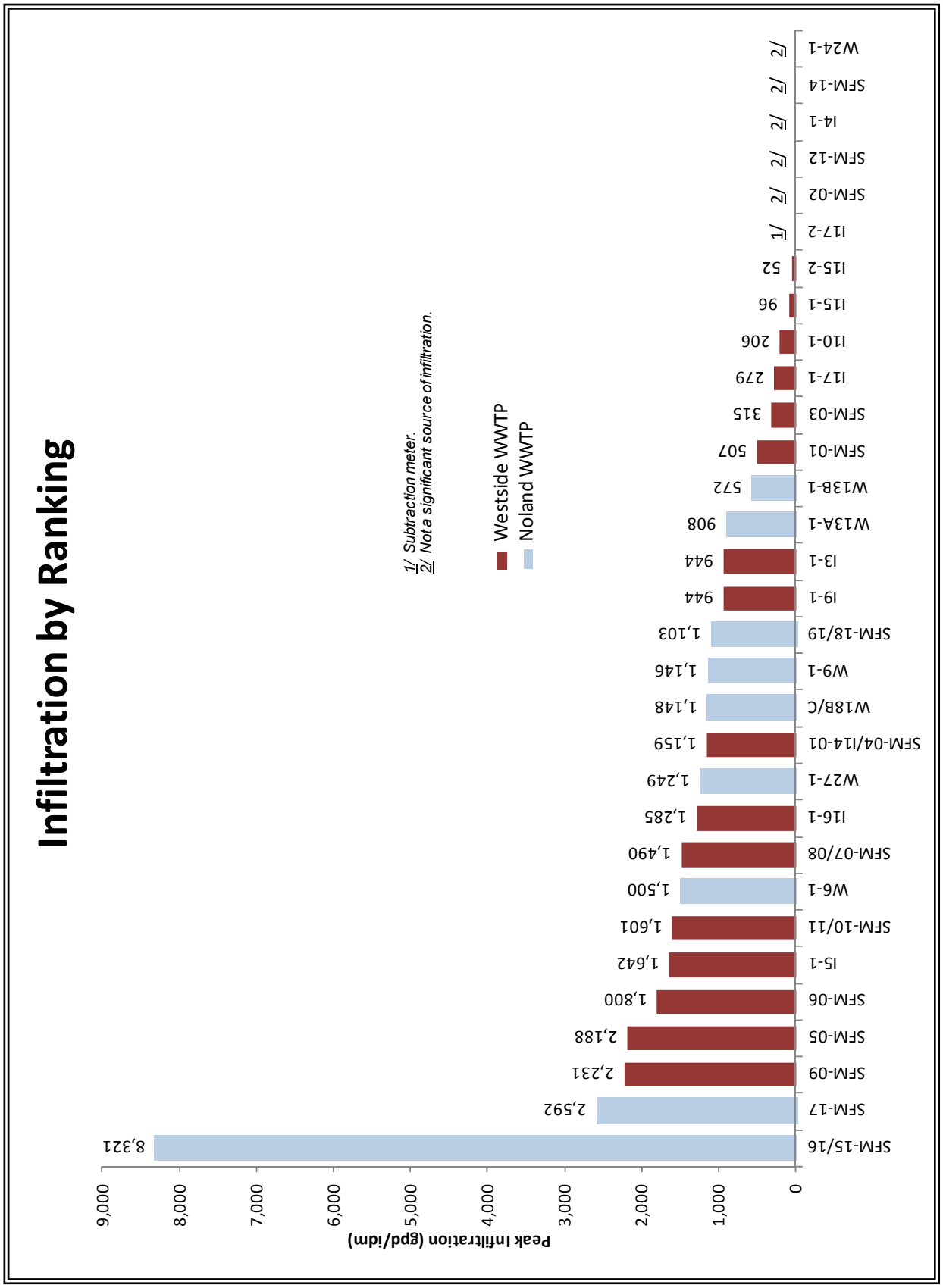
SUMMARY OF PEAK INFILTRATION

Meter Basin	Inch-Diameter-Mile (idm)	Cumulative Peak Monitored Infiltration (mgd)	Basin Peak Monitored Infiltration (mgd)	Basin Peak Unit Infiltration (gpd/idm)	Ranking
Westside WWTP					
SFM-01	249.95	0.127	0.127	507	20
SFM-02	319.11	2/	2/	2/	31
SFM-03	298.67	1.527	0.094	315	21
SFM-04/I14-01	135.21	0.157	0.157	1,159	12
SFM-05	159.01	0.504	0.348	2,188	4
SFM-06	74.71	0.156	0.134	1,800	5
SFM-07/08	124.15	0.294	0.185	1,490	9
SFM-09	168.99	0.635	0.377	2,231	3
SFM-10/11	129.93	0.271	0.208	1,601	7
SFM-12	127.06	2/	2/	2/	29
I3-1	115.60	0.109	0.109	944	17
I4-1	45.14	2/	2/	2/	32
I5-1	64.15	0.105	0.105	1,642	6
I9-1	66.83	0.063	0.063	944	16
I10-1	105.67	0.022	0.022	206	23
I15-1	111.71	0.011	0.011	96	24
I15-2	74.74	0.004	0.004	52	25
I16-1	74.58	0.096	0.096	1,285	10
I17-1	115.29	0.032	0.032	279	22
I17-2	1/	1/	1/	1/	1/
Subtotal	2,560.50		2.072	1,046 (average)	
Noland WWTP					
SFM-14	96.97	2/	2/	2/	26
SFM-15/16	334.45	4.181	2.783	8,321	1
SFM-17	30.31	0.079	0.079	2,592	2
SFM-18/19	499.75	1.119	0.551	1,103	15
W13A-1	47.50	0.043	0.043	908	18
W13B-1	62.88	0.079	0.036	572	19
W18B/C	224.50	0.258	0.258	1,148	13
W24-1	111.10	2/	2/	2/	30
W27-1	62.48	0.078	0.078	1,249	11
W6-1	154.10	0.231	0.231	1,500	8
W9-1	106.69	0.122	0.122	1,146	14
Subtotal	1,731.76		4.181	2,356	
Total	**4,291.26**		**6.253**	**1,553** (average)	

1/ Subtraction meter.
2/ Not a significant source of infiltration.



SUMMARY OF PAEK INFILTRATION
CITY OF FAYETTEVILLE
FIGURE 2.3


Infiltration by Ranking
Peak Infiltration (gpd/idm)
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
SFM-15/16
8,321
SFM-17
2,592
SFM-09
2,231
SFM-05
2,188
SFM-06
1,800
I5-1
1,642
SFM-10/11
1,601
W6-1
1,500
SFM-07/08
1,490
I16-1
1,285
W27-1
1,249
SFM-04/I14-01
1,159
W18B/C
1,148
W9-1
1,146
SFM-18/19
1,103
I9-1
944
I3-1
944
W13A-1
908
W13B-1
572
SFM-01
507
SFM-03
315
I17-1
279
I10-1
206
I15-1
96
I15-2
52
I17-2
1/
SFM-02
2/
SFM-12
2/
I4-1
2/
SFM-14
2/
W24-1
2/
Westside WWTP
Noland WWTP
1/ Subtraction meter.
2/ Not a significant source of infiltration.

INFLOW CONDITIONS

The Infoworks Hydraulic model developed during this study is also a model that allows for the projection of peak inflow rates for each individual basin. This method of inflow projections is generally more accurate in larger systems due to the fact that it is not impacted by the time of concentration or time lag of peak flow relative to one basin to another. Therefore, the model generated inflow rates were used to project inflow rates to a design storm event.

The analysis projected the peak inflow rate of 84.608 mgd for the design storm (5-year/24-hour storm with a 2-year/60-minute event embedded). A summary of the projected peak wet-weather flow rates is given in Table 2-F and shown graphically on page 2-20. The basin unit inflow rate expresses the magnitude of peak inflow relative to other basins. A basin flow diagram giving inflow rates is shown on Figure 2-4.

PEAK FLOW RATES

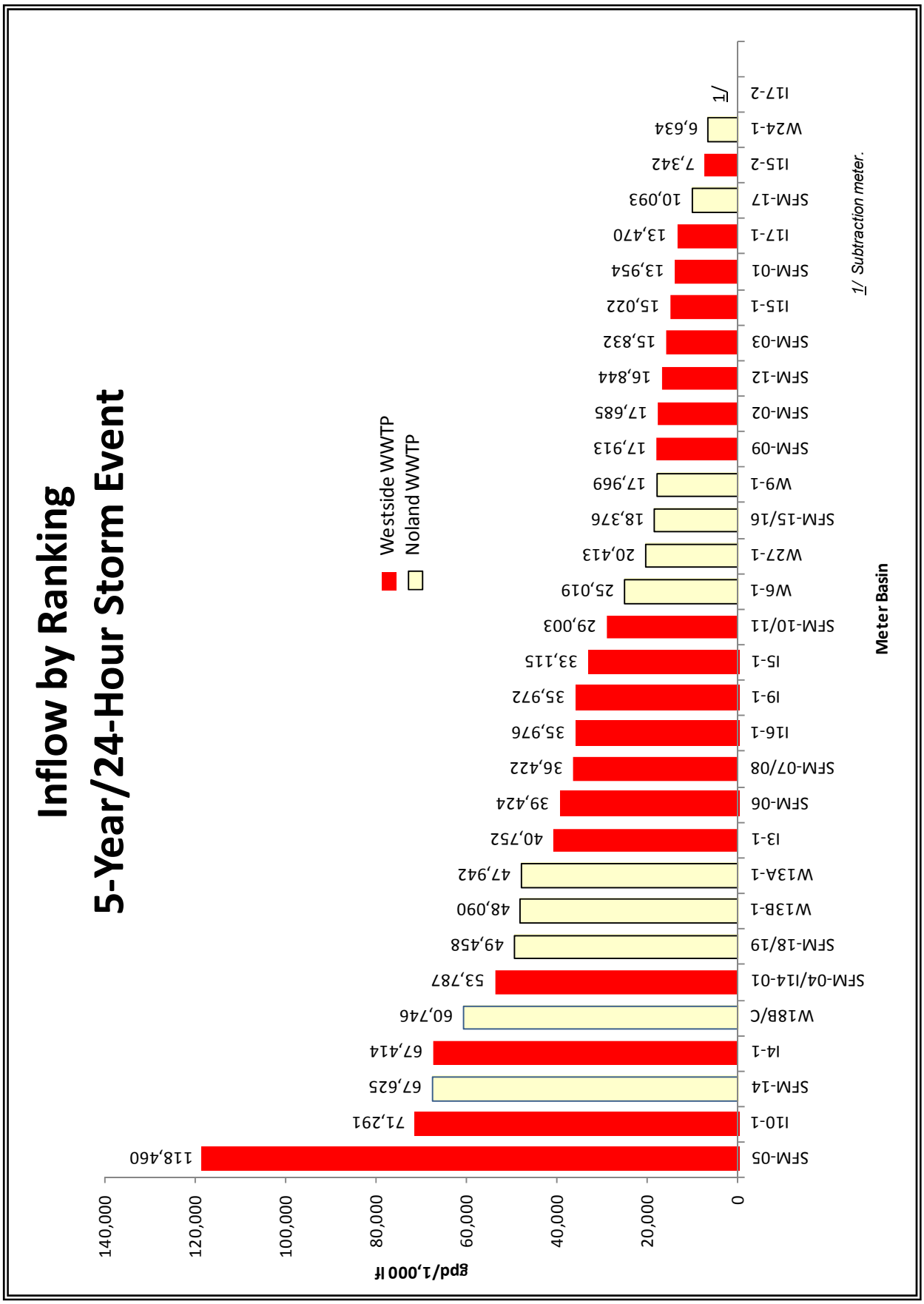
The total peak hour wet-weather flow projected during the design storm is approximately 120.75 mgd. This consists of 24.89 mgd of peak hourly dry-weather flow, 6.25 mgd of peak infiltration, and 84.61 mgd of inflow. Based on an average daily dry-weather flow of 8.25 mgd, this would result in a wet-weather peaking factor average of 9.63. Table 2-G summarizes the flows. Peaking factors varied from 1.77 to 27.84.

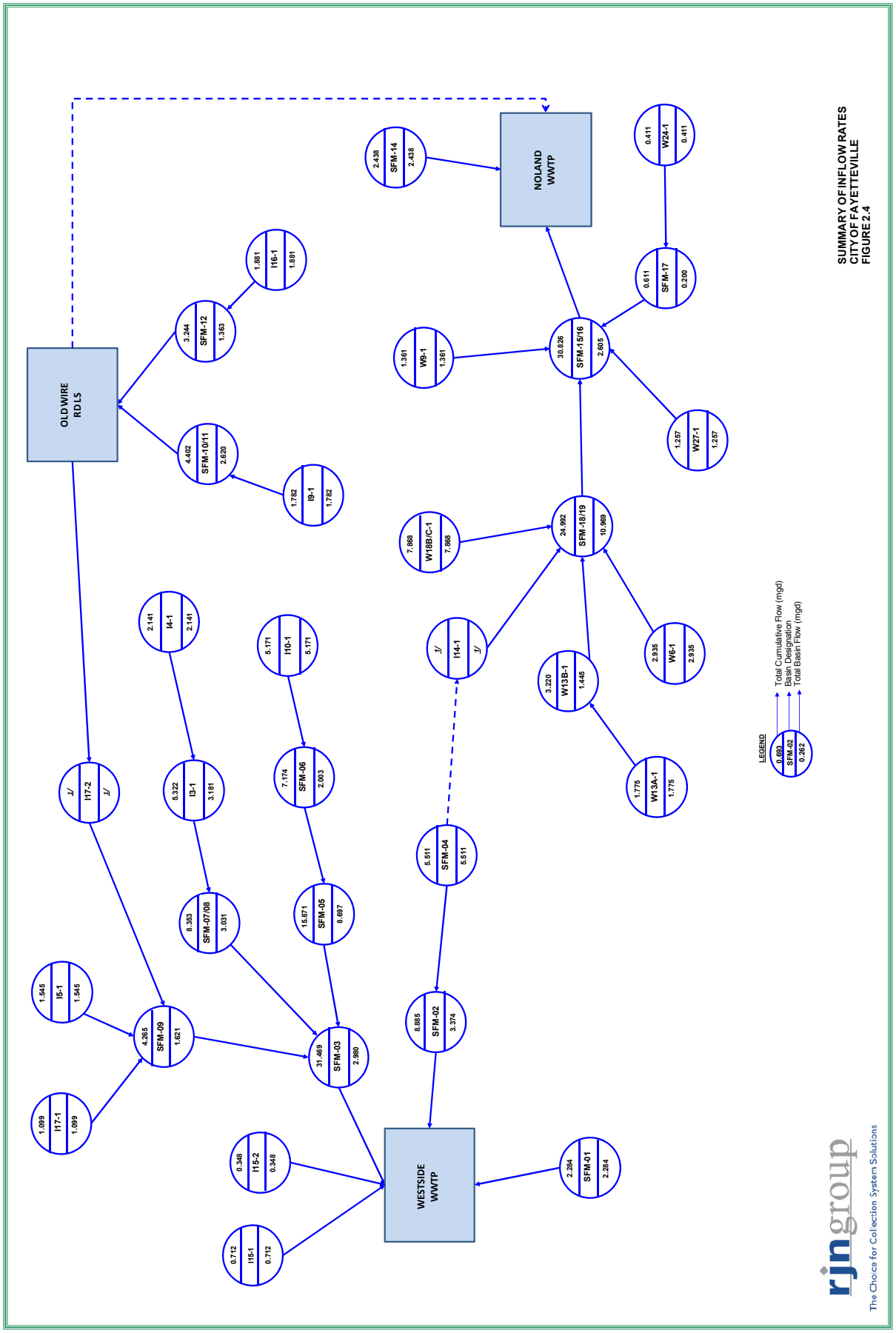
Table 2-F

SUMMARY OF PEAK HOUR INFLOW RATES

Meter Basin	Basin Size (lf)	Basin Peak 5-Year/24-Hour Inflow (mgd)	Basin Unit Inflow Rate (gpd/1,000 lf)	Basin Unit Inflow Rate 5-Year/24-Hour Ranking (gpd/1,000 lf)
Westside WWTP				
SFM-01	163,682	2.284	13,954	26
SFM-02	190,784	3.374	17,685	22
SFM-03	188,226	2.980	15,832	24
SFM-04/I14-01	102,459	5.511	53,787	6
SFM-05	73,417	8.697	118,460	1
SFM-06	50,807	2.003	39,424	11
SFM-07/08	83,219	3.031	36,422	12
SFM-09	90,492	1.621	17,913	21
SFM-10/11	90,335	2.620	29,003	16
SFM-12	80,920	1.363	16,844	23
I3-1	78,057	3.181	40,752	10
I4-1	31,759	2.141	67,414	4
I5-1	46,655	1.545	33,115	15
I9-1	49,539	1.782	35,972	14
I10-1	72,534	5.171	71,291	2
I15-1	47,398	0.712	15,022	25
I15-2	47,398	0.348	7,342	29
I16-1	52,285	1.881	35,976	13
I17-1	81,586	1.099	13,470	27
I17-2	1/	1/	1/	1/
Subtotal	1,621,552	51.344	35,773 (average)	
Noland WWTP				
SFM-14	36,052	2.438	67,625	3
SFM-15/16	141,764	2.605	18,376	19
SFM-17	19,816	0.200	10,093	28
SFM-18/19	221,783	10.969	49,458	7
W13A-1	37,024	1.775	47,942	9
W13B-1	30,048	1.445	48,090	8
W18B/C	129,523	7.868	60,746	5
W24-1	61,956	0.411	6,634	30
W27-1	61,579	1.257	20,413	18
W6-1	117,309	2.935	25,019	17
W9-1	75,741	1.361	17,969	20
Subtotal	932,595	33.264	33,851	
Total	**2,554,147**	**84.608**	**34,812** (average)	

1/ Subtraction meter.


Inflow by Ranking
5-Year/24-Hour Storm Event
Westside WWTP
Noland WWTP
gpd/1,000 lf
140,000
120,000
100,000
80,000
60,000
40,000
20,000
0
SFM-05 118,460
I10-1 71,291
SFM-14 67,625
I4-1 67,414
W18B/C 60,746
SFM-04/I14-01 53,787
SFM-18/19 49,458
W13B-1 48,090
W13A-1 47,942
I3-1 40,752
SFM-06 39,424
SFM-07/08 36,422
I16-1 35,976
I9-1 35,972
I5-1 33,115
SFM-10/11 29,003
W6-1 25,019
W27-1 20,413
SFM-15/16 18,376
W9-1 17,969
SFM-09 17,913
SFM-02 17,685
SFM-12 16,844
SFM-03 15,832
I15-1 15,022
SFM-01 13,954
I17-1 13,470
SFM-17 10,093
I15-2 7,342
W24-1 6,634
I17-2 1/
Meter Basin
1/ Subtraction meter.



SUMMARY OF INFLOW RATES
CITY OF FAYETTEVILLE
FIGURE 2.4

Table 2-G

TOTAL PEAK HOUR 5-YEAR/24-HOUR WET-WEATHER FLOW

Meter Basin	Basin Average Daily Dry-Weather Flow (mgd)	Peak Hourly Dry-Weather Flow Rate (mgd)	Peak Monitored Infiltration (mgd)	Peak 5-Year/24-Hour Inflow (mgd)	Peak Wet-Weather Flow (mgd)	Wet-Weather Peaking Factor
Westside WWTP						
SFM-01	0.321	0.549	0.127	2.284	2.960	9.21
SFM-02	0.262	0.993	1/	3.374	4.367	16.65
SFM-03	0.122	3.788	0.094	2.980	6.863	56.25
SFM-04/I14-01	0.381	0.656	0.157	5.511	6.324	16.6
SFM-05	0.428	1.622	0.348	8.697	10.667	24.94
SFM-06	0.307	1.158	0.134	2.003	3.295	10.75
SFM-07/08	0.380	1.170	0.185	3.031	4.386	11.55
SFM-09	0.173	0.973	0.377	1.621	2.971	17.17
SFM-10/11	0.165	0.534	0.208	2.620	3.362	20.43
SFM-12	0.187	0.486	1/	1.363	1.849	9.89
I3-1	0.338	0.622	0.109	3.181	3.912	11.58
I4-1	0.096	0.178	1/	2.141	2.319	24.28
I5-1	0.154	0.286	0.105	1.545	1.936	12.57
I9-1	0.144	0.267	0.063	1.782	2.112	14.71
I10-1	0.249	0.598	0.022	5.171	5.791	23.27
I15-1	0.049	0.338	0.011	0.712	1.061	21.52
I15-2	0.104	0.234	0.004	0.348	0.586	5.61
I16-1	0.099	0.164	0.096	1.881	2.141	21.69
I17-1	0.321	0.583	0.032	1.099	1.714	18.734
Subtotal	4.278	15.200	2.072	51.344	68.616	17.58 (average)
Noland WWTP						
SFM-14	0.207	0.569	2/	2.438	3.007	14.50
SFM-15/16	0.355	5.483	2.783	2.605	10.871	30.65
SFM-17	0.136	0.304	0.079	0.200	0.583	4.29
SFM-18/19	1.107	4.188	0.551	10.969	15.709	14.19
W6-1	0.414	0.597	0.043	1.775	2.415	5.83
W9-1	0.142	0.231	0.036	1.445	1.712	12.02
W13A-1	0.460	0.783	0.258	7.868	8.909	19.38
W13B-1	0.182	1.012	1/	0.411	1.423	7.80
W18B-1/W18C-1	0.623	0.903	0.078	1.257	2.238	3.59
W24-1	0.075	0.156	0.231	2.935	3.322	44.07
W27-1	0.269	0.462	0.122	1.361	1.945	7.24
Subtotal	3.970	14.687	4.181	33.264	52.132	14.87
Total	**8.249**	**29.887**	**6.253**	**84.608**	**120.748**	**16.23** (average)

1/ Not a significant source of infiltration.

CHAPTER 3

POPULATION AND FLOW PROJECTIONS

This chapter presents the existing and future population development and how it was used in the hydraulic model. It also provides a breakdown of the methodology used to project the flows under the 2030 and ultimate population scenarios.

EXISTING POPULATION

To facilitate model flow loading and calibration, existing population and flows were determined and analyzed.

RESIDENTIAL POPULATION

Population data was obtained from the U.S. Census Bureau for Fayetteville by census block. Using this data, population was assigned to residential households using the residential customer billing points provided by the City. The total population in the Fayetteville model is 78,618. A summary of the population by flow metering basin is shown in Table 3-A.

COMMERCIAL/INSTITUTIONAL/INDUSTRIAL

Commercial, institutional, and industrial flows were modeled using the customer water billing data provided by the City. The commercial and industrials sewage flows were calculated based on factored water usage for each customer. The City provided trade waste details for the institutional and industrial customers who discharge a significant volume to the sewer. These users were located and their discharge explicitly modeled.

A summary of modeled commercial/institutional/industrial flows is shown in Table 3-B.

Table 3-A

FAYETTEVILLE RESIDENTIAL POPULATION BY METER BASIN

Meter Basin	Average Population Per Household	Total Population
I10-1	2.1	1,988
I15-1	2.4	1,376
I15-2	2.5	1,289
I16-1	2.3	1,458
I17-1	2.2	3,063
I3-1	1.9	2,516
I4-1	2.1	428
I5-1	2.3	1,513
I9-1	2.2	1,385
SFM-01	2.4	4,986
SFM-02	2.3	8,188
SFM-03	2.4	3,802
SFM-04	2.1	4,191
SFM-05	1.8	5,164
SFM-06	4.2	2,974
SFM-07/08	2.1	1,580
SFM-09	2.1	1,692
SFM-10/11	2.4	2,727
SFM-12	2.8	2,065
SFM-14	3.0	1,570
SFM-15/16	1.9	1,545
SFM-17	2.5	481
SFM-18/19	1.9	7,343
W13A-1	1.9	954
W13B-1	2.0	4,440
W18B/C	1.9	4,121
W24-1	2.5	1,135
W27-1	2.7	662
W6-1	3.1	1,603
W9-1	2.4	1,662
Westside	2.6	717
Total		**78,618**

Table 3-B

SUMMARY OF COMMERCIAL/INSTITUTIONAL/INDUSTRIAL LAND USE

Meter Basin	Commercial/Institutional/Industrial Flow (mgd)
I10-1	0.1184
I15-1	0.0088
I15-2	0.0072
I16-1	0.0029
I17-1	0.0301
I3-1	0.0601
I4-1	0.0306
I5-1	0.0164
I9-1	0.0011
SFM-01	0.0546
SFM-02	0.0303
SFM-03	0.1221
SFM-04	0.0151
SFM-05	0.0128
SFM-06	0.0232
SFM-07/08	0.1452
SFM-09	0.0529
SFM-10/11	0.0014
SFM-12	0.0330
SFM-14	0.0024
SFM-15/16	0.0378
SFM-17	0.0000
SFM-18/19	0.6534
W13A-1	0.0098
W13B-1	0.0494
W18B/C	0.1767
W24-1	0.0000
W27-1	0.3078
W6-1	0.2710
W9-1	0.0019
Westside	0.0000
Total	**2.2763**

RESIDENTIAL FLOW

Using the flow data collected from each meter during a dry period in January 2013, average weekday and weekend hydrographs were calculated and graphed for each flow meter with a primarily residential catchment. Weekday and weekend diurnal profiles of usage were developed by removing permanent groundwater infiltration. Profiles were then grouped based on profile shape and averaged across basins to determine three representative units-less residential patterns. These profiles were input into the model and used to modulate dry-weather flows in the hydraulic model. The weekday profiles used in the model are shown in Figure 3.1.

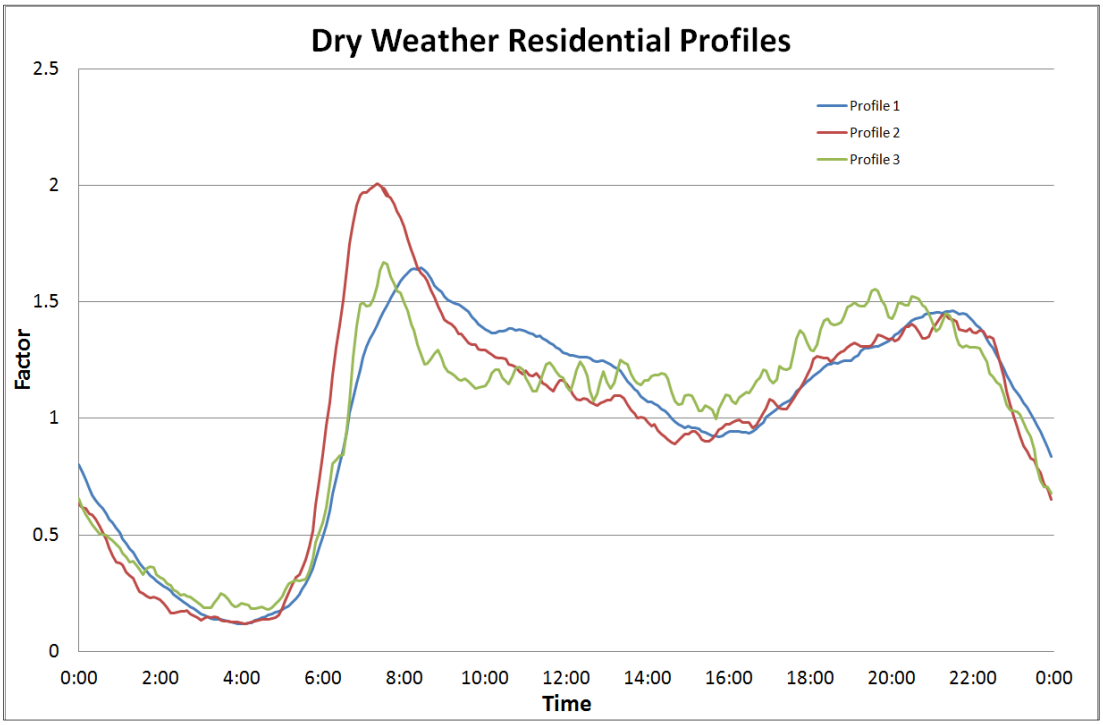


Figure 3.1

COMMERCIAL, INSTITUTIONAL, AND INDUSTRIAL FLOW

A set of RJN standard non residential flow profiles were assigned to commercial/institutional/industrial subcatchments based on the predominant type of business, institution or industry in each subcatchment. Two unique flow patterns were identified. The first for institutional developments in Basin I10-1. The second profile was developed for Superior Industries located on Borick Drive within Basin W27-1. These flow profiles were developed from the recorded flow data and used at these two locations. Figure 3.2 shows the commercial flow profiles that were used in the model.

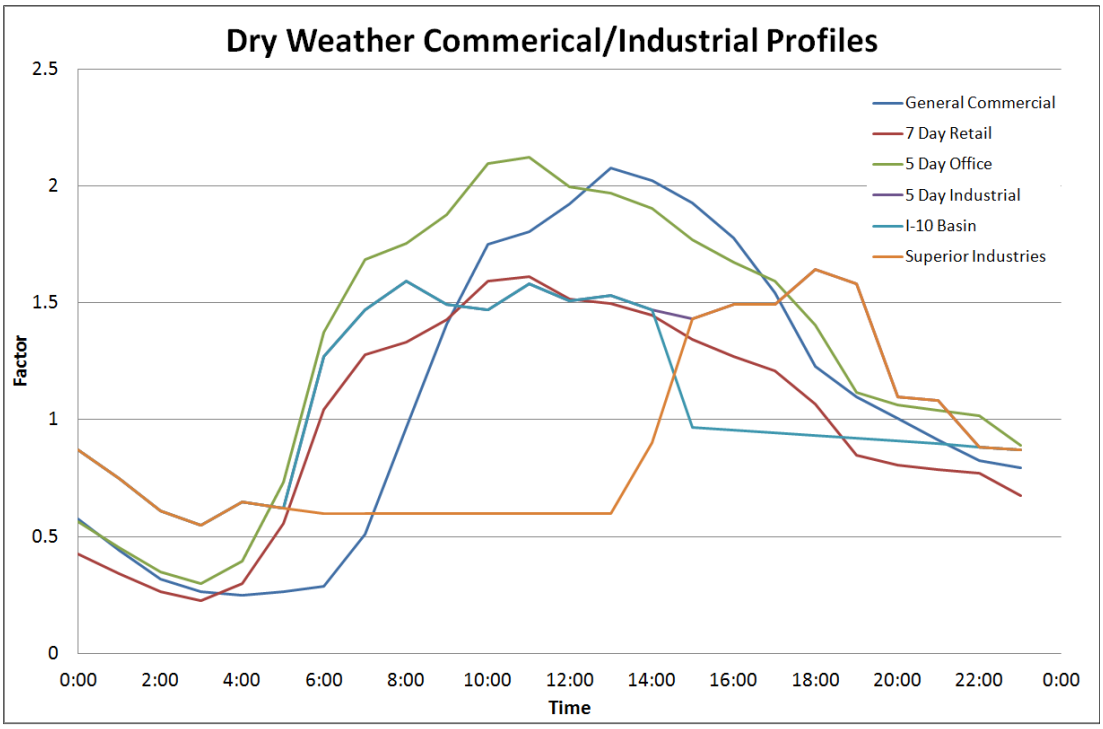


Figure 3.2

Once diurnal patterns were determined, analysis of the dry weather flow began. Per capita flow rates were calculated from the average daily dry weather flow for each meter basin for residential areas. Permanent groundwater infiltration levels were also estimated. A summary of dry weather flow by meter basin is shown in Table 3-C.

POPULATION PROJECTIONS

To meet the objectives of the Master Plan Update , the hydraulic model was updated to include future growth projections. The two planning horizons which were modeled are 2030 and Ultimate Buildout.

2030 GROWTH PROJECTION

The 2030 population is forecast to increase to 105,061 in line with the population figures used in the Water Master Plan. The City had identified growth areas and provided values for the distribution of that growth. The locations and distribution of the 2030 growth are presented in Figure 3.3 on page 3-7.

Table 3-C

SUMMARY OF DRY-WEATHER FLOW

Meter Basin	Population	Per Capita Flow (g)	Residential Flow (mg)	Commercial/ Institutional/ Industrial Flow (mg)	Permanent Groundwater Infiltration (mg)	Total Daily Dry Weather Flow (mg)
I10-1	1,988	50	0.099	0.118	0.040	0.258
I15-1	1,376	50	0.069	0.009	0.013	0.090
I15-2	1,289	60	0.077	0.007	0.013	0.097
I16-1	1,458	50	0.073	0.003	0.031	0.107
I17-1	3,063	70	0.214	0.030	0.115	0.359
I3-1	2,516	60	0.151	0.060	0.108	0.319
I4-1	428	60	0.026	0.031	0.091	0.147
I5-1	1,513	60	0.091	0.016	0.100	0.207
I9-1	1,385	50	0.069	0.001	0.091	0.161
SFM-01	4,986	60	0.299	0.055	0.062	0.416
SFM-02	8,188	50	0.409	0.030	0.080	0.519
SFM-03	3,802	60	0.228	0.122	0.017	0.367
SFM-04	4,191	70	0.293	0.015	0.190	0.499
SFM-05	5,164	50	0.258	0.013	0.218	0.489
SFM-06	2,974	50	0.149	0.023	0.107	0.279
SFM-07/08	1,580	60	0.095	0.145	0.118	0.358
SFM-09	1,692	50	0.085	0.053	0.005	0.142
SFM-10/11	2,727	60	0.164	0.001	0.098	0.263
SFM-12	2,065	60	0.124	0.033	0.023	0.179
SFM-14	1,570	60	0.094	0.002	0.012	0.109
SFM-15/16	1,545	50	0.077	0.038	0.086	0.201
SFM-17	481	60	0.029	0.000	0.004	0.033
SFM-18/19	7,343	50	0.367	0.653	0.029	1.049
W13A-1	954	70	0.067	0.010	0.062	0.139
W13B-1	4,440	60	0.266	0.049	0.034	0.350
W18B/C	4,121	70	0.288	0.177	0.311	0.776
W24-1	1,135	70	0.079	0.000	0.014	0.093
W27-1	662	60	0.040	0.308	0.116	0.463
W6-1	1,603	50	0.080	0.271	0.132	0.483
W9-1	1,662	70	0.116	0.002	0.056	0.174
Westside	717	50	0.036	0.000	0.006	0.042
Total	**78,618**		**4.514**	**2.276**	**2.382**	**9.172**

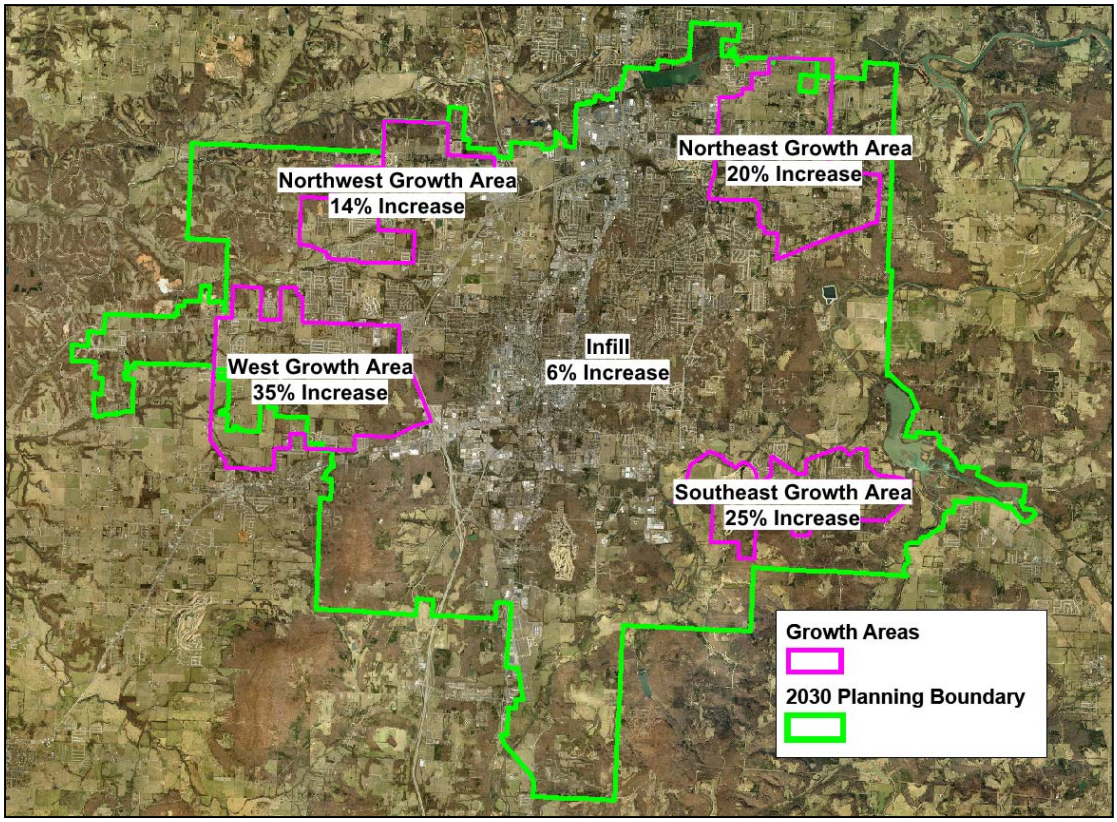


Figure 3.3: 2030 Growth Areas

The increase in population to be distributed across the model was calculated as follows:

- Pop Increase = 105,061 – 78,618 = 26,443.

The population increase of 26,443 was then distributed between the growth areas based on the distribution percentages provided by the City as detailed in Table 3-D.

Table 3-D

DEVELOPMENT DENSITIES

Growth Area	Distribution (%)	Population
Infill	6	1,587
West	35	9,255
North West	14	3,702
North East	20	5,289
South East	25	6,611
Total	**100**	**26,443**

Applying the 2030 population increase to the model was undertaken in two stages. The first task was to add population in the vacant residential lots within each growth area. Then using the land use data provided by the City, undeveloped land within each growth area was updated into the model to match the required populations.

The development densities applied to the each land use class were provided by the City and are detailed in Table 3-E. An average household size of 2.17 persons/unit was applied to the densities to calculate a population. The population per household ratio was taken from the Census 2010.

Table 3-E

DEVELOPMENT DENSITIES

Land Use Class	Units/Acre
Natural Areas	4
Rural Residential Area	4
Residential Neighborhood Area	4
City Neighborhood Area	4
Complete Neighborhood Plan	4
Hillside/Hilltop Overlay	1
Flood Plain	0

ULTIMATE BUILDOUT GROWTH PROJECTION

The second planning horizon modeled was based on ultimate buildout of the City's planning area. As no population or commercial projections were available for ultimate buildout, projections were made using spatial analysis in GIS. The City Plan showing the future land use class is displayed in Figure 3.4 below.

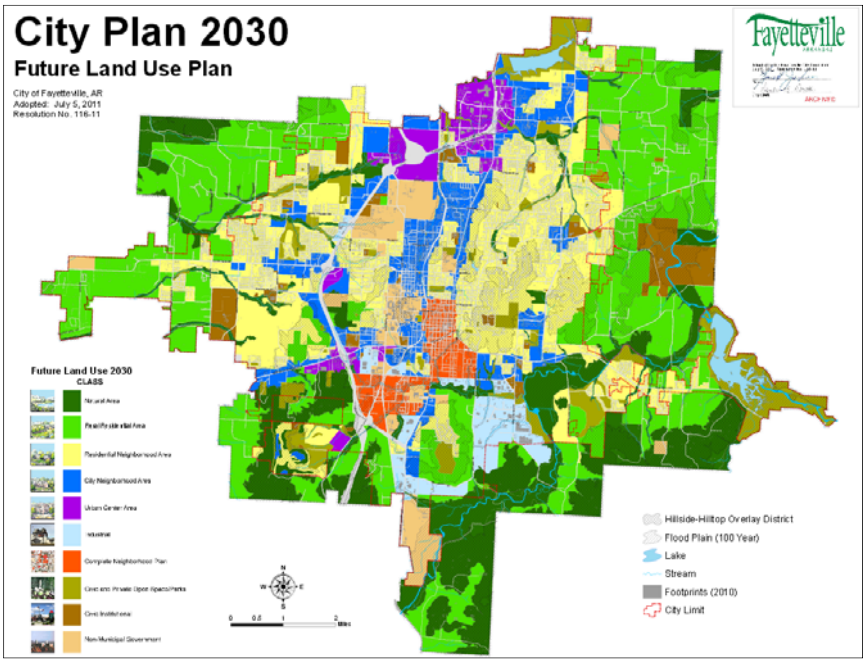


Figure 3.4: City Plan

The development densities applied to each land use class were the same as those detailed in Table 3-E. An average household size of 2.17 persons/unit was applied to the densities to calculate a population. The calculated ultimate buildout population was 304,085.

The ultimate build out population was applied to the model by creating a set of future drainage basins covering the currently undeveloped areas on the periphery of the City. The future basins were overlaid on the land use data and populations updated into the basins spatially. A map showing existing and future basins along with existing development is shown in Exhibit 2.

FLOW PROJECTIONS

DRY-WEATHER FLOWS

A flow rate of 60 gallons per capita per day was used for the future growth areas. This was the average usage rate determined during model calibration. This rate is exclusive of permanent groundwater infiltration and represents only the sewer usage of individual customers.

WET-WEATHER FLOWS

Following the dry-weather flow projections, the subcatchments were programmed to generate wet weather flows. During wet-weather calibration, the system response to RDII was identified and the hydraulic model was calibrated to re-create the observed response within each flow meter basin. The model utilizes a series of coefficients and area distributions to generate wet weather flows as described in Chapter 4. By using this methodology, the coefficients can be applied to other subcatchments and their wet-weather response will scale with the subcatchment size.

It is anticipated that rainfall derived infiltration/inflow (RDII) rates would be low for future growth areas, assuming good construction practices in new sanitary sewers. The model calibration revealed that existing basin I15-2 had a low RDII rate resulting in only a small response to rainfall. The runoff parameters from existing basin I15-2 were applied to the future growth basins.

This method provides a more accurate model as the rainfall response more closely matches how the system is behaving rather than using a fixed wet weather peaking factor.

CHAPTER 4

MODEL DEVELOPMENT

This Chapter provides a summary of the model development, calibration, and capacity analysis.

MODEL DESCRIPTION

In order to analyze the performance of the Fayetteville sanitary sewer system, the existing hydraulic model was updated and expanded. The InfoWorks CS software by Innovyze was utilized for the model update. Infoworks CS is a fully dynamic hydraulic model capable of analyzing large, complex sewer systems.

The objectives of this task were to:

- Update the 1996 hydraulic model including all sewer lines and auxiliary facilities.
- Calibrate the model to reflect current recorded flows and surcharge depth data.
- Create future growth models for both the 2030 and ultimate development projections.
- Simulate a design storm on the existing, 2030 and ultimate development models to identify areas with insufficient capacity and overflows.
- Develop a staged system implementation strategy to eliminate wet-weather sanitary sewer overflows (SSO's).
- Evaluate alternatives to optimize system performance.

MODEL UPDATE

COLLECTION SYSTEM

The existing hydraulic model of Fayetteville was constructed by RJN in 1996, from record drawings and available Geographical Information System (GIS) data, using the XP SWMM software. The model consisted of sewer mains 10 inches and larger in diameter and represented approximately 10% of the entire Fayetteville sanitary sewer system.

As part of the scope of this project, RJN was to incorporate all new sewer mains 10 inches and larger in diameter constructed since 1996, plus all mains smaller than 10 inches. The final model being an all pipe model, which would include every sewer main in the Fayetteville system. The additional sewer mains provide routing and flow loading for the system and eliminate the need for complicated storage compensation and time of concentration assumptions.

Due to the large amount of network changes from 1996 to the present, it was determined that the best course of action would be to construct a new model based upon the current GIS database, augmented with pipe attribute data from the 1996 model as appropriate. This approach provided more accurate system geometry than incorporating new mains into the 1996 model.

Where manhole elevations were missing in the GIS these manholes were interpolated from a 3-D terrain model built from 2' contours provided by the City. Invert elevations were then calculated to maintain continuity and slopes based on engineering judgment. The model was also updated with data from a both record drawings and field inspections.

The updated model reflecting the 2013 system was then checked for proper slopes and connectivity using the validation tools built into InfoWorks CS. Line segments found to be disconnected or that contained negative or questionable slopes were manually adjusted based on field verifications or engineer's judgment.

All data in the hydraulic model network was color coded / flagged to define its data source. More accurate information such as record drawings or survey data was used in preference to GIS or interpolated data.

The hydraulic model contains 2,585,576 linear feet of gravity mains, and 122,281 linear feet of force mains. Table 4-A contains the pipeline characteristics for the hydraulic model.

Table 4-A

PIPELINE CHARACTERISTICS

Force Mains		Gravity Sewers	
Pipe Diameter (in)	Length (lf)	Pipe Diameter (in)	Length (lf)
3	9,979	4	3,065
4	8,206	5	45,326
6	11,306	6	698,790
8	10,052	7	28,010
10	8,093	8	1,300,232
12	14,226	9	1821
14	1,116	10	69,807
16	13,962	11	12,805
18	8,496	12	119,586
20	10,037	14	11,709
24	11,503	15	19,035
30	15,305	16	32,618
Total Force Mains	**122,281**	17	316
		18	54,490
		20	1,781
		21	10,684
		24	52,099
		27	3,340
		30	19,746
		33	5,715
		36	31,547
		40	190
		42	36,603
		48	26,262
		Total Gravity Sewers	**2,585,576**

PUMPING STATIONS

The updated hydraulic model includes all of the major pumping stations located within the sanitary sewer system. The pump stations included in the model are listed below.

Airport East	Hamestring	Silverthorne
Airport North	Heritage II	Skyler Place
Airport South	Industrial Park	Stonebridge Meadows II
Bohannon	Legacy Pointe V	Stonebridge Meadows V
Broyles	Lowes	Stonebridge Meadows
Copper Creek II	Lowes-Zion	Stonebridge
Crescent Lake	Mally Wagnon	Stonewood
Crofton Manner	Masters	Timber Lake Office Park
Embry Acres	McDonald	Timbertrails
Farmington West	Office Park	Willow West
Futrall Drive	Old Wire	Timbercrest
Greenland	Owl Creek	
Gregg	Porter	

Pump station geometry was entered into the model from record drawings, provided by the City of Fayetteville. If records drawings were not available then pump station dimensions were estimated. Where pump curves were available these were entered in the model. Pump capacities were provided by the City for the remaining pump stations. Pump control levels were obtained from operations manuals provided by the City.

WASTEWATER TREATMENT FACILITIES

The hydraulic model terminates at the inlet of the wastewater treatment plants. To ensure the outfall conditions were correctly represented, a limiting discharge orifice has been modeled to match the inlet capacity of the plants. The inlet capacities have been modeled as 32 MGD and 42 MGD at Noland and West Side treatment plants respectively.

SUBCATCHMENTS

Subcatchments are geometric sub-areas within each drainage basin. These modeled areas contain all the parameters for loading flow into the sewer network including population, non-residential flow and rainfall derived inflow/infiltration (RDII) runoff parameters. Subcatchments were imported into the model from existing parcel data from City GIS. By loading flow from individual parcels, more detailed flow analysis can be performed. This also eliminates the need for time of concentration assumptions.

The subcatchments encompasses all areas that contribute flow into the sewer system. Areas such as parks, golf courses, cemeteries, etc. that are not connected to the sewer system were not imported from the parcel data as they do not contribute any wastewater flow or RDII. The hydraulic model contains 25,412 subcatchments with an average size of 0.42 acres.

EXISTING POPULATION PROCESSING

Population data was sourced from the 2010 US census and projected to match current population estimates. Households were located within the sanitary sewer system using the residential customer billing data (points) from the City GIS. Populations were assigned to individual households (customer billing points) within each census block. The populations from the customer billing points were then totaled within each modeled subcatchment. The total population in the Fayetteville model is 78,618.

COMMERCIAL/INSTITUTIONAL/INDUSTRIAL LOADING

Commercial, institutional, and industrial flows were loaded into the model using the customer water billing data provided by the City. The commercial, institutional, and industrial flows were calculated applying a return to sewer factor to the water usage data.

MODEL TESTING

A series of validation tests was undertaken on the model to confirm logical network connectivity as well as consistent vertical alignment. A standard residential hydrograph was applied to each subcatchment in the model and validation simulation undertaken. After a few minor alterations the model was able to run a 24-hour dry-weather simulation without any issues and was considered to be a stable platform for calibration to proceed.

DRY-WEATHER CALIBRATION

PROCESS

Model calibration is necessary for the model to accurately represent the behavior of the sanitary sewer system. Calibration is a process through which model variables and coefficients are adjusted through multiple iterations until flow, depth, and velocity matches actual flow meter data recorded during events. The model is calibrated to recreate sewer performance in both dry-weather and wet-weather conditions.

DRY-WEATHER PERIOD

Dry-weather calibration ideally requires at least a 7-day period unaffected by rainfall derived flows. This period must also include at least one weekend. The recorded flow data was assessed in conjunction with the rainfall data and from this comparison, the span from January 18, 2013 through January 25, 2013 was selected as a typical dry-weather period for the area for dry-weather calibration.

CALIBRATION

Although wet-weather flows are generally greater than dry-weather flows, it is sound modeling practice to have a reasonably accurate representation of these flows in the model. Calibrating the model for dry-weather flow was achieved by modifying:

- Permanent ground water infiltration rates
- Per capita flow rates
- Commercial/institutional/industrial flow rates.

The calibration is considered successful when minimum flow, peak flow and total volume at all meter sites matches recorded data within five (5) percent.

The final dry-weather flow data summary in the model is as follows:

- Contributing area: 9,086 Acres
- Average per capita flow: 60 gal/day
- Total Daily Residential wastewater flow: 4.7 MG

- Total Daily Commercial/Institutional/Industrial flow: 2.3 MG
- Total Daily Permanent groundwater (dry-weather) infiltration: 2.4 MG
- Total Daily Dry-Weather flow: 9.4 MG

Final input values and comparison hydrographs for the dry-weather period can be found in the Appendix.

WET-WEATHER CALIBRATION

ANTECEDENT CONDITIONS

Prior to the monitoring period, there was below average rainfall across the region. To replicate the antecedent conditions prior to the flow survey, historical rainfall data was obtained and loaded into the model. In addition, evaporation data was provided by the Southern Regional Climate Center monitoring site located at the Drake Field Airport, Fayetteville. Both sets of data were run through the model before the calibration simulations in order to correctly initialize the saturation conditions in the subcatchments.

DESCRIPTION OF STORMS

The rainfall recorded during the flow survey period provided four (4) distinct storm events. These events occurred on January 19, 2013, February 10, 2013, February 21, 2013, and March 9/10, 2013.

Rainfall was recorded using ten (10) rain gauges distributed throughout the study area. Storm volumes, intensities, and durations were consistent over the monitoring period and were considered good for modeling purposes.

WET-WEATHER CALIBRATION

Wet-weather flows were generated in the model using both fixed response surfaces as well as infiltration flows:

- Up to three “fixed” response surface areas were calibrated for each subcatchment. These surface types are fundamentally independent of the catchment condition prior to the rainfall event and represent fast responses from areas such as illegally connected roof drainage and storm water cross connections.

- Infiltration was modeled using hydrology in the Ground Infiltration module within InfoWorks. This hydrological module has soil and groundwater storage zones and the inflow into the model is dependent upon the wetness of the catchment prior to the rainfall event. These flows represent the delayed ingress of storm water from the ground into the sewer system through cracks and leaks in sewers and private drains.

Review of the wet-weather response to rainfall indicated that the level of inflow (fast response) was in line with what would be expected in a sanitary sewer system. However the slower response which is due to rainfall raising groundwater levels and increasing ingress into the sewer system was significant. During the flow survey the total rainfall recorded over a 45 day period was 6.83 inches. This resulted in the catchment becoming extremely saturated and ground water ingress increasing throughout the survey period.

The model was not able to fully replicate the continual increase in ground water ingress recorded during the flow survey. The following Figure 4.1, illustrates a comparison between the flow meter data (green) and the model flows (blue). A good match has been achieved for the first storm event on January 29 2013. However, the model was not able to fully match the response recorded during the storm event March 9/10, 2013.

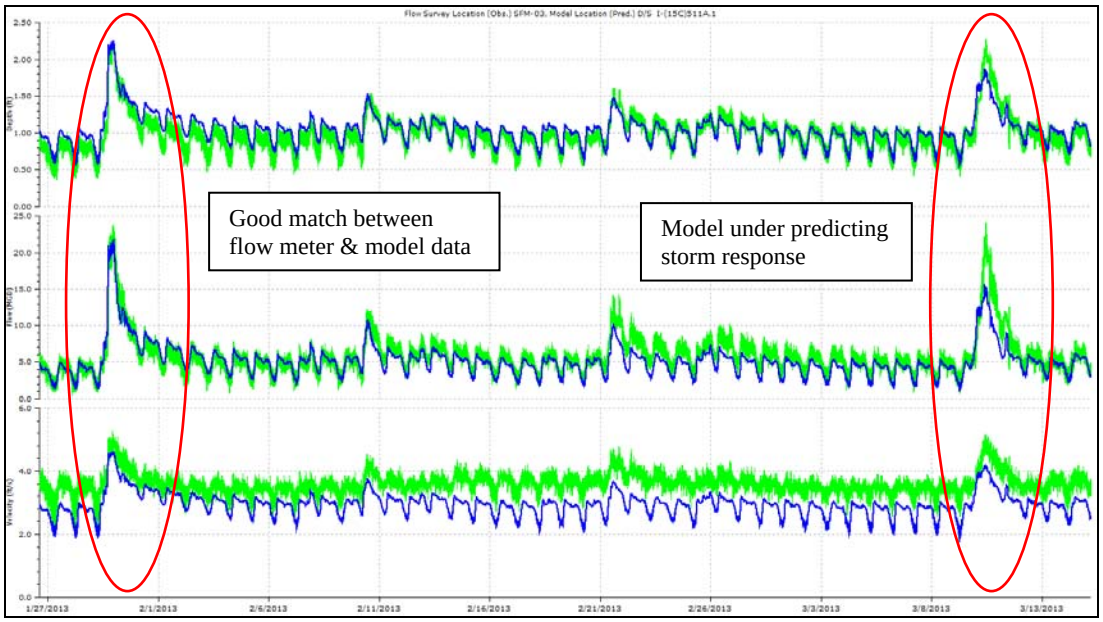


Figure 4.1: Flow Meter Response v Model Response

The wet weather calibration focused primarily on matching the response during storm event 1. It was felt that increasing the model response to match a period with such saturated ground conditions could lead to over prediction of system flows during a design storm event.

Final input values and comparison hydrographs for the wet-weather period can be found in the Appendix.

CHAPTER 5

CAPACITY ANALYSIS

This Chapter provides a summary of the hydraulic model capacity analysis for the existing, 2030 and ultimate build out scenarios.

DESIGN STORM

A 5-year/24-hour storm event with 2-year/60-minute peak intensity, with a peak intensity of approximately 1.80 inches/hour and total rainfall depth of 5.26 inches, shown in Figure 5.1 was selected as the design storm. The system response to the design storm was assessed to determine the collection system's capacity to transport peak wet weather flow.

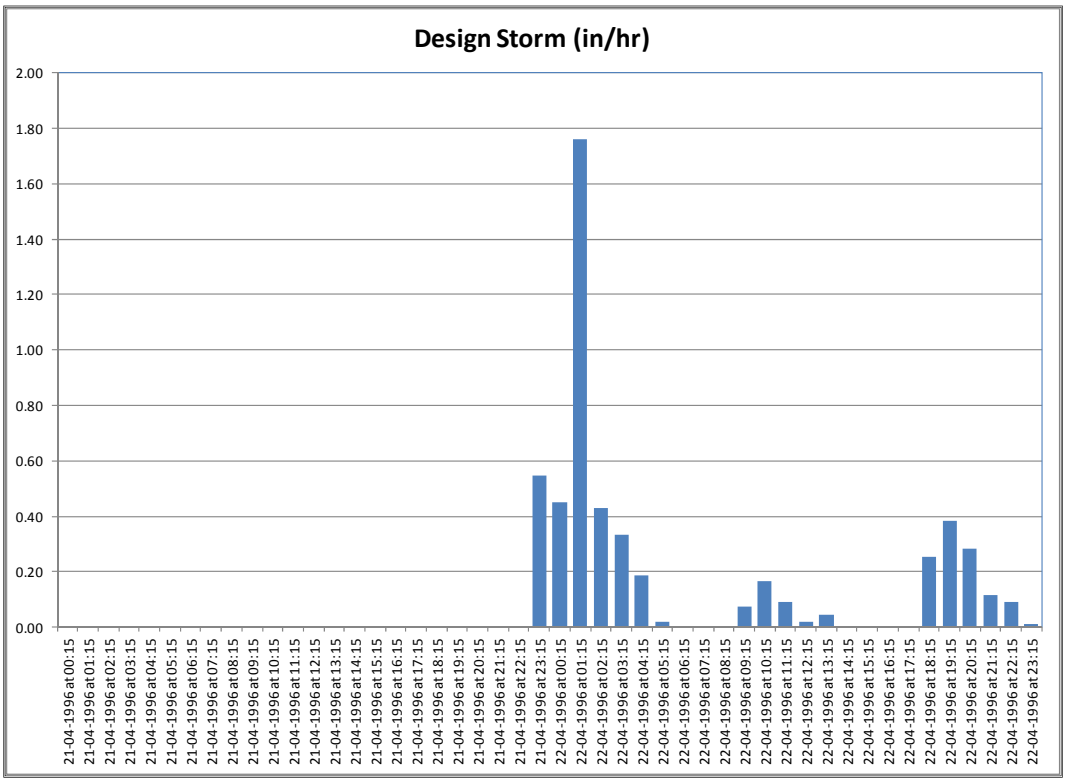


Figure 5.1

DESIGN CRITERIA

The design criteria for the Fayetteville sewer network is to convey all flows while maintaining a level of surcharge not to exceed three feet below manhole rim elevations.

EXISTING SYSTEM PERFORMANCE

DRY-WEATHER

The hydraulic model was calibrated to simulate the flows recorded during dry-weather. Based upon the dry-weather calibration all mains within the sanitary sewer system have sufficient capacity to transport existing dry-weather flows. All pump stations and force mains also show adequate capacity to convey existing dry-weather flow. A map that indicates the capacity utilized under existing dry-weather flow conditions by percentage for each line segment is shown in Exhibit 3.

WET WEATHER

The sewer system does not have sufficient capacity to convey wet weather flows. Large spills are predicted at the following locations:

- branch sewers connecting into the existing 16" main which drains north to Gregg PS
- 12" main along Overcrest St, upstream of Old Wire PS
- downstream of the flow split on Ramsey Avenue

The surcharge levels in 20.5 miles of the existing pipe network are within three feet of the manhole rim elevation. The pipe sections that are under capacity include the following:

- 10" main along Kitty Creek
- 16" main along Scull Creek
- 12" main discharging into Old Wire PS
- 11" mains downstream of the flow split at Ramsey Avenue

A map that indicates the capacity utilized under existing wet-weather flow conditions by percentage for each line segment and manhole spill locations is shown in Exhibit 4.

2030 SYSTEM PERFORMANCE

DRY-WEATHER

The existing sanitary sewer system has sufficient capacity to transport the dry-weather flows predicted for the 2030 scenario. All pump stations show adequate capacity to convey 2030 dry weather flows except for Stonewood PS which is predicted to run continuously for 16 hours during the day.

A map that indicates the capacity utilized under 2030 dry-weather flow conditions by percentage for each line segment is shown in Exhibit 5.

WET WEATHER

In addition to the spill locations identified in the existing wet weather system performance analysis, manhole spills are also predicted to occur at the following locations for the 2030 wet weather scenario:

- upstream of Stonewood PS
- upstream of Owl Creek PS

- upstream of Greenland PS
- upstream of McDonald PS
- upstream of Legacy Pointe V PS

A map that indicates the capacity utilized under 2030 wet-weather flow conditions by percentage for each line segment and manhole spill locations is shown in Exhibit 6.

ULTIMATE SYSTEM PERFORMANCE

Upgrades were added to the existing system in order to convey 2030 flows. The ultimate loading scenario was then applied to the sewer network model with 2030 upgrades. Details of the system performance of the 2030 network with ultimate flows are provided below.

DRY-WEATHER

The 2030 network does not have sufficient capacity to transport the dry-weather flows predicted for the ultimate scenario. Spills from manholes are predicted at the following locations:

- 8" main along Briarhill Drive, Farmington
- upstream of Owl Creek PS

A map that indicates the capacity utilized under ultimate build-out flow conditions by percentage for each line segment is shown in Exhibit 7.

WET WEATHER

Manhole spills are predicted to occur at the following locations in the 2030 network under ultimate wet weather loading conditions:

- upstream of Stonewood PS
- upstream of 12" main through Paradise Valley Golf Course
- upstream of Gregg PS
- 12" main along Overcrest St, upstream of Old Wire PS
- upstream of Noland WWTP
- 42" main along Huntville Rd
- upstream of Mally Wagnon PS
- upstream of Greenland PS
- 8" mains along Southwinds Rd, Killdeer Dr, Sundown Dr and Briarhill Dr, Farmington
- 6" pipe along Giles St, Farmington
- upstream of Owl Creek PS

A map that indicates the capacity utilized under ultimate build-out wet-weather flow conditions by percentage for each line segment and manhole spill locations is shown in Exhibit 8.

WASTEWATER TREATMENT PLANT FLOWS

A capacity analysis for both wastewater treatment plants was performed for Existing Conditions, Year 2030 Conditions and Ultimate Build-out Conditions. Flows were calculated for Average Dry Weather Flow (ADWF), Peak Hour Dry Weather Flow, Peak 24-Hr Wet Weather Flow, and Peak Hour Wet Weather Flow. The results are shown in Table 5-A.

Table 5-A

CAPACITY ANALYSIS FOR BOTH WASTEWATER TREATMENT PLANTS EXISTING CONDITIONS, YEAR 2030 CONDITIONS AND ULTIMATE BUILD-OUT CONDITIONS

		Existing	**2030**	**Build-out**
Model Population		78,618	108,570	304,086
		Flow (mgd)		
ADWF	Noland	3.92	6.81	14.00
	Westside	5.26	6.15	12.74
	Total	**9.18**	**12.96**	**26.74**
Peak Hour DWF	Noland	5.39	9.96	20.20
	Westside	8.02	10.17	21.10
	Total	**13.41**	**20.13**	**41.30**
Peak 24 Hr WWF	Noland	20.39	25.48	41.53
	Westside	29.89	31.70	39.89
	Total	**50.28**	**57.18**	**81.42**
Peak Hour WWF	Noland	28.96	32.00	46.00
	Westside	41.12	42.00	52.50
	Total	**70.08**	**74.00**	**98.50**

ENTERTAINMENT DISTRICT

City Staff requested an additional model run to determine the impact within the Entertainment District in the vicinity of Dickson Street as a result of infill, increased development of student housing, and possible future hotel rooms in the area.

The area evaluated is bounded by Lafayette Street to the North, Spring Street to the South, West Avenue to the west and Thompson Avenue to the east. A density of 500 beds/acre was used that resulted in a total population of approx 19,500. A sewer discharge rate of

60 gal/person/day was used. Running the design storm with the additional population results in surcharging sections of the pipeline as shown in Figure 5.2 below.

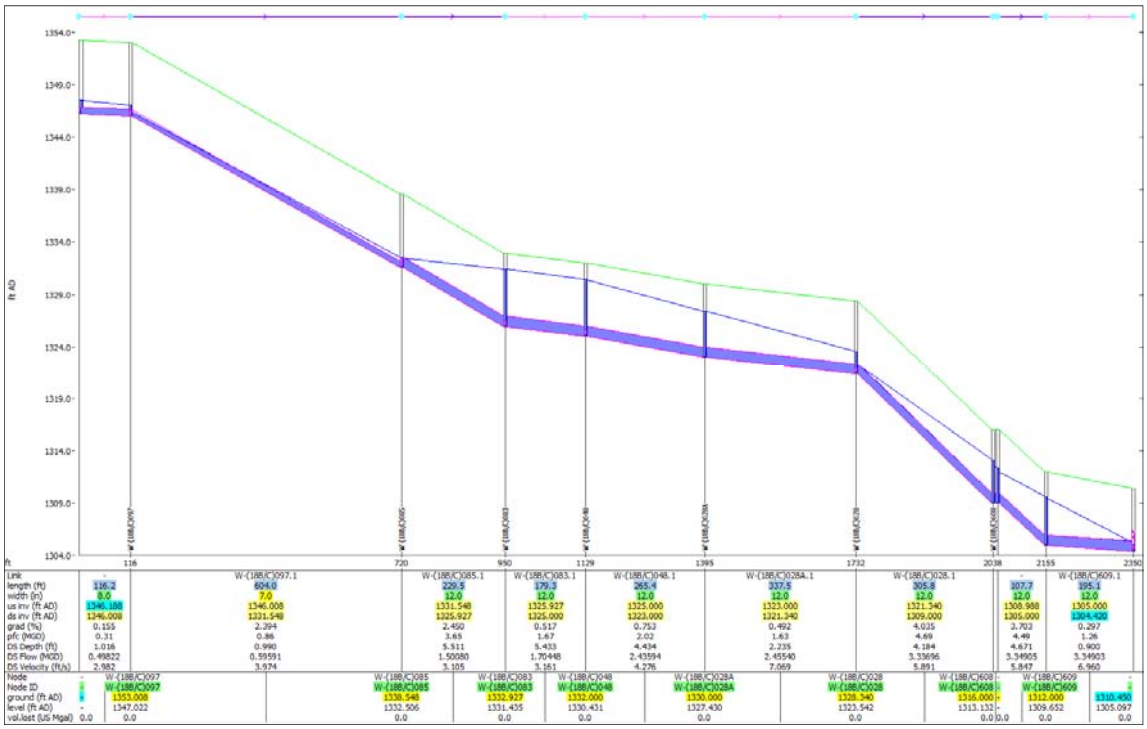


Figure 5.2

There are two sections of surcharged sewer where the water level is within 1.5 ft of the manhole rim level. The design criteria is for surcharge conditions not to exceed 3.0 feet below the manhole rim. It is recommended that the upper section of the sewer system be upsized from 12-inches to 18-in along West Avenue from just north of Dickson Street down to Spring Street from manhole W-(18B/C)083 to manhole W-(18B/C)028 as development occurs. The lower section should be upsized as development occurs along Spring Street.

CITY OF WEST FORK

In preparation of a future connection from the City of West Fork to the collection system, a continuous 2 MGD profile was added into manhole W-(06)042 in the vicinity of the Washington County Detention Center. The collection system upstream of this area does not have sufficient capacity to handle the increased flow from the City of West Fork. In the event that the future connection from the City of West Fork occurs upstream of manhole W-(06)042, then the design of any gravity sewer improvements should include an allowance of 2 MGD.

CHAPTER 6

CAPITAL IMPROVEMENT PLAN

This chapter presents a summary of the required plan to handle future growth flows under peak design storm conditions. The improvements were analyzed using the design criteria for the Fayetteville sewer network to convey all flows while maintaining a level of surcharge not to exceed three feet below manhole rim elevations. The improvements are divided into three phases as described below.

CAPACITY IMPROVEMENTS - IMMEDIATE

The following improvements are designed to eliminate model predicted overflows and to reduce surcharge in the existing sanitary sewer system under current population conditions.

PIPELINE IMPROVEMENTS

The required pipeline improvements include localized improvements to convey flow through the collection system with no overflows. The primary target areas are in the vicinity of Ramsey and Overcrest. These areas are the remaining known overflow locations that were not eliminated as part of the WSIP. The recommended capacity improvements and the estimated construction cost are shown in Table 6-A. The improvements are shown graphically in Figure 6.1.

CAPACITY IMPROVEMENTS – 2030

The following improvements are recommended to eliminate model predicted overflows and to reduce surcharge in the sanitary sewer system under future build-out conditions through year 2030.

PIPELINE IMPROVEMENTS

The required pipeline improvements include localized improvements to convey flow through the collection system with no overflows. The recommended capacity improvements and the estimated construction cost are shown in Table 6-B. The improvements are shown graphically in Figure 6.2.

PUMP STATION IMPROVEMENTS

As pipeline improvements are constructed, several pump station will be abandoned as listed below.

Copper Creek II
Stonebridge Meadows
Stonebridge Meadows II
Stonebridge Meadows V
Crescent Lake
Meadows
Dot Tipton
Masters

Table 6-A

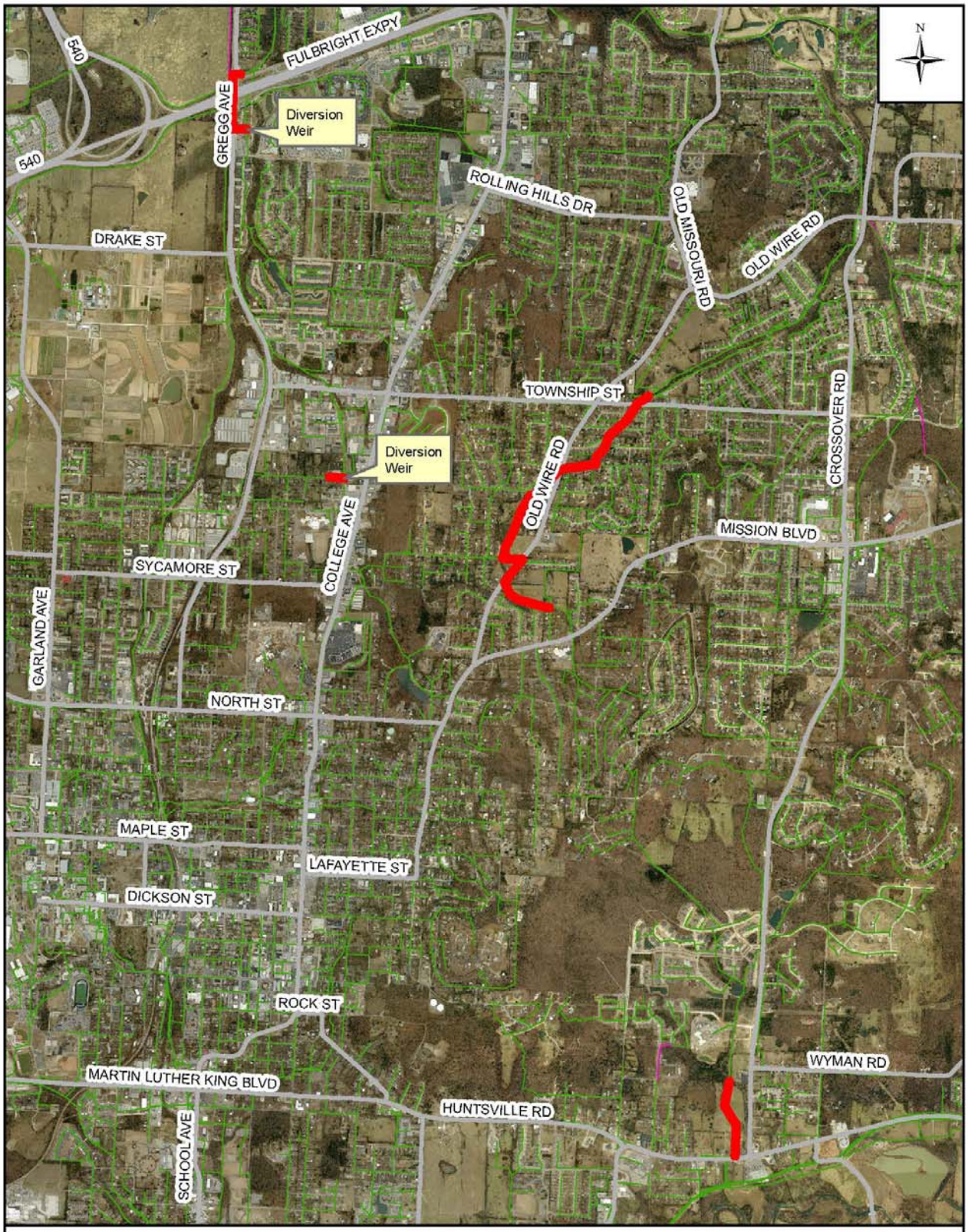
RECOMMENDED IMMEDIATE CAPACITY IMPROVEMENTS

System	Line Segment			Length (ft)	Existing Diameter (in)	Recommended Diameter (in)	Construction Cost ($)
Ramsey	I-(08)002	-	I-(08)001A	265	12	15	67,626
Ramsey	I-(08)038	-	I-(08)002	27	12	15	6,885
Ramsey	I-(08)051	-	I-(08)038	393	12	15	100,292
Ramsey	I-(08)055	-	I-(08)051	198	12	15	50,516
Ramsey	I-(08)062	-	I-(08)055	260	12	15	66,249
Ramsey	I-(08)063	-	I-(08)062	165	12	15	41,948
Ramsey	I-(08)066	-	I-(08)063	46	11	15	11,628
Ramsey	I-(08)071	-	I-(08)066	69	11	15	17,595
Ramsey	I-(08)072	-	I-(08)071	139	11	15	35,343
Ramsey	I-(08)076	-	I-(08)072	88	11	15	22,389
Ramsey	I-(08)077	-	I-(08)076	145	11	15	36,975
Ramsey	I-(08)078	-	I-(08)077	78	11	15	19,992
Ramsey	I-(09)001	-	I-(08)078	300	11	15	76,577
Ramsey	I-(09)002	-	I-(09)001	162	11	15	41,310
Ramsey	I-(09)018A	-	I-(09)002	184	11	15	46,869
Ramsey	I-(09)019	-	I-(09)018A	52	8	15	13,184
Ramsey	I-(09)020	-	I-(09)019	384	6	15	97,793
Ramsey	I-(09)021	-	I-(09)020	315	6	15	80,402
Ramsey	I-(09)086	-	I-(09)021	44	New	15	11,322
Ramsey	I-(09)087	-	I-(09)086	358	11	15	91,341
Ramsey	I-(09)088	-	I-(09)087	430	11	15	109,548
Ramsey	I-(09)089	-	I-(09)088	385	11	15	98,201
Ramsey	I-(09)090	-	I-(09)089	170	11	15	43,452
Ramsey	I-(09)091	-	I-(09)090	41	12	15	10,353
Ramsey	I-(09)092	-	I-(09)091	37	12	15	9,333
Ramsey	I-(09)093	-	I-(09)092	403	11	15	102,714
Ramsey	I-(09)096	-	I-(09)093	351	12	15	89,582
Ramsey	I-(09)097	-	I-(09)096	232	12	15	59,237
Ramsey	I-(16)189	-	I-(09)097	1,123	New	15	286,365
	Subtotal			6,843			1,745,016
Crossover	W-(09)013	-	W-(09)012	315	12	18	99,288
Crossover	W-(09)014	-	W-(09)013	314	12	18	98,753
Crossover	W-(09)015	-	W-(09)014	452	12	18	142,506
Crossover	W-(09)016	-	W-(09)015	508	12	18	159,894
	Subtotal			1,589			500,441

Table 6-A (Cont.)

RECOMMENDED IMMEDIATE CAPACITY IMPROVEMENTS

System	Line Segment			Length (ft)	Existing Diameter (in)	Recommended Diameter (in)	Construction Cost ($)
Gregg	I-(01)029	-	I-(01)028	150	16	16	41,361
Gregg	I-(01)030	-	I-(01)029	191	16	16	52,443
Gregg	I-(01)463	-	I-(01)030	399	16	16	109,615
Gregg	I-(01)460	-	I-(01)463	11	16	16	2,915
Gregg	I-(01)462	-	I-(01)460	509	16	16	139,865
Gregg	I-(01)133	-	I-(01)462	299	16	16	82,253
	Subtotal			1,558			428,451
Gregg	I-(01)133			Weir			2,500
Green Acres	I-(03)052			Weir			2,500
	Subtotal						5,000
	Total			**9,990**			**2,678,908**



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Immediate Capital Improvements
City of Fayetteville Master Plan
Figure 6.1

CITY OF Fayetteville ARKANSAS

Table 6-B

RECOMMENDED CAPACITY IMPROVEMENTS THROUGH 2030

System	Line Segment			Length (ft)	Existing Diameter (in)	Recommended Diameter (in)	Construction Cost ($)
Kitty Creek	I-(17)045	-	I-(17)044	330	10	18	103,919
Kitty Creek	I-(17)046	-	I-(17)045	272	10	18	85,680
Kitty Creek	I-(17)047	-	I-(17)046	255	10	18	80,357
Kitty Creek	I-(17)048	-	I-(17)047	157	10	18	49,487
Kitty Creek	I-(17)049	-	I-(17)048	204	10	18	64,134
Kitty Creek	I-(17)050	-	I-(17)049	204	10	18	64,355
Kitty Creek	I-(17)051	-	I-(17)050	204	10	18	64,103
Kitty Creek	I-(17)052	-	I-(17)051	273	10	18	86,027
Kitty Creek	I-(17)053	-	I-(17)052	197	10	18	62,055
Kitty Creek	I-(17)056	-	I-(17)053	107	10	18	33,674
Kitty Creek	I-(17)057	-	I-(17)056	156	10	15	39,806
Kitty Creek	I-(17)060	-	I-(17)057	158	10	15	40,316
Kitty Creek	I-(17)061	-	I-(17)060	159	10	15	40,469
Kitty Creek	I-(17)070	-	I-(17)061	151	10	15	38,505
Kitty Creek	I-(17)071	-	I-(17)070	131	10	15	33,507
Kitty Creek	I-(17)083	-	I-(17)071	158	10	15	40,392
Kitty Creek	I-(17)084	-	I-(17)083	221	10	15	56,432
Kitty Creek	I-(17)085	-	I-(17)084	232	10	15	59,262
Kitty Creek	I-(17)010	-	I-(17)085	25	10	15	6,452
Kitty Creek	I-(17)086	-	I-(17)010	141	10	15	36,032
Kitty Creek	I-(17)096	-	I-(17)086	69	10	15	17,544
Kitty Creek	I-(17)102	-	I-(17)096	245	10	15	62,424
Kitty Creek	I-(17)103	-	I-(17)102	288	10	15	73,389
Kitty Creek	I-(17)104	-	I-(17)103	288	10	15	73,517
Kitty Creek	I-(17)105	-	I-(17)104	188	10	15	47,940
Kitty Creek	I-(17)105A	-	I-(17)105	40	10	15	10,124
Kitty Creek	I-(17)106	-	I-(17)105A	162	10	15	41,336
Kitty Creek	I-(17)107	-	I-(17)106	185	10	15	47,124
Kitty Creek	I-(17)108	-	I-(17)107	204	10	15	52,046
Kitty Creek	I-(17)109	-	I-(17)108	285	10	15	72,573
Kitty Creek	I-(17)110	-	I-(17)109	314	10	15	79,943
Kitty Creek	I-(17)111	-	I-(17)110	294	10	15	74,996
Kitty Creek	I-(17)112	-	I-(17)111	240	10	15	61,124
Kitty Creek	I-(17)117	-	I-(17)112	343	10	15	87,440
Kitty Creek	I-(17)118	-	I-(17)117	343	10	15	87,491

Table 6-B (Cont.)

RECOMMENDED CAPACITY IMPROVEMENTS THROUGH 2030

System	Line Segment			Length (ft)	Existing Diameter (in)	Recommended Diameter (in)	Construction Cost ($)
Kitty Creek	I-(17)119	-	I-(17)118	260	10	15	66,326
Kitty Creek	I-(17)120	-	I-(17)119	296	10	15	75,531
Kitty Creek	I-(17)122	-	I-(17)120	299	9	15	76,169
Kitty Creek	I-(17)123	-	I-(17)122	296	10	15	75,455
Kitty Creek	I-(17)124	-	I-(17)123	328	12	15	83,589
Kitty Creek	I-(17)125	-	I-(17)124	137	10	15	34,808
	Subtotal			**8,838**			**2,385,840**
Mally Wagnon	W-(24B)069	-	W-(24A)088	259	12	18	81,585
Mally Wagnon	W-(24B)070	-	W-(24B)069	320	12	18	100,832
Mally Wagnon	W-(24B)071	-	W-(24B)070	261	12	18	82,310
Mally Wagnon	W-(24B)072	-	W-(24B)071	279	12	18	87,948
Mally Wagnon	W-(24B)073	-	W-(24B)072	386	12	18	121,716
Mally Wagnon	W-(24B)425	-	W-(24B)073	248	12	18	78,215
Mally Wagnon	W-(24B)074	-	W-(24B)425	102	12	18	32,225
Mally Wagnon	W-(24B)412	-	W-(24B)074	211	10	18	66,402
Mally Wagnon	W-(24B)090	-	W-(24B)412	134	10	18	42,210
Mally Wagnon	W-(24B)091	-	W-(24B)090	412	10	18	129,654
Mally Wagnon	W-(24B)092	-	W-(24B)091	385	10	18	121,149
Mally Wagnon	W-(24B)112	-	W-(24B)092	339	10	18	106,848
Mally Wagnon	W-(24B)113	-	W-(24B)112	236	10	18	74,246
Mally Wagnon	W-(24B)114	-	W-(24B)113	247	10	18	77,711
Mally Wagnon	W-(24B)115	-	W-(24B)114	239	10	18	75,222
Mally Wagnon	W-(24B)142	-	W-(24B)115	136	8	18	42,935
Mally Wagnon	W-(24B)143	-	W-(24B)142	268	8	18	84,357
Mally Wagnon	W-(24B)143A	-	W-(24B)143	398	8	18	125,496
Mally Wagnon	W-(24B)143B	-	W-(24B)143A	377	8	18	118,881
Mally Wagnon	W-(24B)143C	-	W-(24B)143B	254	8	18	80,073
Mally Wagnon	W-(24B)143D	-	W-(24B)143C	82	8	18	25,799
Mally Wagnon	W-(24B)143E	-	W-(24B)143D	31	8	18	9,639
Mally Wagnon	W-(24B)144	-	W-(24B)143E	257	8	18	80,861
Mally Wagnon	W-(24B)145	-	W-(24B)144	19	8	18	5,954
Mally Wagnon	W-(24B)146	-	W-(24B)145	252	8	18	79,443
Mally Wagnon	W-(24B)146A	-	W-(24B)146	238	8	18	75,096
Mally Wagnon	W-(24B)147	-	W-(24B)146A	20	8	18	6,300
Mally Wagnon	W-(24B)148	-	W-(24B)147	181	8	18	57,141
Mally Wagnon	W-(24B)149	-	W-(24B)148	39	8	18	12,317
Mally Wagnon	W-(24B)150	-	W-(24B)149	273	8	18	85,869

Table 6-B (Cont.)

RECOMMENDED CAPACITY IMPROVEMENTS THROUGH 2030

System	Line Segment			Length (ft)	Existing Diameter (in)	Recommended Diameter (in)	Construction Cost ($)
Mally Wagnon	W-(24B)151	-	W-(24B)150	117	8	18	36,981
Mally Wagnon	W-(24B)152	-	W-(24B)151	139	8	18	43,848
Mally Wagnon	W-(24B)153	-	W-(24B)152	264	8	18	83,097
Mally Wagnon	W-(24B)154	-	W-(24B)153	152	8	18	47,880
	Subtotal			**7,556**			**2,380,235**
County Jail	W-(06)042A	-	W-(06)042	112	12	18	35,406
County Jail	W-(06)041A	-	W-(06)042A	192	12	18	60,480
County Jail	W-(06)041	-	W-(06)041A	9	12	18	2,709
County Jail	W-(06)040A	-	W-(06)041	491	12	18	154,728
County Jail	W-(06)040	-	W-(06)040A	22	12	18	6,804
County Jail	W-(06)039	-	W-(06)040	321	12	18	101,178
County Jail	W-(06)034	-	W-(06)039	422	12	18	132,993
County Jail	W-(06)032	-	W-(06)034	299	12	18	94,248
County Jail	W-(06)033	-	W-(06)032	394	12	18	124,110
	Subtotal			**2,262**			**712,656**
Airport North	W-(20)001	-	PS Airport North WW	87	12	15	22,134
Airport North	W-(20)002	-	W-(20)001	87	12	15	143,973
Airport North	W-(20)003	-	W-(20)002	565	12	15	92,999
Airport North	W-(20)004	-	W-(20)003	365	12	15	118,091
Airport North	W-(20)008	-	W-(20)004	463	12	15	62,628
Airport North	W-(20)009	-	W-(20)008	246	10	15	27,795
Airport North	W-(20)011	-	W-(20)009	109	10	15	101,031
Airport North	W-(20)012	-	W-(20)011	396	10	15	22,823
Airport North	W-(20)013	-	W-(20)012	90	10	15	52,199
Airport North	W-(20)014	-	W-(20)013	205	10	15	101,439
Airport North	W-(20)015	-	W-(20)014	398	10	15	82,008
Airport North	W-(20)016	-	W-(20)015	322	10	15	93,687
Airport North	W-(20)017	-	W-(20)016	367	10	15	83,870
Airport North	W-(20)017A	-	W-(20)017	329	10	15	77,699
Airport North	W-(20)017B	-	W-(20)017A	305	10	15	54,111
	Subtotal			**4,457**			**1,136,484**
Stonewood	New MH_0035	-	I-(17A)041	1,136	New	12	272,616
Stonewood	New MH_0034	-	New MH_0035	269	New	12	64,632
Stonewood	New MH_0036	-	New MH_0034	696	New	12	167,016

Table 6-B (Cont.)

RECOMMENDED CAPACITY IMPROVEMENTS THROUGH 2030

System	Line Segment			Length (ft)	Existing Diameter (in)	Recommended Diameter (in)	Construction Cost ($)
Stonewood	New MH_0033	-	New MH_0036	1,412	New	12	338,952
Stonewood	New MH_0032	-	New MH_0033	1,269	New	8	203,040
Stonewood	New MH_0024	-	New MH_0032	1,296	New	8	207,392
	Subtotal			**6,079**			**1,253,648**
Crossover/Joyce	New MH_0018	-	I-(17)158	1,324	New	12	238,284
Crossover/Joyce	New MH_0019	-	New MH_0018	801	New	12	144,126
Crossover/Joyce	New MH_0020	-	New MH_0019	761	New	12	136,998
Crossover/Joyce	New MH_0021	-	New MH_0020	298	New	12	53,550
Crossover/Joyce	New MH_0022	-	New MH_0021	331	New	12	59,580
Crossover/Joyce	New MH_0023	-	New MH_0022	1,466	New	12	263,934
	Subtotal			**4,980**			**896,472**
Goose Creek	New MH_0063	-	PS Farmington West	543	New	15	138,491
Goose Creek	New MH_0062	-	New MH_0063	1,239	New	15	315,869
Goose Creek	New MH_0059	-	New MH_0062	1,357	New	15	346,035
Goose Creek	New MH_0061	-	New MH_0059	1,365	New	15	347,973
Goose Creek	New MH_0057	-	New MH_0061	1,058	New	8	169,328
Goose Creek	New MH_0060	-	New MH_0057	760	New	8	121,552
	Subtotal			**6,321**			**1,439,247**
Dot Tipton PS				1,388	New	8	222,080
	Subtotal			**1,388**			**222,080**
Stonebridge	New MH_0047	-	W-(24B)154	944	New	18	297,266
Stonebridge	New MH_0046	-	New MH_0047	1,450	New	18	456,782
Stonebridge	New MH_0044	-	New MH_0046	737	New	18	232,124
Stonebridge	New MH_0045	-	New MH_0044	918	New	18	289,076
Stonebridge	New MH_0043	-	New MH_0045	1,462	New	18	460,404
Stonebridge	New MH_0048	-	New MH_0045	407	New	8	65,040
Stonebridge	New MH_0049	-	New MH_0048	1,144	New	8	183,008
Stonebridge	PS Stonebridge Meadows II WW	-	New MH_0049	946	New	8	151,328
Stonebridge	PS Crescent Lake WW	-	New MH_0048	856	New	8	136,880

Table 6-B (Cont.)

RECOMMENDED CAPACITY IMPROVEMENTS THROUGH 2030

System	Line Segment			Length (ft)	Existing Diameter (in)	Recommended Diameter (in)	Construction Cost ($)
Stonebridge	PS Stonebridge Meadows V WW	-	New MH_0046	380	New	8	60,800
Stonebridge	PS Stonebridge Meadows WW	-	New MH_0047	65	New	8	10,448
	Subtotal			**9,307**			**2,343,154**
Masters	PS Masters WW	-	W-(15X)349A	3,903	New	8	624,480
	Subtotal			**3,903**			**624,480**
Stonewood PS	Upgrade		0.37 to 2.4 MGD				666,624
Airport North PS	Upgrade		1.52 to 2.2 MGD				611,072
Greenland PS	Upgrade		0.84 to 1.5 MGD				416,640
Industrial Park PS	Upgrade		3.38 to 6 MGD				1,666,560
Owl Creek PS	Upgrade		0.9 to 1.8 MGD				444,416
							3,805,312
	Total			**55,092**			**17,498,432**

Capacity upgrades will also be necessary at the following pump stations as areas develop.

Stonewood, increase from 0.37 mgd to 2.4 mgd
Airport North, increase from 1.52 mgd to 2.2 mgd
Greenland, increase from 0.84 mgd to 1.5 mgd
Industrial Park, increase from 3.38 mgd to 6.0 mgd
Owl Creek, increase from 0.9 mgd to 1.6 mgd

Estimated construction costs for the pump station upgrades are included in Table 6-B.

CAPACITY IMPROVEMENTS – ULTIMATE BUILD-OUT

The following improvements are recommended to eliminate model predicted overflows and to reduce surcharge in the existing sanitary sewer system under future Ultimate Build-out conditions.

PIPELINE IMPROVEMENTS

The required pipeline improvements include localized improvements to convey flow through the collection system with no overflows as well as new sewer line to convey flow from new development. The estimated construction cost of the recommended capacity improvements for Ultimate Build-out is $57.08 million dollars. The improvements are shown graphically in Figure 6.3.

PUMP STATION IMPROVEMENTS

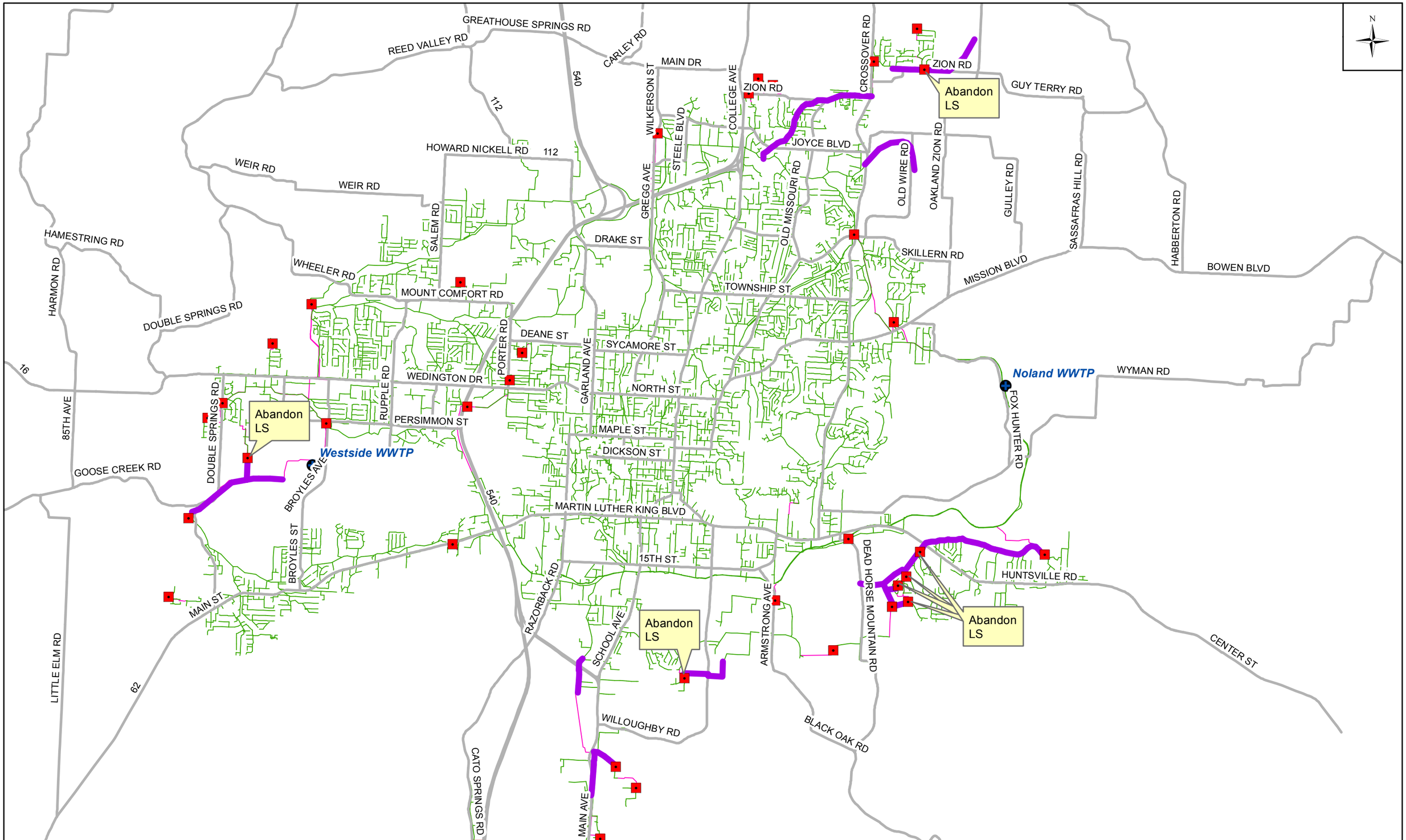
As pipeline improvements are constructed, another pump station will be abandoned as listed below.

McDonald

As development progresses into the Ultimate Build-out areas, the terrain within the City of Fayetteville sewer collection system will require approximately seven new sewer pump stations and upgrades to another nine pump stations. The estimated construction cost of the recommended pump station improvements for Ultimate Build-out is $14.60 million dollars. The improvements are shown graphically in Figure 6.3.

IMPROVEMENT PROJECT COST

The estimated total capital cost to implement the recommended improvements includes engineering, land acquisition, and contingencies which are estimated to be 30% of the construction cost. A summary of the estimated capital cost is given in Table 6-C.



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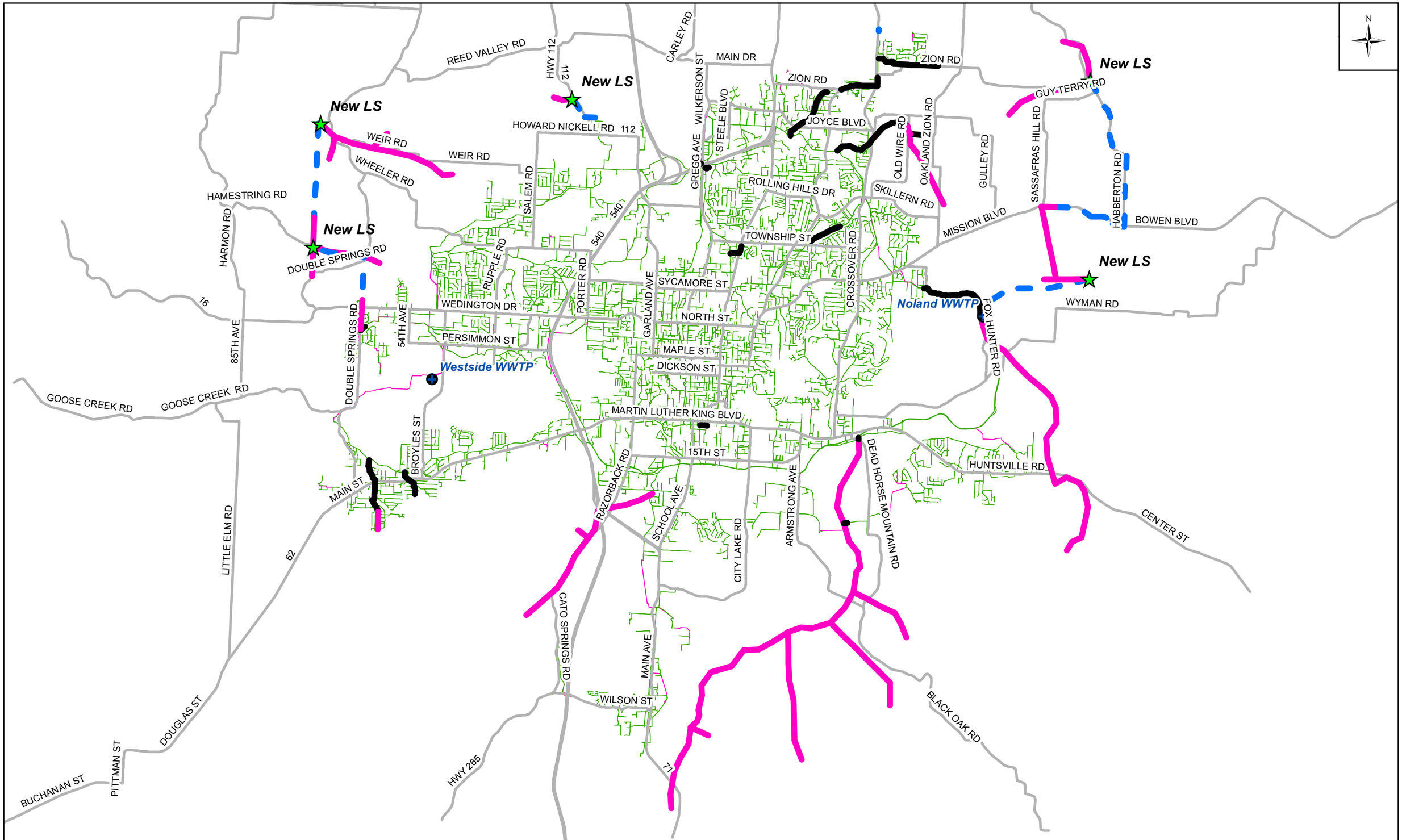
The Choice for Collection System Solutions

2030 Upgrades

Capital ImprovementsThrough 2030
City of Fayetteville Master Plan
Figure 6.2

CITY OF Fayetteville ARKANSAS



rjngroup
The Choice for Collection System Solutions

Ultimate New Forcemains
Ultimate New Gravity Sewer
Ultimate Sewer Upgrades
New LS

Ultimate Capital Improvements
City of Fayetteville Master Plan
Figure 6.3

CITY OF Fayetteville ARKANSAS

Table 6-C

SUMMARY OF CAPITAL IMPROVEMENTS

Item	Description	Estimated Capital Improvement Cost ($ Million)[1/]
Pipeline Improvements		
	Immediate Improvements	3.48
	Improvements Through Year 2030	22.75
	Improvements Through Ultimate Build-out	93.19

1/ *Estimated Capital Cost is given in 2014 dollars. No cost escalation is included for future years.*

PROJECT SCHEDULE

Recommended capacity improvements for existing capacity restrictions should be considered for construction first as these areas are predicted to be under capacity according to the established design criteria. Capacity improvements for future growth areas should be constructed prior to the growth occurring. The timing of this construction will be dependent on the rate and predicted year of development according to the City staff.

A capital improvement schedule is shown in Table 6-D. The table lists recommended improvements by budget year with construction cost (excluding engineering, land acquisition, contingencies). The table includes both immediate improvements for years 2015 and 2016 and the remaining improvements through 2030.

The Masters pump station is recommend to be abandoned in year 2017. The pump station has been identified as a high priority pump station to be abandoned and a new sewer line constructed down the hillside to the east. The pump station is on the side of a hill that is very unstable and the access to the pump station is regularly washed out during rain events and subsequently adds increased cost to routine operation and maintenance costs. As funds become available, City staff may want to consider moving the recommended improvements to an earlier date.

Capital Improvements Through 2030
Fayetteville Wastewater Master Plan

Project	Estimated Construction Cost	Proposed Budget Year															
		2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030
Line Work																	
Gregg Avenue	$ 428,451		$ 428,451														
Ramsey	$ 1,745,016	$ 1,745,016															
Crossover	$ 500,441		$ 500,441														
Diversion weirs	$ 5,000		$ 5,000														
Kitty Creek outfall	$ 2,385,840				$ 1,385,840	1000000											
Mally Wagnon outfall	$ 2,380,235											$ 1,380,235	$ 1,000,000				
County Jail	$ 712,656			$ 712,656													
Airport North PS main line	$ 1,136,484						$ 1,136,484										
Stonewood PS interceptor	$ 1,253,648								$ 1,253,648								
Crossover / Joyce	$ 1,195,296																$ 1,195,296
Goose Creek outfall - increase to 15"	$ 1,439,247									$ 719,624	$ 719,623						
Dot Tipton PS elimination	$ 222,080										$ 222,080						
Deadhorse to Mally Wagnon outfall at White River bridge	$ 1,735,650													$ 1,735,650			
Stonebridge Meadows II PS elimination	$ 399,376														$ 399,376		
Crescent Lake PS elimination	$ 136,880														$ 136,880		
Stonebridge Meadows V PS elimination	$ 60,800														$ 60,800		
Stonebridge Meadows PS elimination	$ 10,448														$ 10,448		
Masters PS elimination	$ 624,480			$ 624,480													
Pump Stations Upgrades																	
Stonewood PS	$ 666,624									$ 666,624							
Airport North PS	$ 611,072							$ 611,072									
Greenland PS	$ 416,640							$ 416,640									
Industrial Park PS	$ 1,666,560														$ 833,280	$ 833,280	
Owl Creek PS	$ 444,416					$ 444,416											
Total	**$ 20,177,340**	**$ 1,745,016**	**$ 933,892**	**$ 1,337,136**	**$ 1,385,840**	**$ 1,444,416**	**$ 1,136,484**	**$ 1,027,712**	**$ 1,253,648**	**$ 1,386,248**	**$ 941,703**	**$ 1,380,235**	**$ 1,000,000**	**$ 1,735,650**	**$ 1,440,784**	**$ 833,280**	**$ 1,195,296**

TABLE 6-D

CHAPTER 7

RECOMMENDED MAINTENANCE PLAN

This chapter presents a summary of the recommended maintenance plan to handle ongoing sewer rehabilitation efforts that were initiated in 1990 and will need to continue into the future.

SEWER REHABILITATION HISTORY

The City of Fayetteville began evaluating the sewer collection system for defects and to identify sources of inflow and infiltrations in 1990. Table 7-A lists the areas within the City that have had SSES work performed and Figure 7.1 shows them graphically. As you can see from the figure, almost the entire City has an SSES performed. There are three areas that have not been evaluated, W-25 and W-9 (except for the Hyland Park area) in the vicinity of Hwy 265 and Hwy 16 and W-24 in the area along Hwy 16 east of the West Fork of the White River. Table 7-A also lists the areas within the City that have had sewer rehabilitation construction projects performed that were identified by SSES and Figure 7.2 shows them graphically.

FUTURE SSES RECOMMENDATIONS

A significant part of the City's core sewer collection system bounded by I-49 to the west, Clear Creek to the north, Hwy 265 to the east, and Hwy 16 to the south has had SSES and rehab project performed in the 1990s. Some of the areas have not been inspected in over 20 years.

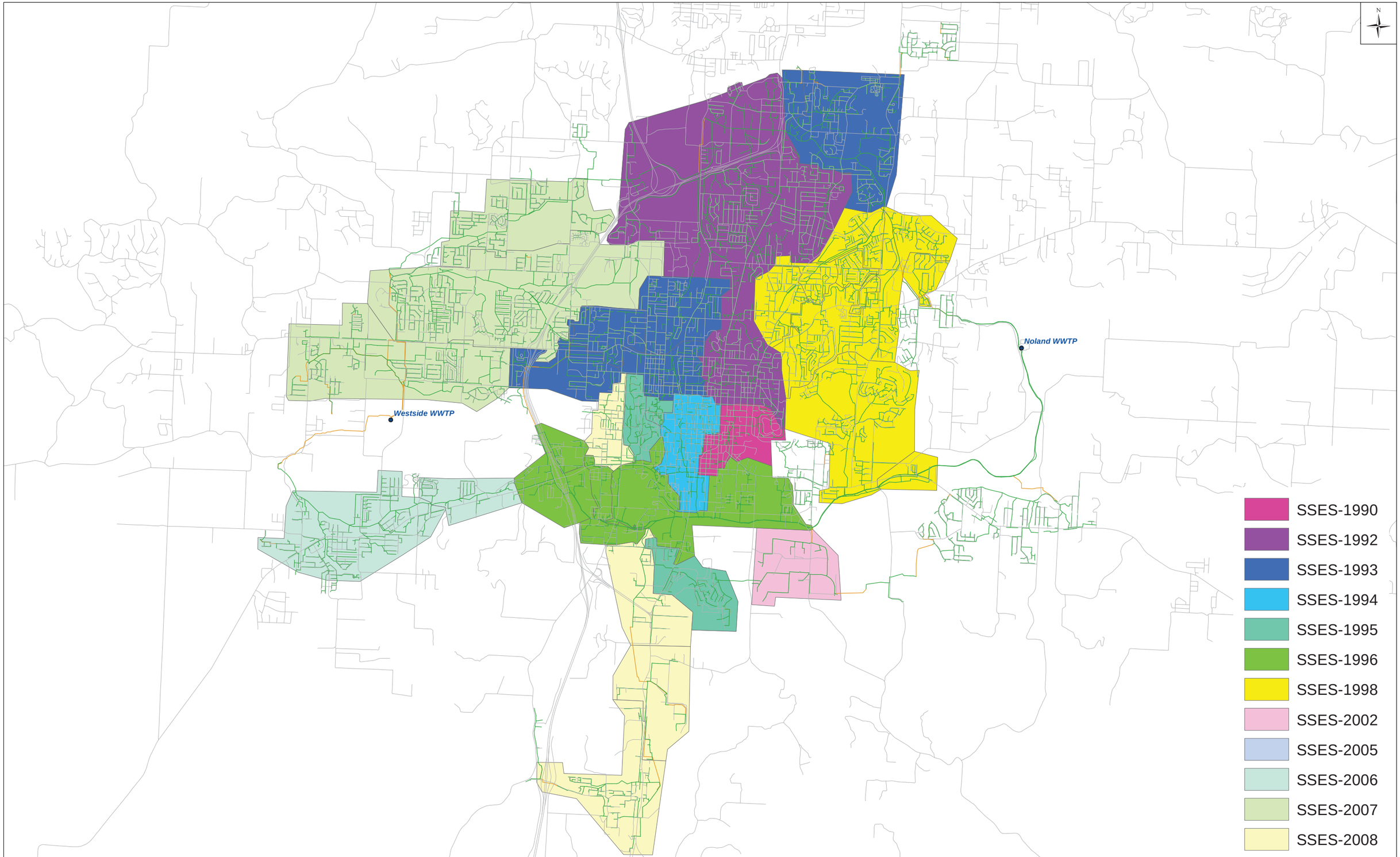
Inflow rankings are shown in Figure 2.4. The EPA maximum recommended inflow rate is 10,000 gallon per day per 1,000 linear feet of sewer line. The basin with the highest inflow rate is SMF-05. Figure 7.3 shows a wet weather hydrograph with two flow meters, SMF-05 and SMF-06. SMF-05 includes flow from SMF-06. As you can see from the figure, if you deduct the amount of flow from SMF-06 from SMF-05, there is still an elevated level of sewer flow that is not attributed to domestic or commercial/industrial water usage during wet weather.

It is recommend that the City continue SSES work on the basins identified in Figure 2.4 beginning with the highest inflow ranked basin (SFM-05) and continuing through Basin SMF-17. Multiple basins can be evaluated in a given year. The approximate maximum sewer line footage that can be inspected is 150,000 linear feet.

Table 7-A

CITY OF FAYETTEVILLE
SSES / REHAB HISTORY

Basin(s)	Type	Year
Basin W-02	SSES	1990
Basins I-1, 2, 3, 4, 5, 10, 11, 18, & 19	SSES	1992
Basins I-12, 13, 14, 17, 20, 21, 22	SSES	1993
Basin W-18 B/C	SSES	1994
Basin W-05	SSES	1995
U of A	SSES	1995
Basins W-7A, 14, & 15	SSES	1996
Basins I-6, 7, 8, 9, 16, & W-9A	SSES	1998
Basins I-1, 2, 3, 4, 5, 18, 19, & W-27	SSES	2002
Basins I-10 & I-11	SSES	2005
Farmington	SSES	2006
Basin I-15	SSES	2007
Basins W-5, 6, 13B, 22, & 32	SSES	2008
City-Wide Master Plan	MP	1996
City-Wide Master Plan Update	MP	2001
Basin I-10 & 11	Manhole & Pipeline Rehab	1993
Basin W-18 B/C	Manhole Rehab	1994
Basins I-12, 13, 14, 17, 20, 21, & 22	Manhole Rehab	1994
Basins I-12, 13, 14, 17, 20, 21, & 22	Pipeline Rehab	1995
Basin W-18 B/C	Pipeline Rehab	1995
Basins W-7A, 13A, 13B, 14, 15 & 18A	Manhole & Pipeline Rehab	1997
Basins W-13B, 15, 18 B/C & I-10, 12, 20, & 21	Pipeline Rehab	1998
Basins I-6, 7, 8, 9, 16 and W-9A	Manhole & Pipeline Rehab	1999
Basins I-1, 2, 3, 4, 5, 18, 19 & W-27	Manhole Rehab	2002
Farmington	Manhole Rehab	2005
Basins I-1, 2, 3, 4, 5, 10, 12, 16, 17, 19, and W-2 &W-27	Pipeline Rehab	2005
Farmington	Pipeline Rehab	2006
Basins I-15, W-02, 5, 6, & 13A	Manhole Rehab	2008
Basin W-32	Pipeline Rehab	2009



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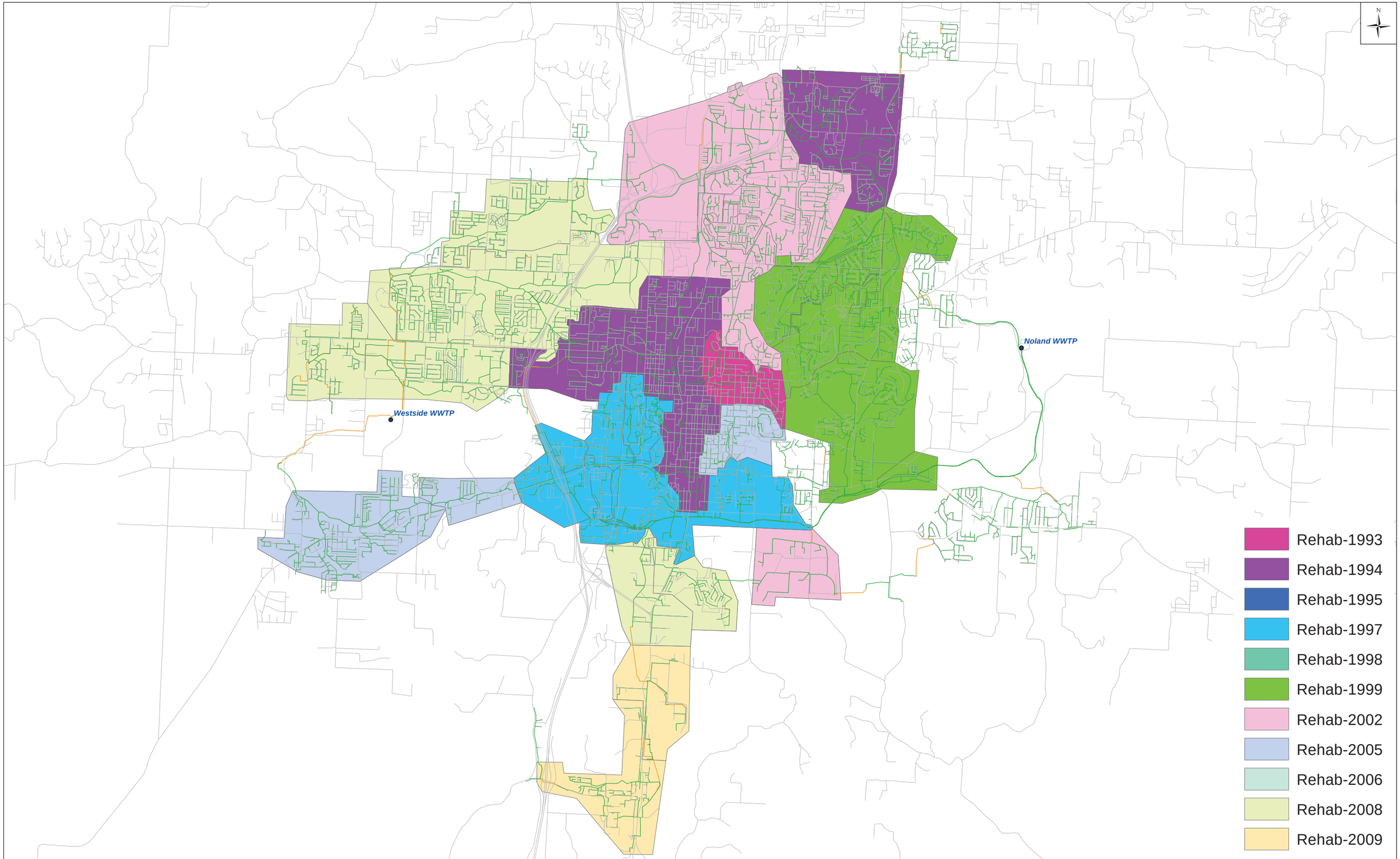
CITY OF Fayetteville ARKANSAS

SSES History
City of Fayetteville Master Plan
Figure 7.1



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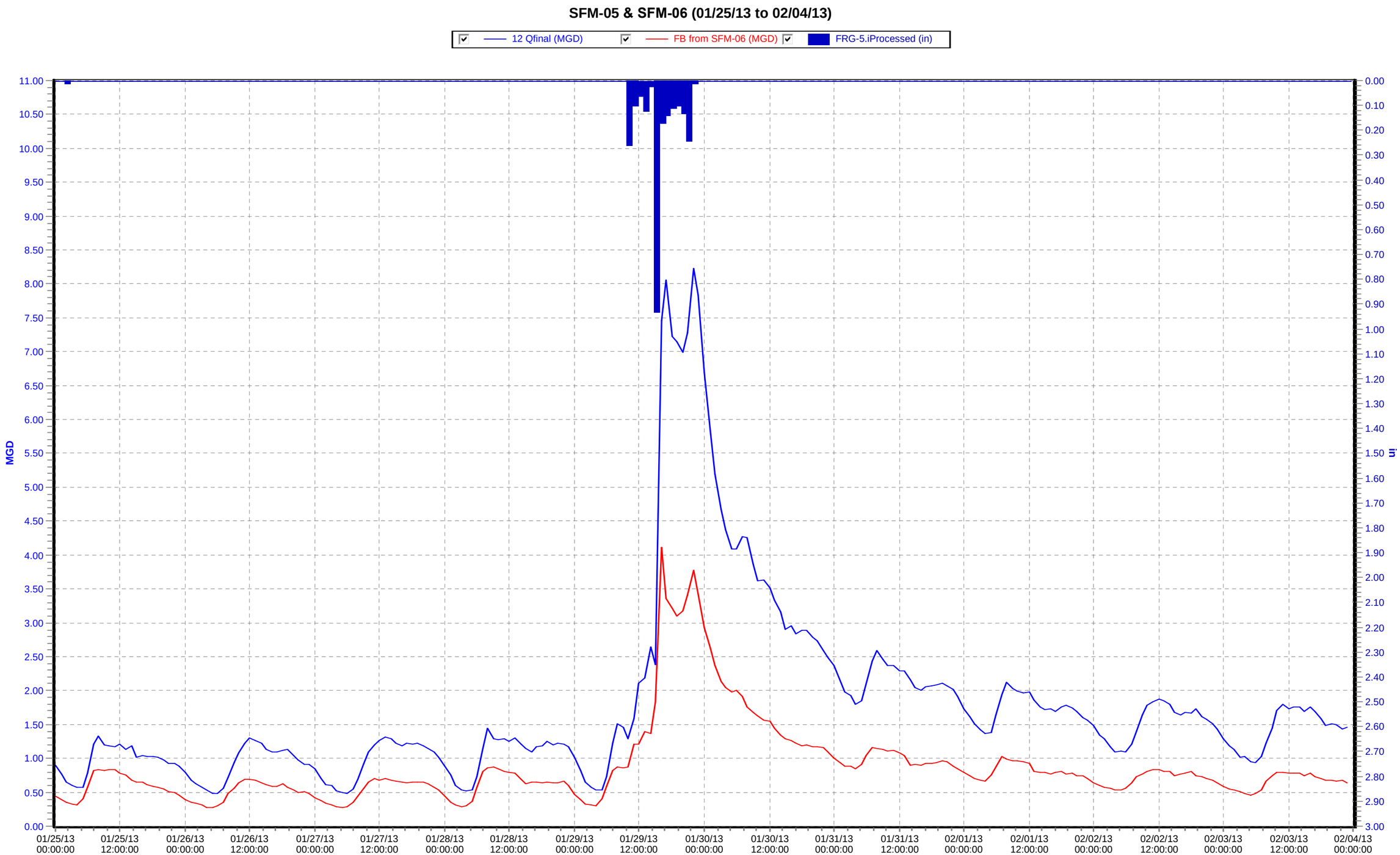
CITY OF Fayetteville ARKANSAS

Rehab History
City of Fayetteville Master Plan
Figure 7.2


SFM-05 & SFM-06 (01/25/13 to 02/04/13)
12 Qfinal (MGD)
FB from SFM-06 (MGD)
FRG-5.iProcessed (in)
MGD
in
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10.00
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ADDITIONAL RECOMMENDATIONS

36-inch Sewer Line – Happy Hollow to Noland WWTP

The existing 36-inch diameter ductile iron sewer line that runs from the City of Fayetteville Compost Facility to the Noland WWTP was constructed in the 1960s. A new parallel 42-inch diameter sewer line was constructed in 2008. Both of the sewer lines are necessary to carry the projected sewer flows through ultimate build-out.

The 36-inch ductile iron pipe was made with a polyethylene liner to reduce the effects of the highly corrosive sewer environment. Over the years, portions of the liner have shown up at the screening facility at the Noland WWTP. While the 36-inch ductile iron pipe was used as a force main from the Happy Hollow PS, the corrosion risk was low as long as the pipe was full. Now that the pipe has been converted back to a gravity sewer line and is not running full, the risk of H2S buildup and bacteria converting the gas to sulfuric acid is significantly higher. Sewer maintenance crews have identified holes within the pipe at two different locations within the last year. It is recommended that the City perform a structural integrity inspection of the existing 36-inch ductile iron sewer line.

The estimated cost for inspection services is $117,000. The estimated cost to line 26,500 feet of 36-inch DIP with cured-in-place-pipe could range from $5.96 Million to $8.22 Million depending on the condition of the pipe. For comparison purposes, the parallel 42-inch sewer that was recently installed was constructed for $10.7 Million.

Fox Hunter Sewer Lines

The parallel sewer lines that start at the top of the hill in the vicinity of the Barrington Park subdivision and follows Fox Hunter down to the Noland WWTP have also been identified as a maintenance issue. These lines are identified in the Ultimate Build-out planning as needing upsized. It is recommended that the sewer lines be further evaluated.

Budget prices were not calculated for this project because the length of sewer pipe as well as the diameter will not be determined until an evaluation has been performed.

Rain Gauges

Rain gauges help attribute rainfall to sewer subcatchments and can assist with wet weather flow managements by giving “real-time” rainfall measurements throughout the City. The City’s rain gauges were purchases over 15-years ago. The average life cycle for rain gauges equipment is 10-years. It is recommended that the City purchase new rain gauges and connect them to the Water/Wastewater SCADA system. Approximately 10 rain gauges will be required to cover the City’s wastewater collection system area.

The estimated cost to purchase rain gauges and install telemetry is $17,350.

Flow Meter Telemetry

The City purchased seventeen (17) flow meters to be permanently installed a few years ago. The field technician with the Water/Sewer Department has to visit each site routinely to download data and perform diagnostics checks. Adding telemetry to the existing flow meters would reduce the frequency of site visits and would provide for remote downloading of the flow data and system diagnostics. This has proven well with the SCADA system for

monitoring the sewer pump stations. It is recommended that the City pursue installing telemetry on the permanent flow meters.

The estimated cost to install telemetry on the permanent flow meters is $51,000. There will be an ongoing monthly charge depending on the cell service provider for each site.

Model Upkeep

The City has made a significant investment in both money and time to build and calibrate the wastewater model. The model will need to be kept up to date by including infrastructure improvements and checking performance with the City's permanent flow meters. Ideally, the model would be kept up to date as new development is approved for construction and verified as construction is completed. This is a daunting task without a full-time assigned employee. Realistically, the model should be updated with all sewer improvements on an annual basis. The model should be ran with the new improvements added and compared to the data obtained from the permanent flow meters to verify that the model is tracking with the real world flow data.

It is recommended that the wastewater model be re-calibrated approximately every 5-7 years depending on the results of the annual updates. The re-calibration of the model will include the installation of temporary flow meters to supplement the City's permanent flow meter network. Sub-Basins and catchments will be reviewed and updated. Population and land use data will also be evaluated and updated to reflect current conditions.

The estimate annual cost to retain a consultant to upkeep the model is $25,000. This will involve adding new sewer lines to the model as a result of development, sewer rehabilitation, and sewer line replacement.