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November 8, 2010

Mr. Howell Anderson, P.E.
Little Rock Wastewater
11 Clearwater Drive
Little Rock, AR 72204

Subject: 2010 System Evaluation and Capacity Assurance (SECAP) Update

Dear Mr. Anderson:

In accordance with the December 1, 2009 Engineering Agreement, RJN Group, Inc. is pleased to submit this final report for the above referenced project. The final report includes the comments received from Little Rock Wastewater.

We appreciate the opportunity to work with Little Rock Wastewater and the excellent cooperation from the LRW staff throughout the project. We look forward to working with LRW in the future. Should you have any questions, please call.

Respectfully Submitted,

RJN GROUP, INC.

Hugh M. Kelso
Vice President

Daniel Jackson, P.E.
Project Manager

HMK/DJ/kb/2382
Enclosure

1200 John Barrow Road, Suite 315 • Little Rock, AR 72204 • 501.224.2248 • fax 501.224.2484

2010 SYSTEM EVALUATION AND CAPACITY ASSURANCE (SECAP) UPDATE

Little Rock Wastewater

Little Rock, Arkansas

November 2010

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.

Dan Jackson

Date: 11/8/10 Registration No.: 13978

EXECUTIVE SUMMARY

This Executive Summary presents the results of the System Evaluation and Capacity Assurance Plan (SECAP) Update developed for Little Rock Wastewater (LRW) by RJN Group Inc. The Update is to the original SECAP prepared for Little Rock Wastewater by Montgomery Watson Harza in 2002 and was authorized by the Little Rock Sewer Committee in December 2009.

BACKGROUND AND SECAP OBJECTIVES

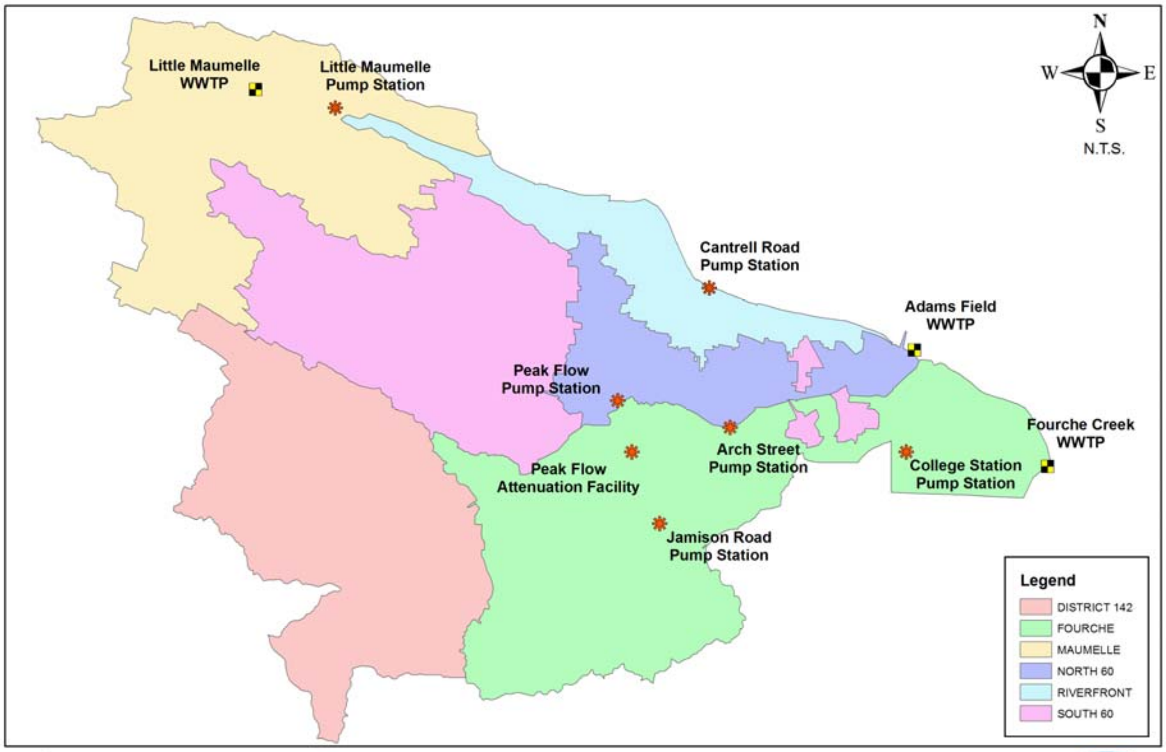
The 2002 SECAP resulted in a Capital Improvement plan to eliminate overflows and bring the wastewater system into compliance with the Consent Administrative Order (CAO) and Settlement Agreement by 2016. Many of the projects contained in the original SECAP have been implemented. LRW retained RJN to evaluate the impact of the completed projects and to validate the need for the remaining improvements and/or develop additional alternatives.

The objectives of the SECAP Update are as follows:

- Complete City-wide wastewater flow monitoring to update existing flows
- Update the existing hydraulic model obtained from LRW
- Identify existing capacity deficiencies and capacity requirements
- Analyze the existing pump stations, flow equalization (EQ) Basins, and Wastewater Treatment Plants (WWTP) and provide recommendations for operational efficiency
- Develop improvement projects and budget estimates for implementing the required collection system capacity improvements
- Provide recommendations for potential inflow/infiltration reduction
- Determine capacity requirements for future growth
- Provide an improvement plan to remove overflows for the design storm

SERVICE AREA AND COLLECTION SYSTEM

Little Rock Wastewater maintains the wastewater collection and treatment facilities for the City of Little Rock. The service area includes six primary basins: Riverfront, Fourche, North 60, South 60, District 142, and Little Maumelle. The Utility’s wastewater service area boundary is shown in Figure 1.



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LITTLE ROCK WASTEWATER UTILITY MAP
FIGURE 1

Little Rock Wastewater

LRW provides service to over 67,000 customers and maintains over 1,300 miles of collection system lines ranging in size from 6 to 60-inches in diameter. Little Rock Wastewater currently operates two wastewater treatment plants, Adams Field and Fourche Creek. In addition, the Little Maumelle Wastewater Treatment Plant (WWTP) is being constructed in the northwest portion of the City and should be operational in the early part of 2011.

Major conveyance facilities operated by LRW include Arch Street, Cantrell Road, College Station, Little Maumelle, and Jamison Road Pump Stations. Little Maumelle Pump Station is currently being reconfigured to convey flows to the new Little Rock Treatment Plant. The existing College Station Pump Station will be removed from service and replaced with a smaller station to convey flow from local services only.

FLOW/RAINFALL MONITORING

RJN conducted City-wide flow and rainfall monitoring as part of this project.

The data from the flow and rainfall monitoring period was analyzed to determine average daily dry and wet-weather flows for each of the monitored sub-basins. The average daily dry-weather flow for the collection system excluding the Little Maumelle Drainage Area was 32.7 mgd while the Little Maumelle area dry-weather flow was 1.8 mgd.

The data indicated that many of the sub-basins have a significant response to wet-weather events. Wet-weather peaking factors varied from 3.0 to 59.3 when compared to average dry-weather flows.

DESIGN FLOWS/DESIGN STORM

The design storm used for the model analysis was provided by LRW and occurred during the original SECAP. This storm is a recorded event that occurred in November 2000. The 48-hour event was recorded in 2x2 km pixels by a NEXRAD system. The average rainfall of 4.15 inches is similar to a 2-year/48-hour storm event for the region. The November 2000 rainfall event equates to a design event with a return period between two and five years.

HYDRAULIC MODEL UPDATE/CALIBRATION

Little Rock Wastewater provided RJN Group, Inc with the hydraulic model used in the 2002 SECAP report. The model which was constructed from record drawings and available Geographical Information System (GIS) data consisted of sewer mains 10 inches and larger in diameter with built in storage compensation for un-modeled mains.

As part of the scope of this project, the model was updated to include any sewer mains 10 inches and larger in diameter constructed since 2002 and add selected 8-inch diameter mains up to reported overflow locations.

In addition, the model update included incorporating existing designed but not yet constructed improvements as well as capacity improvements recommended in the SSES reports prepared by RJN.

CAPACITY ANALYSIS

The updated and calibrated model was used to evaluate the performance of the collection system pipelines, pump stations, and storage facilities during the design storm event. The analysis identified local capacity restrictions as well as system restrictions which cause sanitary sewer overflows because of backwater effects.

ANALYSIS OF ALTERNATIVES

Alternatives were evaluated to develop a capital improvement plan to eliminate sanitary sewer overflows based on the design storm event. The calibrated hydraulic model was utilized to identify capacity improvements that are localized in nature, as well as to evaluate various improvements to address overflows that are more holistic. The evaluations included the entire collection system with the exception of the area tributary to the new Little Maumelle Wastewater Treatment Plant.

Localized improvements are those that are required to eliminate isolated overflows caused by capacity restrictions in a given area and are not related to existing system restrictions that cause a backwater effect. The holistic improvements are for those overflows that are generally caused by multiple deficiencies or backwater from downstream restrictions and must be addressed by a combination of alternatives.

Due to the complexity of the LRW system and the interconnectivity between the various interceptors and WWTPs, it was decided to combine the various LRW service areas into four areas for the purpose of identifying alternatives to eliminate these deficiencies. The areas are as follows:

Cantrell Road Pump Station Area
Rock Creek Area
North 60, South 60/Fourche Interceptor Area
Riverfront Area and Adams Field WWTP

Alternatives were developed and presented during a workshop with Little Rock Wastewater personnel with pros and cons given for each. Each alternative was evaluated and either eliminated from further consideration or further developed to include operational and costing impacts. Alternatives that progressed for further consideration were modeled and then evaluated using the metrics of hydraulic performance, elimination of overflows, increased capacity, and constructability.

CAPITAL IMPROVEMENT PLAN

In conjunction with Little Rock Wastewater, an improvement plan was developed to eliminate sanitary sewer overflows in the collection system during the design storm event. The capital improvement plan is segregated into required improvements and additional improvements.

REQUIRED IMPROVEMENTS

The required improvements are those necessary to eliminate the reported overflows that occur during the design storm event as addressed in the Consent Administrative Order (CAO) and Settlement Agreement. The required improvements include pipeline and peak flow storage facilities as well as operational changes at selected pump stations. Each is addressed below.

Pipeline Improvements

The required pipeline improvements include both localized and holistic improvements. A summary of the pipeline projects is provided in Table 1.

Table 1

SUMMARY OF PIPELINE IMPROVEMENTS

Item	**Description**	**Estimated Capital Improvement Cost ($ Million)**[1/]
Local Improvements	23,002 lf of 10-inch to 60-inch diameter sewer	5.90
System Improvements	15,123 lf of 18-inch to 48-inch diameter sewer	11.74
Total	38,125 lf of 10-inch to 60-inch diameter sewer	**17.64**

1/ *Estimated Capital Cost is based on 2010 dollars. No cost escalation is included for future years.*

PEAK WET-WEATHER STORAGE FACILITIES

The Capital Improvement Plan for the construction of additional peak flow storage facilities includes construction of a new facility at the Mabelvale Pike location and adding an additional basin at the Adams Field Wastewater Treatment Plant. This plan also includes construction of the Rock Creek Storage and Cantrell Road In-Line storage facilities. A summary of each facility is given in Table 2.

Table 2

SUMMARY OF PEAK FLOW STORAGE FACILITIES

Location	**Description**	**Estimated Capital Improvement Cost ($ Million)[1/]**
Mabelvale Pike	51 mg Basin Storage	49.01
Adams Field WWTP	14 mg Basin Storage	12.62
Rock Creek	7 mg In-Line Storage	20.49
Cantrell Road In-Line	4 mg In-Line Storage	12.15
Additional Pump at Peak Flow PS	1-20,560 gpm pump	0.97
Total		**95.24**

1/ *Estimated Capital Cost is based on 2010 dollars. No cost escalation is included for future years.*

PUMP STATION OPERATION PARAMETERS

The evaluation of alternatives identified the pipeline and storage facility improvements that are required to eliminate the known sanitary sewer overflows that occur during the design storm event. In some cases the success of these improvements in eliminating the overflows is dependent on making changes to the operational parameters at several pump stations. These changes basically require one set of operation parameters for dry-weather periods and a different set for wet-weather periods to allow for maximum utilization of the existing LRW infrastructure. The changes are required at the Adams Field, Arch Street, Cantrell Road, and Peak Flow Pump Stations. A detailed discussion of the required changes is included in Chapter 5 and Chapter 6 of this report.

INFLOW/INFILTRATION (I/I) REDUCTION IN THE CANTRELL ROAD BASIN

Peak flow reduction in the area tributary to the Cantrell Road Pump Station may reduce the size of the storage facility in the upper Cantrell Road area and also reduce the demand placed on the pump station and downstream interceptors and treatment facility during wet-weather periods. I/I rates in these areas are significant with observed peaking factors being as high as 14.2. A summary of the estimated cost for the I/I Reduction Program is given in Table 3.

Table 3

SUMMARY OF I/I REDUCTION COST
CANTRELL ROAD PUMP STATION AREA

Description	Estimated Capital Cost ($ Million)[1/]
Sanitary Sewer Evaluation Study (SSES)	1.36
Rehabilitation Design / Construction	13.16
Total	**14.52**

1/ *Estimated Capital Cost is based on 2010 dollars. No cost escalation is included for future years.*

REQUIRED IMPROVEMENT PROJECT COST

The total estimated capital cost to implement the capital improvement plan is $127.6 million. This consists of $17.8 Million for pipeline improvements and $95.3 million for peak flow storage facilities. An additional $14.5 million is included for I/I investigations and collection system rehabilitation in the area tributary to the Cantrell Road Pump Station. The estimated capital cost includes costs for construction, engineering, land acquisition, and 15 percent contingency. A summary of the estimated capital cost is provided in Table 4.

Table 4

SUMMARY OF
ESTIMATED CAPITAL IMPROVEMENT COST

Item	Estimated Capital Cost ($ Million)[1/]
Pipeline Improvements	17.78
Peak Flow Storage Facilities	95.24
Cantrell Road Basin I/I Reduction	14.52
Total	**127.54**

1/ *Estimated Capital Cost is based on 2010 dollars. No cost escalation is included for future years.*

ADDITIONAL IMPROVEMENTS

Additional improvements include investigating and if confirmed, eliminating overflows projected by the model to occur during the design storm event. These model predicted overflows are at manholes not previously documented as an overflow location. Also included are improvements to the Cantrell Road Pump Station. These are addressed in the following paragraphs.

UNDOCUMENTED / MODEL PREDICTED OVERFLOWS

The hydraulic model predicted overflows in locations that have not been documented. As part of this project, pipeline improvements and estimated capital cost were developed to eliminate these potential overflows. A summary of the improvements necessary to eliminate these potential overflows is given in Table 5.

Table 5

SUMMARY OF IMPROVEMENTS FOR UNDOCUMENTED / MODEL PREDICTED OVERFLOWS (IF REQUIRED)

Description	**Estimated Capital Improvement Cost ($ Million)[1/]**
22,069 LF of 8-inch to 21-inch Diameter Sewer	4.97
Total	**4.97**

1/ *Estimated Capital Cost is based on 2010 dollars. No cost escalation is included for future years.*

It is recommended that the LRW conduct a site visit of each these locations to inspect for any evidence of overflow prior to initiating any improvement project.

CANTRELL ROAD PUMP STATION

It is recommended that the electrical and mechanical components be replaced. It is also recommended that an additional force main be constructed and the existing force main inspected and rehabilitated, as required. A summary of the recommended improvements is provided in Table 6.

Table 6

SUMMARY OF CANTRELL ROAD PUMP STATION IMPROVEMENTS

Item	**Description**	**Estimated Capital Improvement Cost ($ Million)[1/]**
Pump Station Upgrade	Includes Mechanical & Electrical Upgrades	6.60
Force Main	Includes New Force Main and Rehab of Existing Force Main	2.22
Total		**8.82**

1/ *Estimated Capital Cost is based on 2010 dollars. No cost escalation is included for future years.*

CAPACITY IMPROVEMENTS FOR FUTURE GROWTH

This update to the 2002 SECAP made allowance for future flows by analyzing planning area maps, zoning requirements, and land use maps outside of the Little Maumelle WWTP tributary area. It is estimated that the Little Rock population has the potential to increase by approximately 13,000 primarily in the southwest portion of the City. As this population develops, additional sewer improvements will be required in the District 142 system. A summary of these future improvements is provided in Table 7.

Table 7

SUMMARY OF CAPACITY IMPROVEMENTS FOR FUTURE GROWTH AREA

Item	Description	Estimated Capital Improvement Cost ($ Million)[1/]
Pipeline Improvements	10,200 lf of 12-inch to 21-inch diameter sewer	3.15
Total		**3.15**

1/ *Estimated Capital Cost is based on 2010 dollars. No cost escalation is included for future years.*

ADDITIONAL INVESTIGATIONS

There are eight reported overflow locations that the hydraulic model did not replicate with all but one being Category C overflows. Category C overflows are defined as an overflow that occurs during a storm greater than the design storm event. It is recommended that LRW conduct CCTV inspections downstream of these locations to determine if there may be a structural cause to the overflows. Pipe diameter and invert/rim elevations should also be obtained to compare to data in the model.

ASSUMPTIONS

The alternatives developed and the capital improvements included in this update are based on the following assumptions:

- LRW will complete construction of all mains previously designed but not constructed
- Complete construction of improvements recommended in previous SSES Reports
- Re-program Adams Field Main Pump Station (MPS), Arch Street, and Cantrell Road Pump Stations with the wet-weather operating levels recommended in this report

The hydraulic model utilized during this update was updated to reflect the above assumptions. If any of the improvements are not implemented, it may have an impact on the success of the overflow elimination.

Summary of Capital Improvement Plan

The Capital Improvement Plan contained in this report will eliminate the reported overflows associated with the Consent Administrative Order and Settlement Agreement. The plan also contains recommendations for a continuing I/I Reduction Program as well as for upgrading the key Cantrell Pump Station. In addition, as growth occurs in the southwest portion of the City, the plan includes pipeline improvements to accommodate growth.

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CHAPTER 1

INTRODUCTION

This report presents the results of the System Evaluation and Capacity Assurance Plan (SECAP) Update that Little Rock Wastewater (LRW) retained RJN Group, Inc. to perform. The update is to the original SECAP prepared by Montgomery Watson Harza dated March 2002.

BACKGROUND AND OBJECTIVES

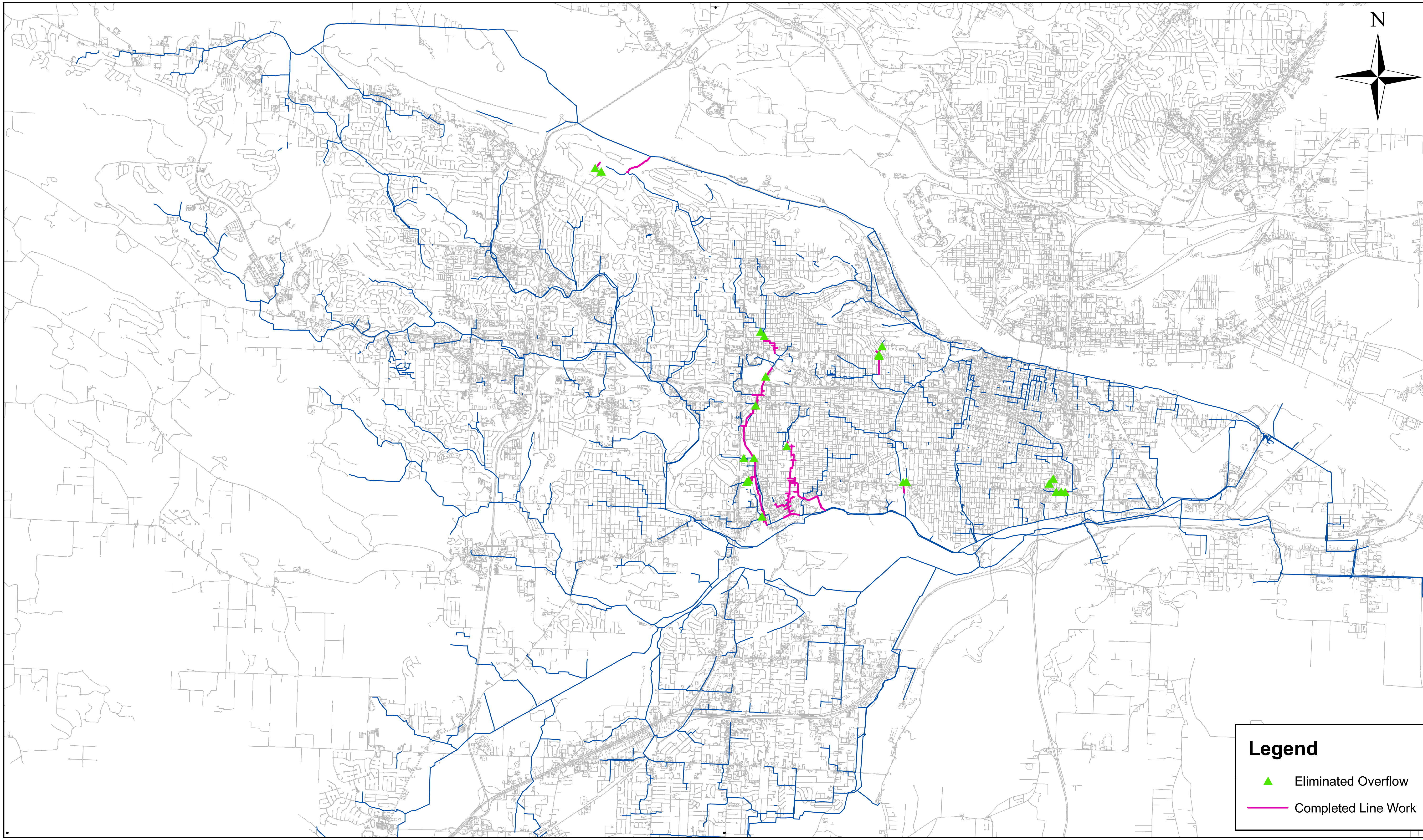
The initial 2002 SECAP resulted in a Capital Improvement Plan to eliminate overflows and bring the wastewater system into compliance with the Consent Administrative Order (CAO) and Settlement Agreement by 2016. Many of the projects contained in the original SECAP have been implemented. LRW retained RJN to evaluate the impact of the completed projects and to validate the need for the remaining improvements and/or develop additional options to eliminate overflows.

The overflows that have been eliminated by constructed projects completed by LRW are shown on Figure 1.1. Overflows that will be eliminated by projects designed by LRW but not yet constructed are shown on Figure 1.2.

The objectives of the SECAP Update are:

- Complete City-wide wastewater flow monitoring to update existing flows
- Update the existing hydraulic model obtained from LRW
- Identify existing capacity deficiencies and capacity requirements
- Analyze the existing pump stations, Flow Equalization (EQ) Basins, and Wastewater Treatment Plants (WWTP) and provide recommendations for operational efficiency
- Develop improvement projects and budget estimates for implementing the required capacity improvements
- Provide recommendations for potential inflow/infiltration (I/I) reduction
- Determine capacity requirements for future growth
- Provide a Capital Improvement Plan to remove overflows for the design storm event

Eliminated Overflows



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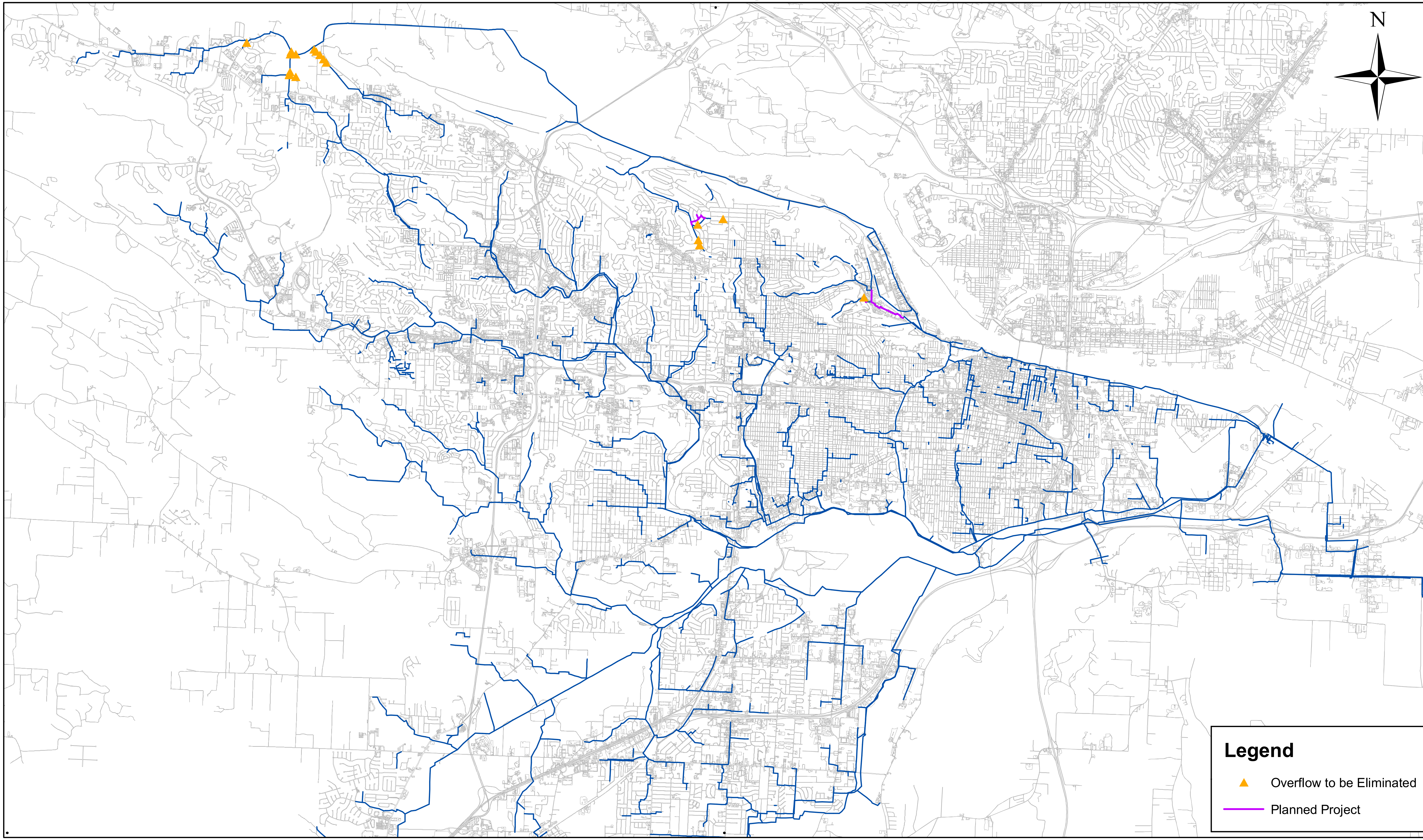
LITTLE ROCK WASTEWATER
SECAP UPDATE
FIGURE 1.1

Overflows to be Removed by Planned Projects



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LITTLE ROCK WASTEWATER
SECAP UPDATE
FIGURE 1.2

Little Rock Wastewater Collection System and Service Area

Little Rock Wastewater provides wastewater collection and treatment facilities for the City of Little Rock. The service area includes six primary basins: Riverfront, Fourche, North 60, South 60, District 142, and Little Maumelle. The LRW's wastewater service area boundary is shown in Figure 1.3 on page 1-5.

LRW provides service to over 67,000 customers and maintains over 1,300 miles of collection system lines ranging in size from 6 to 60-inches in diameter. Little Rock Wastewater currently owns and operates two wastewater treatment plants, Adams Field and Fourche Creek. In addition, since the original report in 2002, the Little Maumelle Wastewater Treatment Plant (WWTP) is being constructed in the northwest portion of the City. Little Maumelle WWTP should be operational in the early part of 2011 and was not evaluated as part of this study. However, wastewater flow data that will enter this treatment facility was monitored. Adams Field WWTP has a design flow of 36 million gallons per day (mgd) with a maximum capacity of 94 mgd. The maximum capacity is based on what the Adams Field Main Pump Station (MPS) can pump into the treatment and on-site storage facilities. Fourche Creek WWTP has a design flow of 16 mgd. Improvements and expansion work currently underway will increase the peak flow rate to 45 mgd. Future planned improvements will increase the plant capacity to 52 mgd.

Major conveyance facilities operated by Little Rock Wastewater include the Arch Street, Cantrell Road, Little Maumelle, and Jamison Road Pump Stations. Little Maumelle Pump Station is being reconfigured to convey flows to the new Little Maumelle WWTP. Arch Street Pump Station is currently undergoing expansion and modification, and based on pump curve data and design criteria, the station will have a capacity of 45 mgd. The existing College Station Pump Station will be removed from service and replaced with a smaller station to only convey flow from local services. Cantrell Road Pump Station has a calculated capacity of 32 mgd and Jamison Road Pump Station can handle a capacity of 16 mgd.

Scope Of SECAP Update

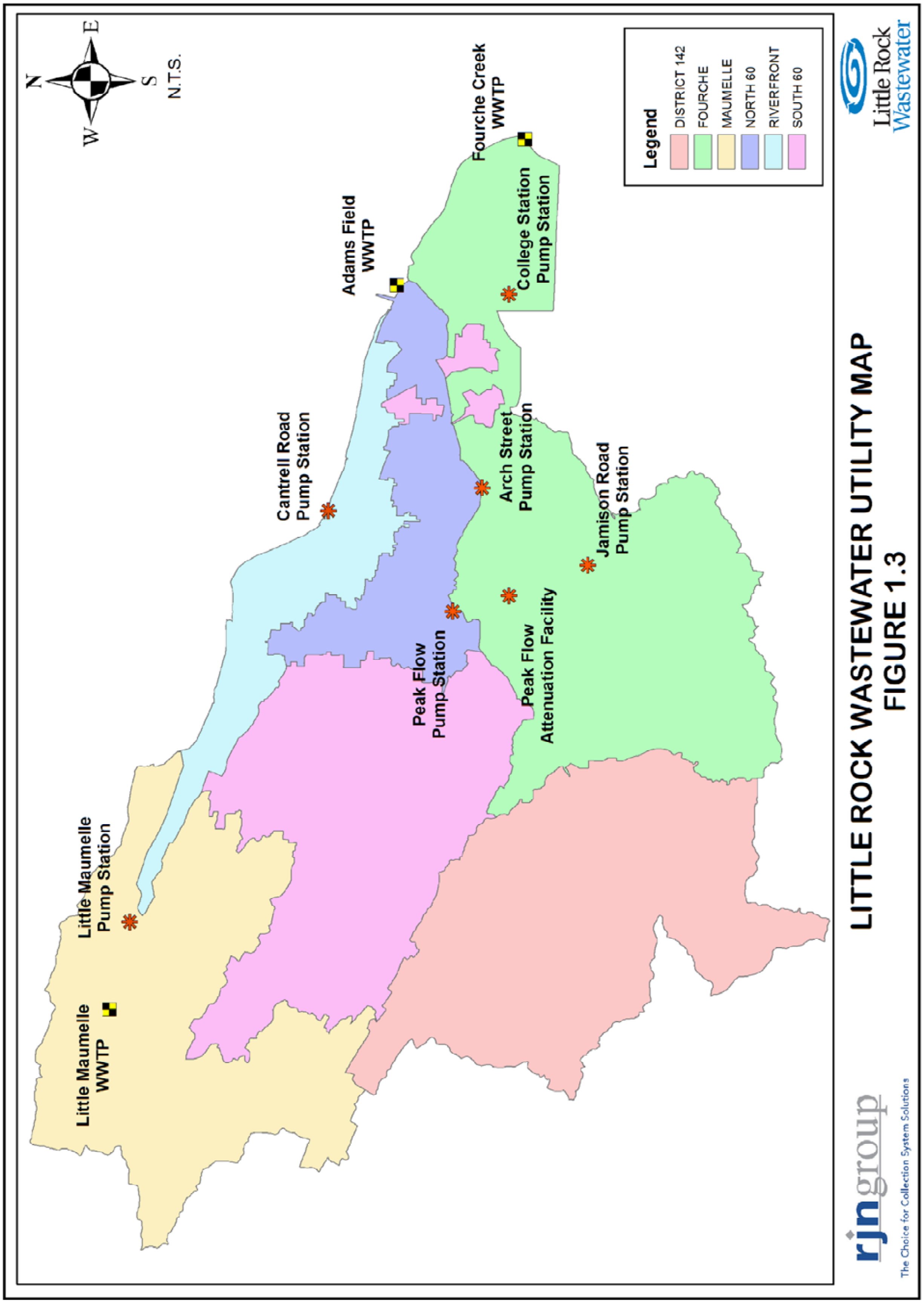
RJN Group, Inc. was authorized by LRW under an agreement dated December 1, 2009 to provide an update to the original System Evaluation and Capacity Assurance Plan conducted by Montgomery Watson Harza in 2002.

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

PROJECT ADMINISTRATION AND MANAGEMENT

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

Project Administration included completing a final schedule of work activities, meeting with LRW staff on a monthly basis to update previous investigative work and to coordinate upcoming tasks.



LITTLE ROCK WASTEWATER UTILITY MAP
FIGURE 1.3

Data Management involved review of existing information provided by Little Rock Wastewater including maps, flow and rainfall records, facility record drawings, pump curves, overflow occurrence records, Capital Improvement Project (CIP) status, existing hydraulic model structures, reports, zoning requirements, and all other pertinent information.

RJN conducted four workshops to establish methodologies for updating the SECAP document. These workshops were a valuable tool in gathering additional information from key LRW personnel and to keep everyone involved abreast of the progress and solutions derived during the study. The four workshops were conducted over a period of six months and were held at milestones of the project. These included model development, model calibration, evaluation of existing facilities, and alternative solution evaluations.

FLOW MONITORING

RJN reviewed the collection system maps, operational information for the collection system, and the hydraulic model network to select strategic flow monitoring locations. Monitoring locations from the City-wide flow monitoring completed in 2000, in conjunction with the original SECAP study, were used in as many locations as possible. An additional six meters were installed at locations that were not monitored during the 2000 study. The flow monitoring occurred over 109 days from October 2009 to February 2010.

In addition, eight (8) rain gauges were installed to supplement the twelve (12) permanent rain gauges that LRW owns and operates. The rainfall data was calibrated using radar images provided by the National Weather Service to provide 5-minute 1 km x 1 km data throughout the City of Little Rock.

The results from the flow and rainfall monitoring were analyzed to develop final calibrated data for each metered basin. Average daily dry and wet-weather flows were established for the hydraulic model. In addition, peak inflow and infiltration rates were established. A technical memorandum is provided with this report in Appendix A.

HYDRAULIC MODEL UPDATE

The existing hydraulic model that was provided to RJN contained line segments 10-inches in diameter and larger in the Wallingford Software InfoWorks. The model was updated to include all line segments within the collection system of LRW. Record drawings were obtained from LRW of all 10-inch and larger sewers constructed since 2002 and input into the model. Model data that had been previously studied by RJN and CDM was also incorporated into the model. Record Drawings of all flow equalization basins, lift stations, pump curves were reviewed and input into InfoWorks. Sub-Basins and catchments were reviewed and some were redefined for better representation in the model. Field verification, both by RJN and LRW staff, were performed to confirm elevations, pipe diameters, and grades of collection lines. 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, in conjunction with data stored on the LRW SCADA system. 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 three storm events with varying rainfall intensities that ideally do not result in overflows in the collection system
- Input radar rainfall adjusted data into model for selected events
- Calibrate model based on selected storm events

DEVELOPMENT OF FUTURE FLOWS

The 2002 SECAP Report was based on the assumption that all of the sewer basins except those tributary to the Little Maumelle WWTP were fully built-out and that no flow increases were anticipated due to population growth. The hydraulic analysis was completed utilizing the existing 2000 measured dry-weather flow. This update to the existing SECAP has made allowance for future flows by analyzing zoning data areas currently served by septic systems. Sewer service boundaries were compared to current aerial maps for future development. Population and land use maps were evaluated and a full build out model was developed for estimated future wastewater flows.

EVALUATION OF MAJOR LIFT STATIONS

The 2002 SECAP identified several major pump stations and force mains improvements to transport peak flows after construction of pipeline capacity improvements. The Arch Street Pump Station and force main improvements were under construction during this study period. The Cantrell Road Pump Station and Jamison Road Pump Station were evaluated during this task. Tasks included reviewing record drawings and pump curves for each station. Site visits at each station were performed to evaluate hydraulic, mechanical, and physical conditions. Interviews of LRW staff concerning operation modes and philosophy were conducted. Alternatives and cost estimates for upgrades were completed.

EVALUATION OF EXISTING PEAK FLOW EQUALIZATION FACILITIES

LRW has constructed and put into service two peak flow storage facilities since the submittal of the 2002 SECAP Report. One of these is a 13 million gallon (mg) basin located at the Adams Field WWTP and the other is a 30 million gallon facility located at 5200 Scott Hamilton Drive. The 30 mg facility includes a 10 mg and a 20 mg basin and was placed in service in August 2009. The facility also includes a 50 mgd pump station located at 3505 Mabelvale Pike. This task included evaluation of record drawings, pump station pump curves, and diversion structures related to the operation of the basins. Site visits were performed and parameters were studied of when the basins are put into service.

EVALUATION OF WASTEWATER TREATMENT PLANTS HYDRAULIC LOADS

An evaluation of the peak hydraulic capacity of Adams Field and Fourche Creek treatment facilities was completed. The evaluation did not include any process review with the focus directed to identifying any hydraulic bottlenecks that may impact overflows upstream in the system. This task included reviewing pertinent reports of the treatment plants, review of operating data for a range of flows and to implement different operating scenarios into the hydraulic model to assist with wet-weather flow durations (assess hydraulic performance during extended wet-weather flows).

ALTERNATIVE EVALUATION / SECAP UPDATE

The SECAP Update consists of several tasks to evaluate the existing system performance, and verify remaining improvement recommendations or identify alternative improvement projects to eliminate the wet-weather sanitary sewer overflows (SSO's) identified by the CAO. These tasks included utilizing the calibrated hydraulic model to determine the most cost effective and constructible solutions to bring LRW into compliance with the CAO and Settlement Agreement.

DEVELOPMENT OF SECAP CAPITAL IMPROVEMENT PLAN (CIP)

This task included developing specific capital improvement projects required to address modeled system deficiencies and future capacity requirements. These projects have been prioritized and correlated to specific overflow locations.

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, 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 capital improvement plan including model data, breakdown of individual projects with accompanying cost estimates, and technical memoranda for the flow monitoring and model analysis.

CHAPTER 2

EXISTING AND FUTURE FLOWS

This chapter presents the flow monitoring data and how it was used in the model development. It also provides a summary of the flow monitoring data for calibration of the model, inflow / infiltration (I/I) analysis, development of dry and wet-weather flows, and the design rainfall event used to assess capacities and solutions.

FLOW MONITORING SUMMARY

Temporary flow monitoring was performed for a period of 109 days from October 22, 2009 to February 08, 2010 within the collection system of Little Rock, AR. The objective of the flow monitoring was to collect dry and wet-weather sewer flows for model calibration and to measure I/I quantities for each metered basin.

RJN installed 69 gravity flow meters and 8 rain gauges to supplement 12 LRW gauges throughout the study area. Flow monitoring locations were chosen based upon the previous city wide flow monitoring study performed in 2000. Flow site locations from the previous 2000 study were considered based on suitable hydraulics and installation conditions. Locations found unsuitable for monitoring were adjusted. An additional six (6) flow meters were installed providing a more accurate representation of the sewer system flow for modeling purposes. An area map denoting the locations of the flow meters and rain gauges, plus site sheets depicting each location is provided in Appendix A.

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 sixty-nine (69) basins. The dry-weather period selected for this analysis was from November 14, 2009 through November 21, 2009. The analysis determined that the average daily dry-weather flow during the monitoring period was approximately 34.5 mgd with 1.8 mgd tributary to the new Little Maumelle Treatment Plant and 32.7 mgd tributary to the Adams Field and Fourche Creek Treatment Plants. It should be noted that during the monitoring period, this week was the best suited for dry-weather analysis. However, unusually long periods of rainfall in 2008 and into 2009 most likely led to higher than normal groundwater conditions, even during a period with no rain. 2009 was actually the wettest year on record for the Little Rock area.

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 1.73. Peaking factors varied from a minimum of 1.27 to a maximum of 3.13. These are given in both table and graphical format located in Appendix A.

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 high amounts of rainfall that occurred during the study, it was determined that groundwater conditions were favorable to determine peak infiltration conditions during the monitoring period.

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 eighteen (18) basins exhibited significant infiltration with rates in excess of 5,000 gpd/idm. The system resulted in a total peak infiltration rate of 36.7 mgd. This is comprised of 2.0 mgd from the Little Maumelle Sewershed and 34.7 from the Adams Field and Fourche Creek Sewersheds. A more detailed explanation and individual basin results are provided in Appendix A.

Inflow Conditions

Flow data during wet-weather periods was analyzed to determine peak inflow originating in each basin. To determine the peak inflow rate, the calibrated model was used to perform the analysis. By isolating each basin's inflow component and treating its outfall as free flow, the peak design inflow rates were calculated. Several storm events of various intensities were used to calibrate the model. This ensures that the model has appropriately distributed inflow.

The analysis projected the peak 1-year storm inflow (1.55 inches/hour) rate to be 263.4 mgd with 11.4 mgd generated in the Little Maumelle Sewershed and 252.0 mgd in the Adams Field and Fourche Creek Sewersheds. The system overall exhibited severe inflow with rates exceeding 10,000 gpd/linear foot for the 1-year/60-minute inflow. A summary of the projected peak wet-weather flow rates during a 1-year/60-minute storm event is given in Table A-7 and is shown graphically on page A-27 of Appendix A.

Observed Rainfall Data

The rainfall data used for the SECAP update is gauge-adjusted NEXRAD radar-rainfall data provided by CALAMAR. Radar rainfall data provides an accurate account of the spatial distribution of rainfall which is critical to model calibration. In the past, hydraulic models have been calibrated using rainfall data collected from rain gauge networks providing accurate rain measurements at discrete points, but with sub-standard estimates falling between gauges. Conversely, radar is able to see between the gauges but lacks the consistency in estimating rainfall at a specific point. By calibrating the radar images with the rain gauges on the ground, an accurate estimation of rainfall was defined for every 1 km X 1 km pixel across the service area of LRW. Figure 2.1 shows the spatial variation of total rainfall during a rain event in early December.

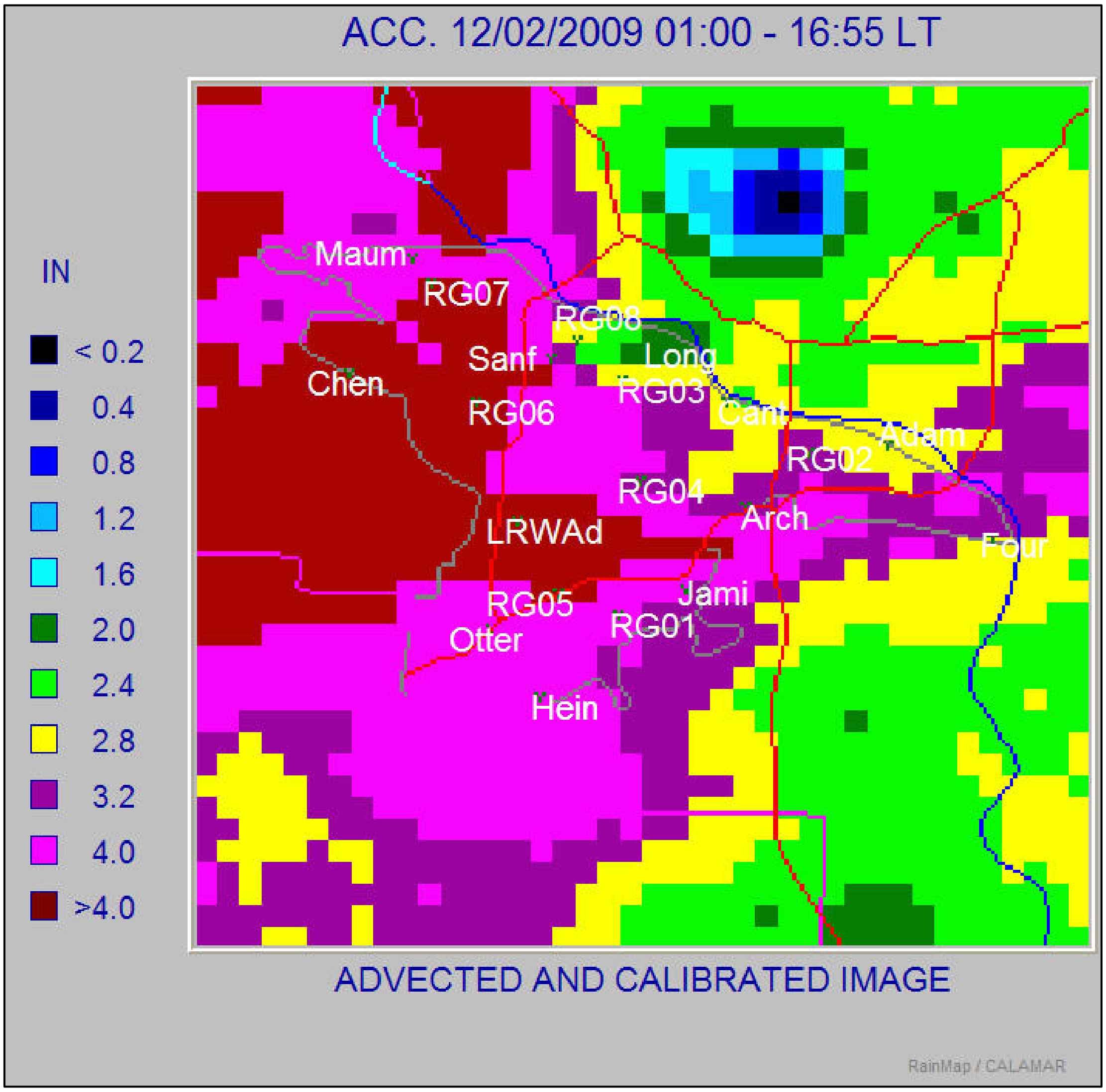


Figure 2.1: Calibrated Radar Rainfall

The rain gauge network on the ground consisted of twelve (12) permanent gauges owned and operated by LRW and an additional eight (8) temporary gauges installed by RJN. During the monitoring period a significant number of rainfall events occurred. Seven rain events occurred with intensities greater than 0.25 inches/hour. The rainfall total during the study period was 27.5 inches.

PEAK FLOW RATES

The total peak hour wet-weather flow projected during a 1-year storm event is approximately 356.5 mgd. This consists of 53.7 mgd of peak hourly dry-weather flow, 36.7 mgd of peak infiltration and 263.4 mgd of inflow. Based on an average daily dry-weather flow of 34.5 mgd, this would result in a wet-weather peaking factor of 10.3. Table 2-1 summarizes the flows for the Little Maumelle, Adams Field and Fourche Creek Sewersheds. Peaking factors varied from 3.0 to 59.3 and are given for each basin in Table A-8 and shown graphically on page A-32 in Appendix A.

Table 2-1

TOTAL PEAK HOUR 1-YEAR/60-MINUTE WET-WEATHER FLOW 2009

Basin	Basin Peak Hourly Flow Rate (mgd)	Basin Peak Monitored Infiltration (mgd) 12/24 Storm	Basin Peak 1-Year/60-Minute Inflow (mgd)	Basin Peak Wet-Weather Flow (mgd)	Wet-Weather Peaking Factor
Little Maumelle Service Area					
Subtotal	3.491	1.987	11.378	16.856	9.272
Adams Field & Fourche Creek Service Areas					
Subtotal	50.172	34.713	251.980	336.865	10.302
Total	**53.663**	36.700	**263.358**	**353.721**	**10.245**

EXISTING MODEL FLOWS

The hydraulic sewer model requires dry-weather and wet-weather flow analysis to evaluate the hydraulic performance of the existing sewer system. Sewer flows are generated from residential populations, commercial and industrial flows, groundwater infiltration and rainfall induced inflow/infiltration.

RESIDENTIAL FLOWS

Population data is critical to generate dry-weather flows from residential areas. Residential flows were developed through a multi-stage process using 2000 US Census block data and the property coverage provided by the city.

The first stage of the process involved estimating the number of residential housing units in each property based upon the structure code. Each property was then referenced to the Census Block in which it resided. The number of residential housing units within each Census Block was then summarized and compared with the number of housing units determined in the 2000 Census. The population density per household was then calculated for each Census Block using the 2000 and 2009 housing unit estimates.

In general, each property was assigned a population based on the 2009 housing unit density, to ensure that the total population within each Census Block was uniformly distributed.

Investigations were undertaken on Census Blocks where there was a large discrepancy between the number of properties between 2000 and the present. The majority of these Blocks were in the west region of the City, where it was apparent by the age of the sewers that significant development had occurred since the 2000 Census and, in some cases; major apartment blocks had been constructed. In these Census Blocks the properties were assigned the population density based upon the 2009 housing unit numbers which resulted in an increased population estimate for the Census Block from 2000 numbers.

The modeled sub-catchments were superimposed over the property coverage and the populations summarized for each sewer sub-catchment, resulting in an estimated total population served by the sewerage system of approximately 206,000 people.

Each subcatchment was assigned an appropriate per capita flow rate and profile as described below and calibrated to the nearest downstream flow monitor. This ensured any errors in population estimation and distribution would only have a minor impact within a given flow monitored catchment and would not affect the system as a whole

Using the flow data collected from each meter during a dry period in November 2009, weekday and weekend average hydrographs were calculated and graphed for each flow monitor with a primarily residential catchment. Weekday and weekend dimensionless diurnal profiles were developed through a process of groundwater subtraction and normalization. From this process, 9 unique residential flow profiles were created. These profiles were input into the model and used to modulate dry-weather flows. Figure 2.2 graphically depicts the 9 residential flow profiles used for the SECAP update.

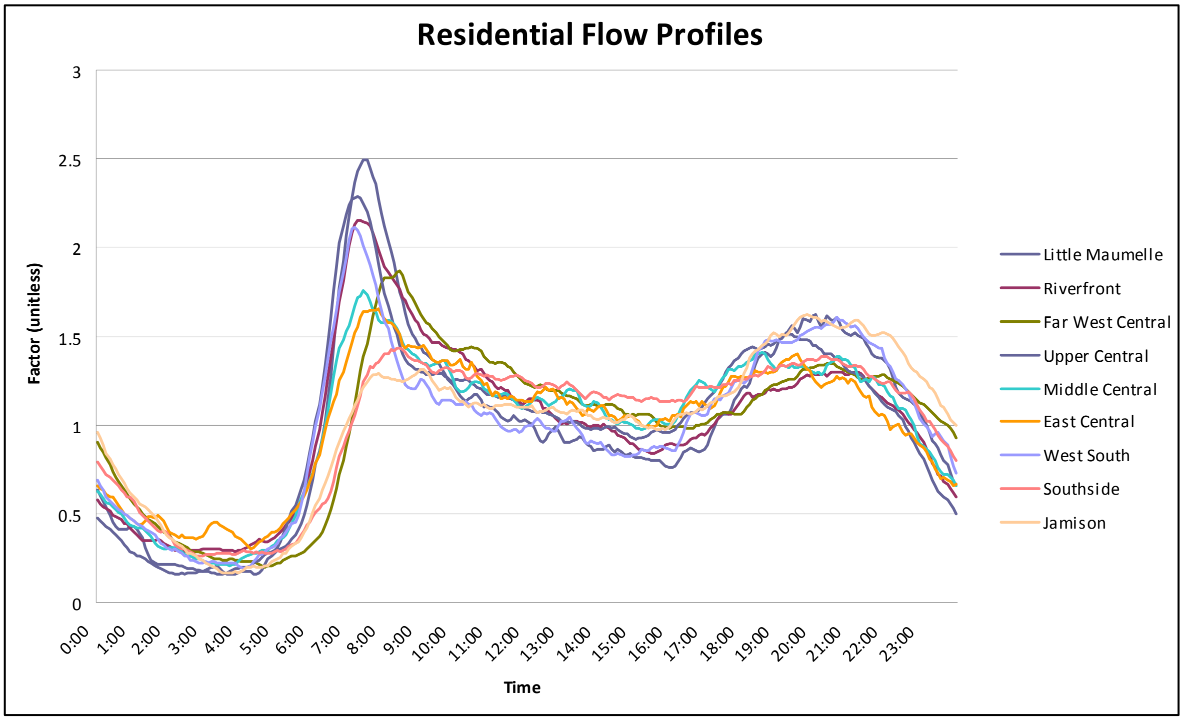


Figure 2.2: Residential Flow Profiles

COMMERCIAL AND INDUSTRIAL FLOWS

Commercial and industrial areas produce unique flow patterns that are dissimilar to residential areas. The flow profiles used for commercial and industrial areas were previously determined during the detailed Sanitary Sewer Evaluation Studies conducted by RJN on the Little Rock sewer system. These profiles, as well as a set of standard profiles, were input into the model. The profiles were assigned based on the predominant type of business or industry in each sub-catchment, as determined during model construction. Figure 2.3 graphically depicts both commercial and industrial flow profiles used for this project.

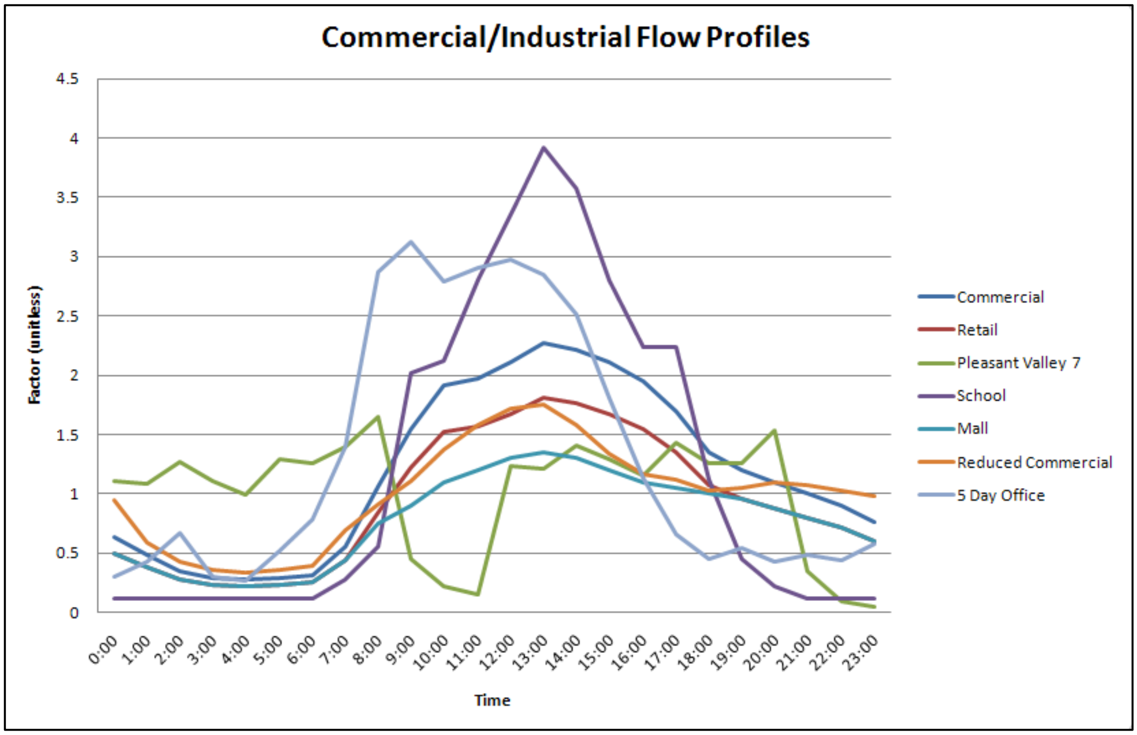


Figure 2.3: Commercial / Industrial Flow Profiles

FUTURE FLOWS

SYSTEM GROWTH

The City of Little Rock is projected to have significant growth to the west of the current city extensions. Maps were provided by Little Rock Wastewater showing the future city limits and proposed zoning. These maps were analyzed in combination with terrain maps to determine future development areas and their drainage paths.

Once the development areas were determined, the data was digitized into model sub-catchments and assigned population values by RJN. Housing density was assumed to be 4 houses per acre in residential areas, and 1 house per acre if located on a hillside that could be developed. A population of four per household was estimated. The total buildout population in the future growth areas was estimated to be approximately 13,000 people. Future commercial and industrial flows were also assigned to the sub-catchments based on proposed zoning. Rainfall runoff and groundwater infiltration values for the future sub-catchments were adjusted to produce a wet-weather peaking factor of 3.

Sewer mains were placed in the model following the natural terrain and drainage paths for the future areas. These mains were connected to the existing sewer system at logical connection points. The lines were then sized using the design storm to allow free flow conditions during peak wet-weather events. Further explanation of future growth projections as well as a map outlining these areas, are included in Appendix B.

DESIGN STORM

The design storm used for the model analysis was provided by Little Rock Wastewater. This storm is a recorded event that occurred in November 2000 under the original SECAP program. The 48-hour event was recorded in 2x2 km pixels by a NEXRAD system. The average rainfall of 4.15 inches is similar to a 2-year/48-hour storm event for the region. The November 2000 rainfall event equates to a design event with a return period between two and five years. During the initial SECAP, the November 2000 event was selected because the rainfall event:

- Exceeds LRW design criteria
- Provides a realistic spatial distribution of rainfall
- Coincides with reported hydraulic wet-weather overflows
- Was used for confirming model calibration with the permanent flow meters available in 2000
- Occurred after an unusually long period of rainfall, giving rise to saturated soils, high groundwater infiltration and therefore creating a worst-case scenario
- Had available rainfall data providing an accurate spatial representation of rainfall depths.

Figure 2.4 shows the design rainfall event hyetograph.

For additional information regarding the design storm and parameters used in developing the design flows please refer to Appendix B.

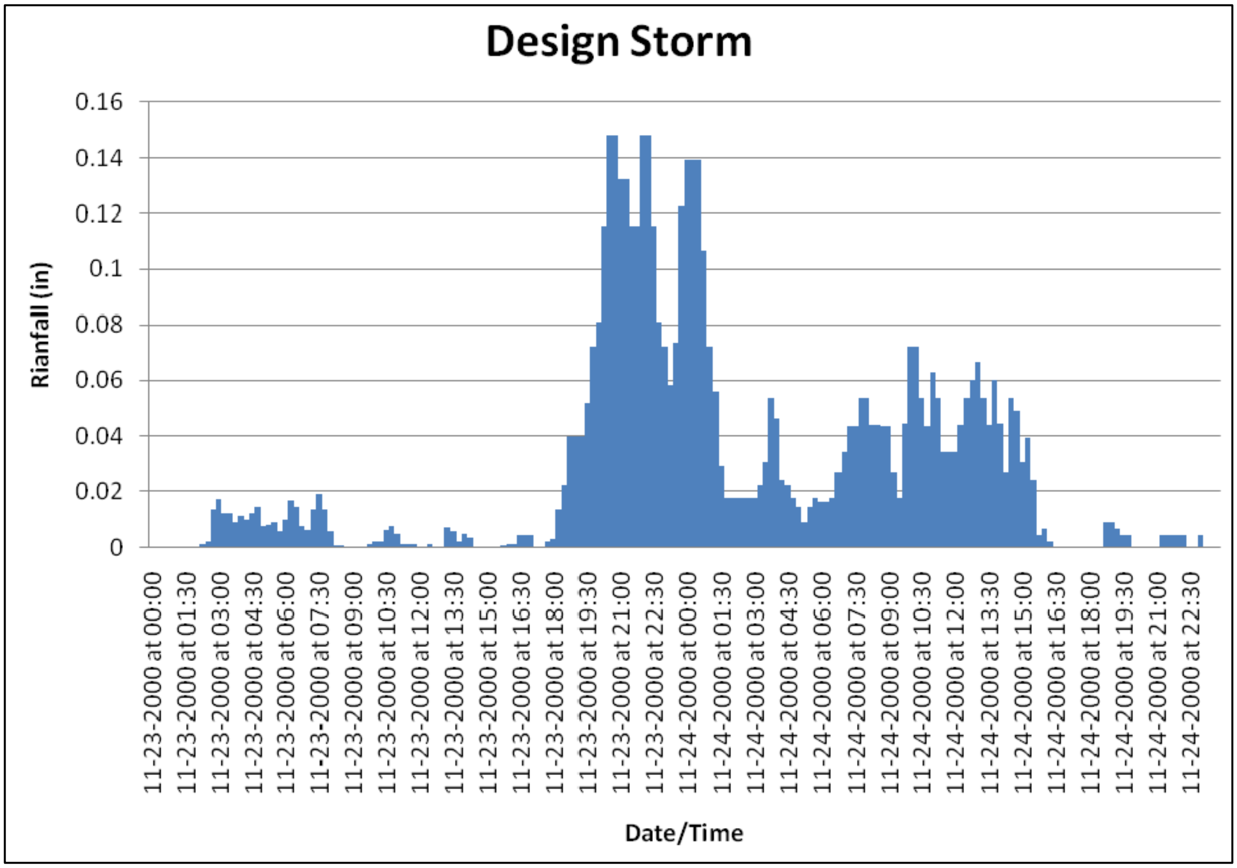


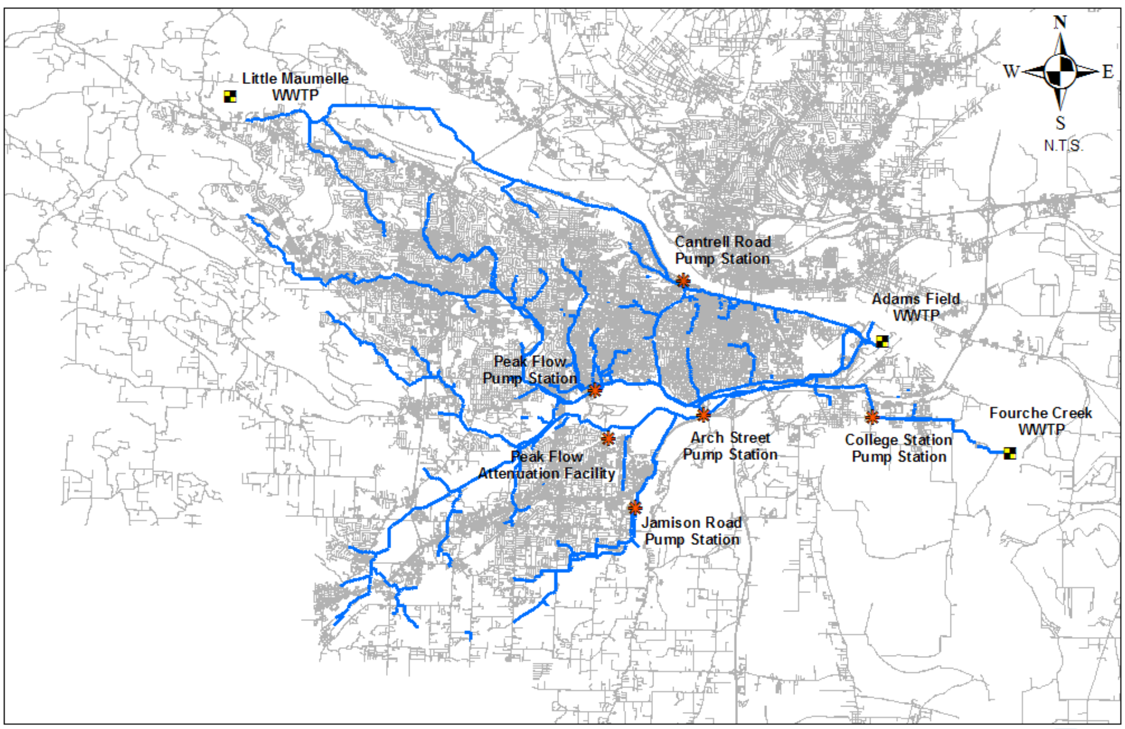
Figure 2.4: Design Storm Hyetograph

CHAPTER 3

FACILITY EVALUATION

BACKGROUND

The overall evaluation of the Little Rock Wastewater Collection System and validation of the SECAP included an assessment of the condition and operations of the Arch Street, Cantrell Road and Jamison Road Pump Stations and the Peak Flow EQ basin pump station. Also included was a review of the hydraulic and capacity conditions at the Adams Field and Fourche Creek Wastewater Treatment Plants (WWTP). A map showing the locations of the facilities can be seen below in Figure 3.1.



LITTLE ROCK WASTEWATER FACILITY MAP
FIGURE 3.1

This chapter discusses the physical condition of the pump stations and hydraulic conditions of the WWTPs. Operational modifications such as "pump down" of the collection system prior to storm events, capacity improvements, and pumping to storage facilities are discussed in other sections of this report.

The facility evaluation included the collection of record drawings and pump curves for each pump station. Site visits were undertaken to each station to evaluate the hydraulic, mechanical and physical conditions and to observe operations. Interviews were conducted with LRW staff concerning operation modes and philosophy. Back-up power requirements were also reviewed.

The following sections describe the evaluation of each facility.

PUMP STATION EVALUATION

ARCH STREET PUMP STATION

This station was undergoing expansion and modification during the project period and no evaluation was conducted. The design flow rates and new pump curve data were used in the hydraulic model.

CANTRELL ROAD PUMP STATION

The Cantrell Road Pump Station was constructed in 1967 and was modified with bar screens and two dry-pit submersible Flygt pumps in 1986.

Overall, the wet well, dry well and building structure are in good condition. Two of the four pumps are original pumps from when the station was constructed in 1967. The other two pumps are replacement pumps that were installed in 1986. Two bar screens were also installed in 1986. A portion of the switch gear is original while some components were replaced or added in 1986. Dry-weather flow at the station ranges between 3.5 mgd and 6.6 mgd. Peak pumping capacity is approximately 32 mgd.

The pump station does not have installed back-up power. LRW has relied on the availability of power from the electric grid shared with the State Capital.

Conclusions:

Mechanically and electrically, the pump station components are in need of replacement. With portions of the equipment at 43 years old and the remaining at 24 years, the reliability of equipment and availability of replacement parts have and will be an issue. Recommendations are to:

1. Replace the four pumps. Recommend replacing the four pumps with 5 pumps. Size three pumps to handle wet-weather flow with a 2 + 1 arrangement. Size two pumps to handle dry-weather flow with a 1+1 arrangement.
2. Replace all of the electrical gear. Pumps will be on VFD drives.
3. Replace existing bar screens with Headworks® type Bar screens. These would be installed in a new wet well.
4. A concern (beyond the scope of this evaluation) is how reliant is LRW on the constant availability of power from the electric grid. Does LRW want to continue with no on-site back-up power?
5. Construct a new force main and inspect/rehabilitate existing force main.

The recommended upgrades will improve the reliability and efficiency of the station.

A detailed estimate of construction cost to implement the recommendations is included in Chapter 5 and Appendix C.

JAMISON ROAD PUMP STATION:

The Jamison Pump Station was constructed in 1993. Overall the wet well, valve vault and building structure are in good condition. There are areas that need painting.

The station consists of five submersible pumps, which include two 25 hp and three 150 hp. There are two grinders and screens; one on each of the inlet channels. Dry-weather flow at the station is approximately 2 mgd. Peak pumping capacity is approximately 16 mgd.

The pump station does not have installed back-up power.

Conclusions:

The pump station is functioning as designed. No changes are immediately required at the station at this time. Recommendations are to:

1. Consider Installation of back-up power. ($750,000)
2. Surface clean and paint ferrous surfaces ($50,000)
3. Grinders are 17 years old. When maintenance becomes an issue, consider replacing with a mechanical bar screen to remove trash from the influent. ($650,000)

FLOW EQUALIZATION BASINS

The flow equalization pump station was placed in service in 2009. Recommendations to expand this facility are discussed in Chapter 5 of this report.

WASTEWATER TREATMENT PLANTS

ADAMS FIELD WWTP

The plant's hydraulic rated conditions are as follows:

Adams Field WWTP has a head works with lift pumps located at the plant. The WWTP is NPDES permitted for a design flow of 36 mgd and is allowed to bypass secondary treatment at flows in excess of 60 mgd. The lift pumps have a capacity of approximately 94 mgd.

Using 10 States Standards as the design guide; based on primary clarification of 1,000 gpd/sq-ft and a 2,000 gpd/sq-ft peak design flow; the primary clarifiers are sized for an average daily flow of 31.2 mgd and a peak design flow of 62.4 mgd.

Based on 40 lbBOD/day/1,000 cubic feet, aeration has the capacity to treat 47 mg/l BOD at 60 mgd and 31 mg/l BOD at 90 mgd. 10 States Standards limits are 40 mg/l. There is additional aeration capacity available that could handle up to 70 mgd.

The UV disinfection unit is sized for 72 mgd.

The plant has successfully passed up to 90 mgd. The NPDES permit places no restrictions on peak flow over 60 mgd as long as the discharge continues to meet permit limits. However, to exceed 60 mgd the plant is blending. USEPA is considering regulation changes that may prevent blending in the future. Blending is defined as wastewater that does not receive secondary treatment mixing with wastewater that has received secondary treatment.

The most economical capacity increase for the WWTP would only be an additional 10 mgd. But even this increase would require an additional primary and secondary clarifier and hydraulic structure changes. The incremental cost to add additional storage capacity is less than the cost to add additional treatment capacity.

There are two hydraulic restrictions within the plant, JB-4 and JB-6. JB-4 is used to control flow to the EQ basin, the aeration tanks, and the 60-inch bypass. Because of either box configuration or weir wall heights, or both, the ability to control the flows into and out of JB-4 is limited. A plan view of JB-4 is shown in Figure 3.2

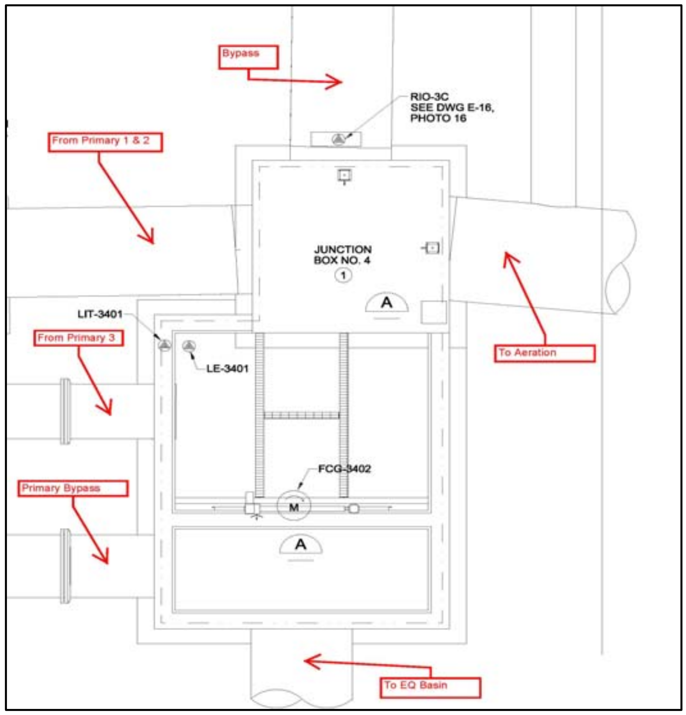


Figure 3.2: Plan view of JB-4

JB-6, also referred to as the Octagon Box, is limited to approximately 60 mgd. Flows over 60 mgd causes the box to surcharge. This restriction limits the amount of wastewater that could flow through the secondary clarifiers. A plan view of JB-6 is shown in Figure 3.3.

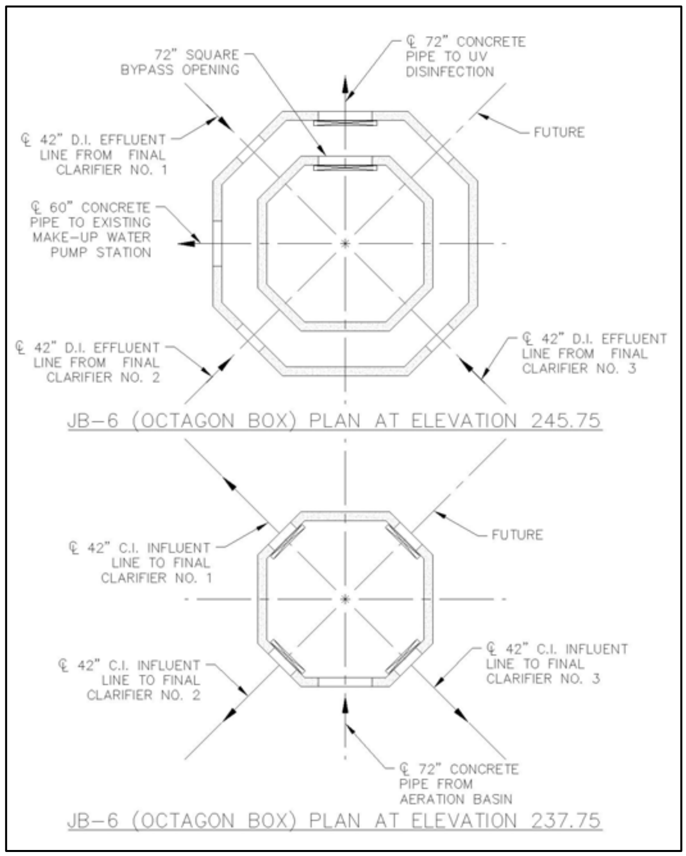


Figure 3.3: Plan view of JB-6

A process condition that also would impact the hydraulic performance is the amount of return activated sludge (RAS) that can be re-circulated through the secondary clarifiers. The RAS pumps are sized for about 20 percent recycle. These pumps should be sized for return rates of up to 100 percent recycle. However, doing this will also complicate the hydraulic conditions at JB-6 and reduce the hydraulic capacity unless modifications are made. The benefits would be better control of the activated sludge population during peak flow events.

It is recommended that modifications be made to both boxes to alleviate these hydraulic restrictions. The estimated cost for these modifications is approximately $480,000.

FOURCHE CREEK WWTP

Fourche Creek WWTP does not have a pump station at the plant. All of its flows are through a force main coming from the College Station Pump Station and other small pump stations located between College Station and the WWTP. Upon completion of the Arch Street Pump Station improvements, the existing College Station Pump Station will be removed from service and a smaller station to convey flow from local services will be constructed. It is anticipated the new station will be placed in service in early 2013. There are three other small pump stations feeding into this same force main.

Fourche Creek WWTP has begun improvements and expansion work that will increase the peak flow rate to 45 mgd. Future planned improvements will increase the capacity to 52 mgd. The current NPDES permit lists the plant as having a design flow of 16 mgd. There are no provisions for bypassing or limitations for peak flows.

The plant can only handle peak flows for three days. Beyond three days affects the biology and washes out the plant.

A complete assessment of the plant was limited due to pending modifications.

PUMP STATION AND FLOW EQUALIZATION BASIN OPERATION PARAMETERS

The impact of existing operating parameters and proposed future operation parameters are discussed in Chapter 4 and Chapter 5.

CHAPTER 4

MODEL DEVELOPMENT AND CAPACITY ANALYSIS

This Chapter provides a summary of the model development, calibration, and capacity analysis. A detailed description of these tasks is provided in Appendix B.

MODEL DESCRIPTION

In order to analyze the performance of the Little Rock Wastewater sanitary sewer system, the existing computer based hydraulic model was updated and expanded. The Infoworks CS software by MWHSoft, Inc. 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 hydraulic model from the 2002 SECAP including all auxiliary facilities
- Calibrate the model to reflect current recorded flows and surcharge depth data
- Evaluate existing system capacity to transport dry and wet weather flows
- Simulate a 2-year/48-hour design storm on the calibrated system model 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

Little Rock Wastewater provided RJN Group, Inc with the hydraulic model used in the 2002 SECAP report. The model, which was constructed from record drawings and available Geographical Information System (GIS) data, consisted of sewer mains 10 inches and larger in diameter with built in storage compensation for un-modeled mains.

As part of the scope of this project, RJN was to incorporate any new sewer mains 10 inches and larger in diameter constructed since 2002 and add selected 8-inch diameter mains up to reported overflow locations. During this process the 2002 model was compared with the current LRW GIS. This comparison determined there were more changes and additions than anticipated.

Because of the large amount of network changes from 2002 to the present, it was determined that the best course of action would be to construct the new model geometry based upon the current GIS database augmented with pipe attribute data from the 2002 model as appropriate. This approach provided a more accurate system geometry than incorporating current changes into the 2002 model.

The updated model reflecting 2009 conditions was then verified for proper slopes and connectivity using the validation tools built into Infoworks. 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.

PUMPING STATIONS

The updated hydraulic model includes all of the major pumping stations located within the sanitary sewer system. The pump stations modeled include Adams Field MPS, Arch Street PS, Cantrell Road PS, College Station PS, Jamison Road PS, Little Maumelle PS and the Peak Flow PS. The existing College Station PS was not included in the updated model since it is scheduled to be replaced with a smaller station.

Pump station geometry was entered into the model from record drawings provided by Little Rock Wastewater including wet wells, influent chambers, and gates. Pump curves supplied by LRW were also used for all pumps. Pump control levels were extracted from the Little Rock Wastewater SCADA system.

During the flow monitoring period, the Little Maumelle PS was being re-constructed and a temporary pump station was installed to maintain flow from the Little Maumelle basin. Little Rock Wastewater provided the information for the temporary pumps installed and the model was updated to reflect this correct configuration during the monitoring period.

PEAK FLOW ATTENUATION SYSTEM

The Little Rock sanitary sewer system contains a 30 million gallon (mg) offline storage facility that stores flow during wet weather events. This facility known as the Peak Flow Attenuation Facility includes a 10 mg and a 20 mg basin which was incorporated into the model. The attenuation facility is fed from the Peak Flow Pump Station which receives flow from two diversion structures that relieve the North 60 and South 60 trunk sewers. Once the system activates, the flow is pumped to the storage facility where it is detained until being released back into the Fourche Creek Interceptor sewer via the Fourche Creek Interceptor diversion valve, once capacity becomes available following a storm event.

The model contains the peak flow pump station and force main, the north and south diversion structures, the Fourche Creek diversion valve, the attenuation basins and their associated grit chamber. The information for each of the facilities was input in the model from record drawings. The controls for each of the peak flow facilities were derived from the SCADA system.

MODEL CALIBRATION

PROCESS

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

DRY-WEATHER

Dry-weather calibration ideally requires at least a 7-day period, including one weekend, unaffected by rainfall induced flows. The recorded flow data was assessed in conjunction with the rainfall data and the period from November 11, 2009 through November 20, 2009 was selected as a representative dry-weather period.

Calibrating the model for dry-weather flow was achieved by modifying:

- Permanent groundwater infiltration rates
- Per capita flow rates
- Commercial / 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: 36,600 Acres
- Average per capita flow: 63 gal/day
- Total Residential wastewater flow: 12.8 MG
- Commercial/Industrial flow: 3.6 MG *(Does not include port area)*
- Permanent groundwater (dry-weather) infiltration: 6.4 MG
- Total daily dry-weather flow: 22.8 MG

WET-WEATHER

Review of the wet-weather response to rainfall indicated that there is a significant amount of inflow and infiltration throughout the sewer system. Once the correct antecedent groundwater conditions were established, all modeled storm events produced consistent runoff and were used for 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 stormwater cross connections.

- Rainfall induced infiltration was modeled using hydrology in the Ground Infiltration Module (GIM) 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, taking into account both preceding rainfall and evaporation. These flows represent the delayed ingress of storm water through the ground into the sewer system through cracks and leaks in sewers and private drains.

During the calibration process, peak flows, infiltration time, depth, surcharge time, and velocity was compared to all metered sites during rain events. The storm event occurring on December 24, 2009 was excluded from calibration due to the rainfall depth being in excess of a 100 year storm event for the region.

Data recorded by the Little Rock Wastewater SCADA system was used in conjunction with the metered flow data to confirm depth and flow at pump stations, the Peak Flow Attenuation system, and Adams Field WWTP during the wet-weather events.

At the Peak Flow Attenuation Facility, the SCADA data also enabled calibration of the filling and draining processes that occurred during the study.

EXISTING SYSTEM ANALYSIS

SYSTEM GEOMETRY

During calibration, several areas of concern were identified due to irregular flow patterns. The following are some of the more pertinent issues that were verified through field investigations:

- Manhole 2O024: Sanitary sewer aerial crossing located in Hindman Park is buckled upward due to debris from the creek. (Figure 4.1)

Figure 4.1: Hindman Park Aerial Crossing

- Manhole 4L013: Box type crossing underneath creek is filled with silt and debris. (Figure 4.2)

Figure 4.2: Manhole 4L013 Junction Box Crossing

- 119 manhole cover elevations were re-surveyed by Little Rock Wastewater at the request of RJN. This elevation data was entered into the model to reflect an accurate representation of the collection system.

SYSTEM PERFORMANCE

The silt and debris found in MH 4L013 was of particular interest because it verified the atypical behavior predicted in this area. During rain events, the South 60 interceptor surcharges very quickly and causes the Brodie Creek trunk main to reverse flow. This main connects to the South 60 downstream of MH 4L013. The flow reversal accounts for loss in velocity and the ability to transport sediment through the box structure. This reversal of flow significantly affects the system during wet weather events.

In dry weather conditions, flow from Rock Creek and the majority of Brodie Creek flows to the North 60 and South 60 interceptors. The remaining flow from Brodie Creek is conveyed to the Fourche Creek interceptor. In addition, there are several high level bypasses between the parallel sections of the Brodie Creek and Fourche interceptors that allow flow to transfer between the two. Overall, there are no hydraulic issues in the system under dry weather conditions.

In wet weather conditions the pattern is altered. Once the North 60 and South 60 interceptors surcharge, a hydraulic restriction is created at the junction of the two systems. As rainfall continues, discharge from Rock Creek is forced upstream through the main from Brodie Creek causing a reverse flow. This flow continues upstream until it spills across the high level bypass into the Fourche Creek interceptor. The activation of the Peak Flow Attenuation system in its current configuration has insufficient impact upstream to prevent the flow reversal occurrence.

INFLOW AND INFILTRATION

Several of the metered basins experience high inflow rates (above 10,000 gpd/1,000 lf of pipe) and infiltration rates (above 5,000 gpd/idm) which were observed during flow monitoring. Figure 4.3 is an example of one basin with high inflow rates.

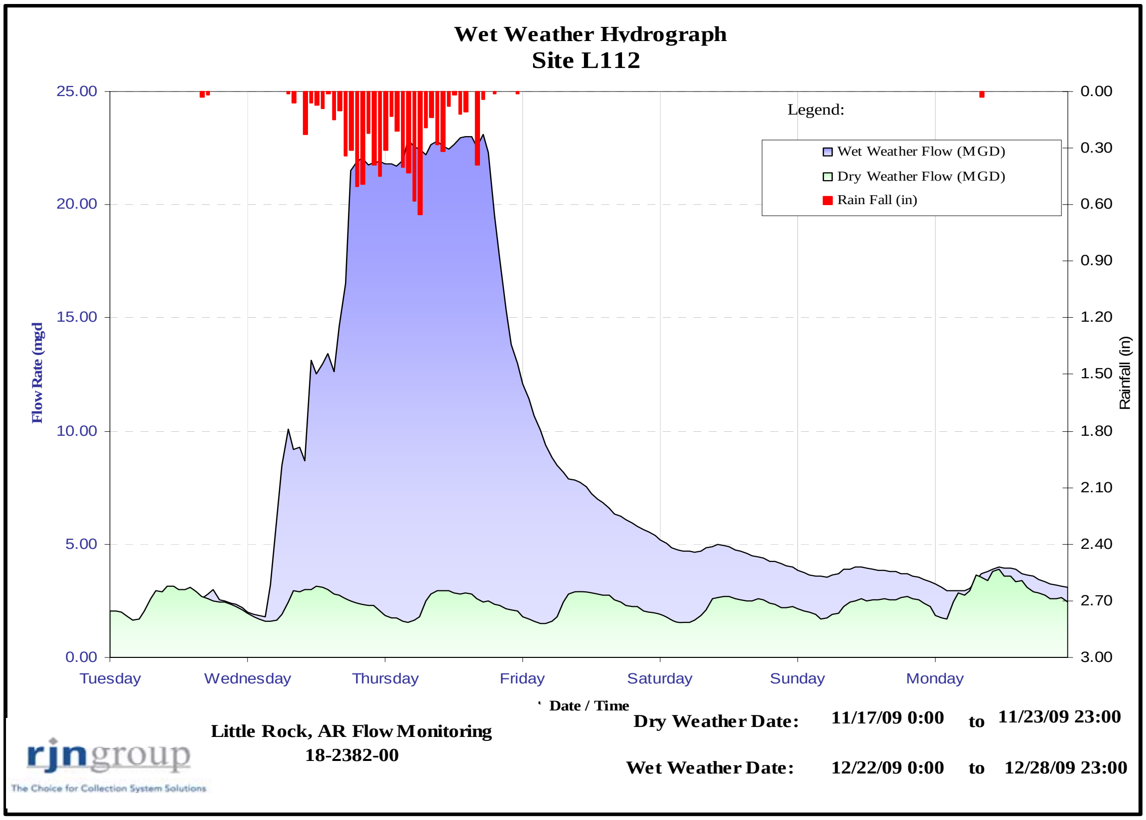


Figure 4.3: High inflow/infiltration observed at Meter L112.

OVERFLOWS

Several overflows were predicted to be extremely large and required verification by field staff. The field investigations confirmed that the overflows at these sites are extreme in volume when they occur due to heavy erosion plumes around the manholes.

Figure 4.4 is an example of erosion occurring at MH 3K059. The locations of recorded overflows are shown on Figure 4.5.

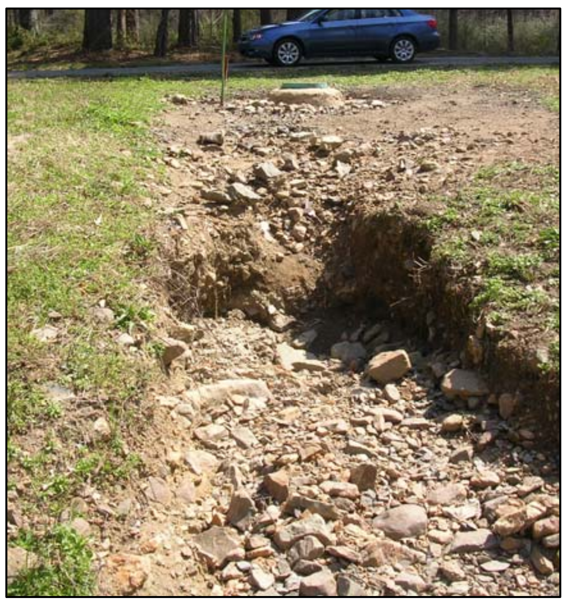
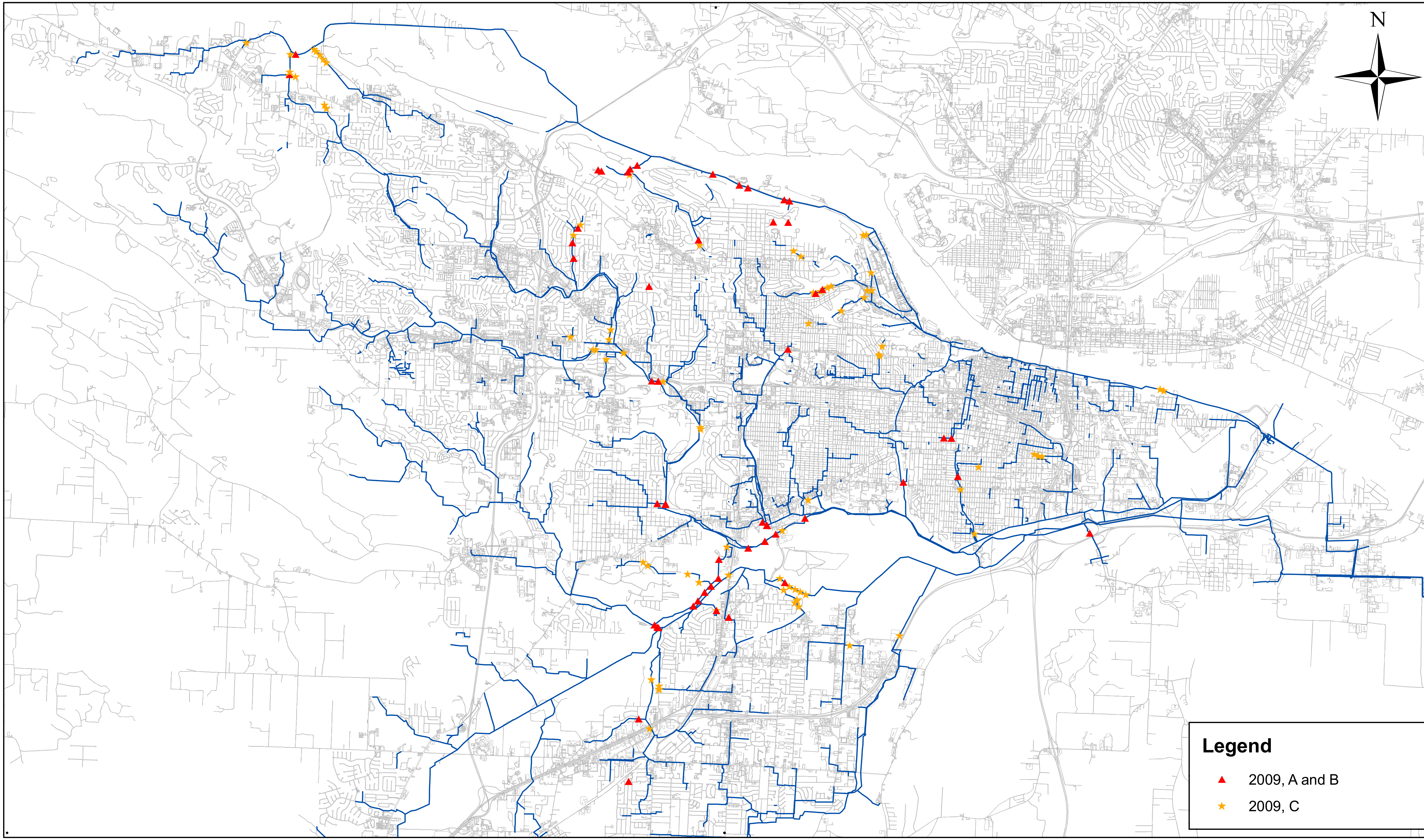


Figure 4.4: Overflow erosion at MH 3K059

Recorded Overflows



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**LITTLE ROCK WASTEWATER
SECAP UPDATE
FIGURE 4.5**

SYSTEM UPDATES

In order to analyze the system and identify capacity improvements under the design storm, the model was updated to reflect existing planned improvements. The updates include:

- Recent construction in Jimmerson and Allsopp basins
- Sewer mains that have been designed, but not constructed
- Capacity improvement recommendations from previous SSES projects completed by RJN Group, Inc.
- Arch Street Pump Station upgrades with the additional force main
- Abandonment of existing College Station Pump Station
- Removal of Little Maumelle sub-system due to new WWTP construction

OPERATIONAL ASSUMPTIONS

In addition to the geometry updates, assumptions were made on how the system can operate in wet weather conditions. All assumptions were verified with Little Rock Wastewater staff prior to the modeling of the design storm.

- Adams Field MPS – 94 mgd capacity
- Adams Field WWTP – 60 mgd treatment capacity + 34 mgd to storage (13 mg total volume)
- Arch Street PS – 45 mgd design capacity with dual force mains
- Fourche Creek WWTP – 45 mgd treatment capacity after current improvements, 52 mgd after future planned improvements
- Peak Flow PS – 68 mgd with additional pump

The Adams Field MPS can sustain 94 mgd for approximately 12 hours. This breaks down as 60 mgd being treated and 34 mgd (13 mg volume) conveyed to the Adams Field storage basin. After the storage basin is full, the MPS needs to be cut back to the peak treatment rate of 60 mgd. The current practice of blending at a 72 mgd rate was not incorporated into the model due to potential changes by USEPA.

DESIGN STORM

The design storm used for the model analysis was provided by Little Rock Wastewater. This storm is a recorded event that occurred in November 2000. The 48-hour event was recorded in 2x2 km pixels by a NEXRAD system. The average rainfall of 4.15 inches is similar to a 2-year/48-hour storm event for the region. The model subcatchments were updated to reflect the new pixel size and rainfall profiles. Additional information regarding the design storm is provided in Chapter 2.

LOCAL IMPROVEMENTS

The first step in analyzing the system performance was to isolate local basin capacity restrictions from overall system restrictions. The model was split into eight (8) large sub-sewersheds and each was given a free outfall to remove any downstream restrictions. All capacity issues and overflows predicted by the model were recorded. Improvements were made to increase conveyance capacity and eliminate overflows within each sub-sewershed. The improvements fell into two categories, those that were required to eliminate reported/documented overflows and those that eliminated unconfirmed model predicted overflows. In addition, solutions to resolving overflows in several locations only required raising manhole rim elevations in remote areas. After the local capacity restrictions were resolved, the model was recombined to evaluate the overall system capacity issues.

CANTRELL ROAD PUMP STATION

The Cantrell Road PS is located at 1795 Cantrell Road. The basins that are tributary to the station experience high levels of I/I. This produces high wet-weather flows that exceed the pump station's capacity and result in high levels of surcharge. The sewer main into the station is very deep and provides a moderate amount of inline storage, however once this main is filled, overflows occur upstream as the rainfall continues. The surcharge in the main provides a benefit to the Cantrell Road PS as the increased head elevates the pumping capacity to 32 mgd.

Another issue identified at the Cantrell Road PS are the pump level controls. The last pump is not programmed to activate until the sewer main is completely filled and begins to surcharge. By the time the pump activates, a significant length of available in-line storage has already been used up. Figures 4.6 and 4.7 graphically depict the hydraulic grade line in the interceptor upstream of Cantrell Road Pump Station shortly before a wet-weather event and during the design storm event.

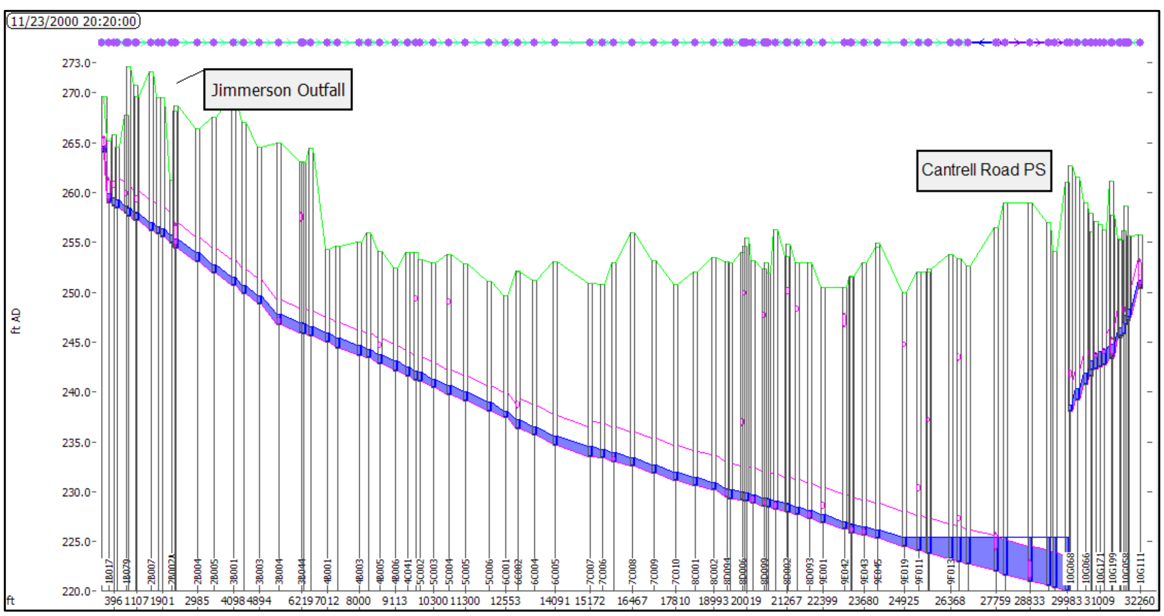


Figure 4.6: Dry-Weather Condition for Cantrell Road Interceptor

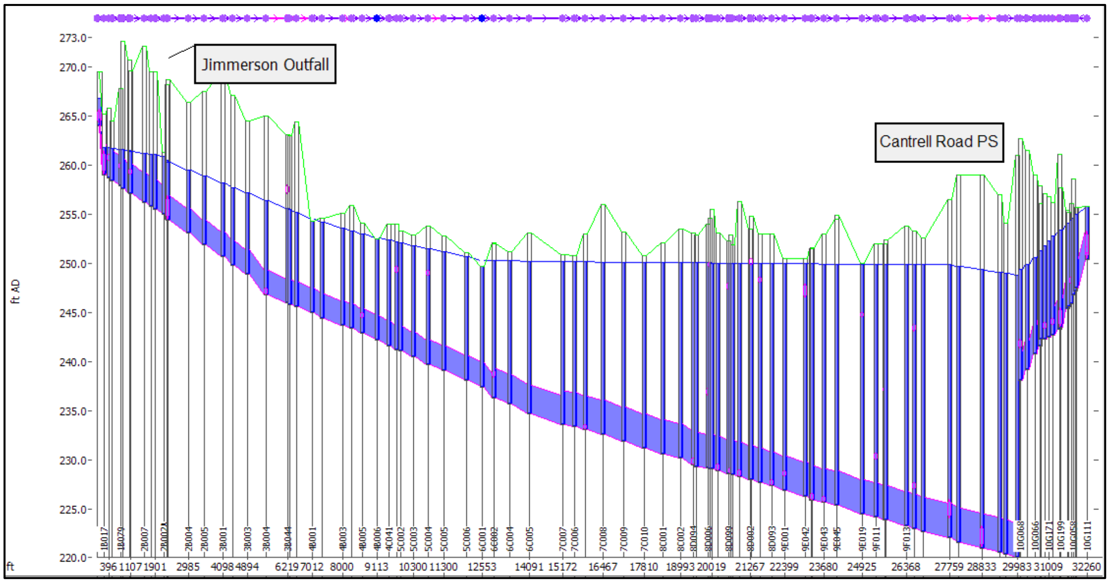


Figure 4.7: Wet-Weather Condition for Cantrell Road Interceptor.

RIVERFRONT

The Riverfront Interceptor section extends from the Cantrell Road PS to the Adams Field WWTP. There are several critical manholes in this area with known overflows due to their low elevation near the grounds of the William Clinton Presidential Library. In addition, the main has a limited free flow capacity of 30 mgd. The limited capacity of the main, combined with the high pump controls at the Adams Field MPS causes surcharging upstream to the Cantrell Road PS. Once the interceptor surcharges, any additional flow from upstream will initiate overflows at the shallow manholes. The impact of the Adams Field MPS operation levels can be seen in Figure 4.8.

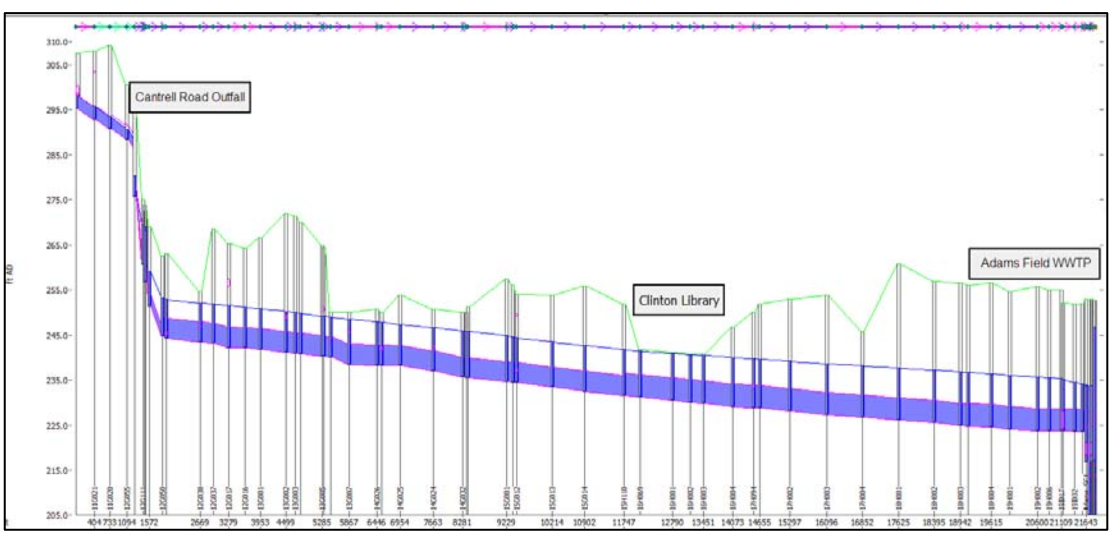


Figure 4.8: Levels of Riverfront Interceptor

ROCK CREEK

The Rock Creek system extends from the beginning of the North 60 and South 60 interceptors to the northwest sections of Little Rock. The upper section, the Grassy Flat mains, contains two parallel sewer mains, an 18-inch and a 24-inch. This area generates a hydraulic restriction which creates overflows upstream.

Another restriction along Rock Creek is near Henderson Middle School. At this location, a 30-inch main and a 36-inch main merge into a single 42-inch main. This intersection of interceptors creates surcharging in both mains. Figure 4.9 outlines the location of the upper restriction.

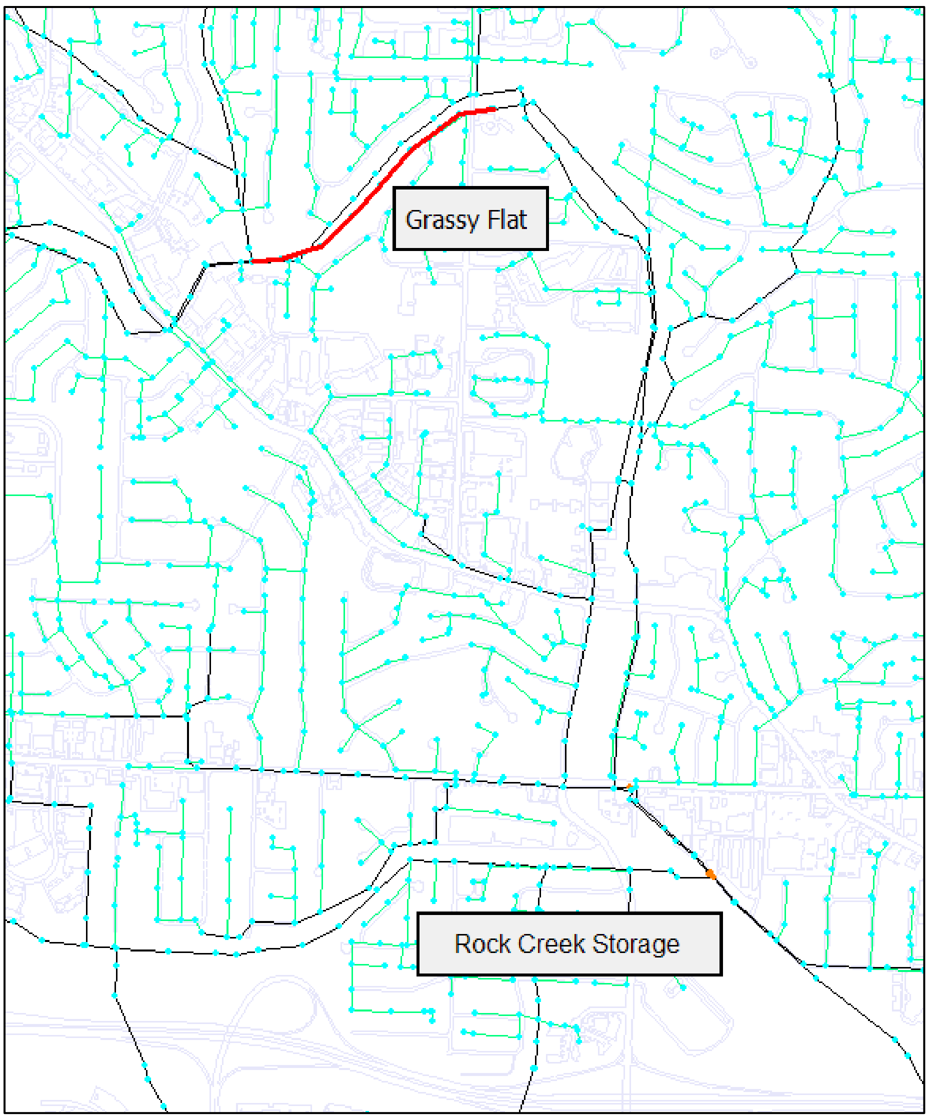


Figure 4.9: Rock Creek Hydraulic Restriction Locations

The primary cause of overflows in Rock Creek is due to surcharging and backup of flow from the North 60 and South 60 interceptors. The existing outfall of Rock Creek is tempered due to the surcharging and large volumes of overflows that occur in Boyle Park.

BRODIE CREEK AREA

As discussed previously, the interceptor from the Brodie Creek area converts to a box structure to connect to the North 60 and South 60 interceptors. During wet weather events, the flow in this main reverses and flows upstream until it discharges over a high level bypass into the Fourche Creek interceptor. The surcharge from the flow reversal causes large overflows to develop in Hindman Park. Figure 4.10 shows the hydraulic grade line from Adams Field WWTP to Hindman Park.

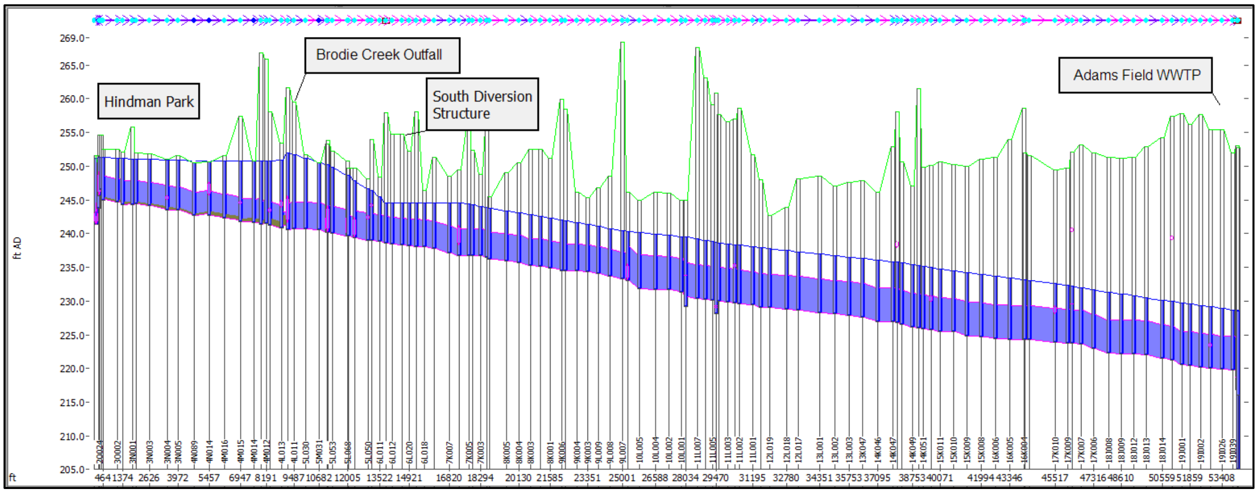


Figure 4.10: Reversal of Flow from Brodie Creek Outfall to Hindman Park

FOURCHE CREEK / NORTH 60 / SOUTH 60 INTERCEPTORS

There are three primary interceptors in the south part of the City that convey flow to the Adams Field WWTP and the Fourche Creek WWTP. The North 60 and South 60 flow directly to the Adams Field WWTP and the Fourche Creek interceptor flows to the Arch Street PS which pumps all flow to the College Station PS which pumps to Fourche Creek WWTP. The existing College Station PS is being removed from service and Arch Street PS will pump directly to Fourche Creek WWTP. During the design storm, all three interceptors surcharge and are the primary restrictions of conveying flow to the plants.

The Peak Flow Attenuation System was designed to reduce the peak flow in the North 60 and South 60 interceptors and provide offline storage of wastewater during a large rain event. The current Peak Flow system removes flow from the North 60 and South 60 interceptors and stores it. Eventually, the storage basins release back into the Fourche Creek interceptor. The system, as it currently exists, is undersized to properly alleviate the surcharge in the North 60 and South 60 interceptors. In addition, the Fourche Creek interceptor is overloaded and cannot receive additional flow draining from the storage basins.

ARCH STREET PUMP STATION AND INTERSTATE PARK GATE

Upstream of the Arch Street PS, the Interstate Park Gate can be opened to remove flow from the North 60 and South 60 interceptors and convey it to Arch Street PS. Opening the gate has a significant impact on the North 60 and South 60 surcharge levels. However, additional flow to Arch Street PS restricts flow from the Fourche Creek interceptor causing surcharge levels to rise.

ADAMS FIELD WWTP

The overall system performance hinges on the Adams Field MPS and the storage capacity available at the Adams Field WWTP. The Adams Field MPS pump controls are set high which means that the pump station is not running at full capacity until after the interceptors are surcharged. Figure 4.11 shows the impact of Adams Field MPS on the surcharge levels in the system. In addition, the limited amount of storage available at the Adams Field WWTP prevents the Adams Field MPS from maintaining 94 mgd for more than 12 hours.

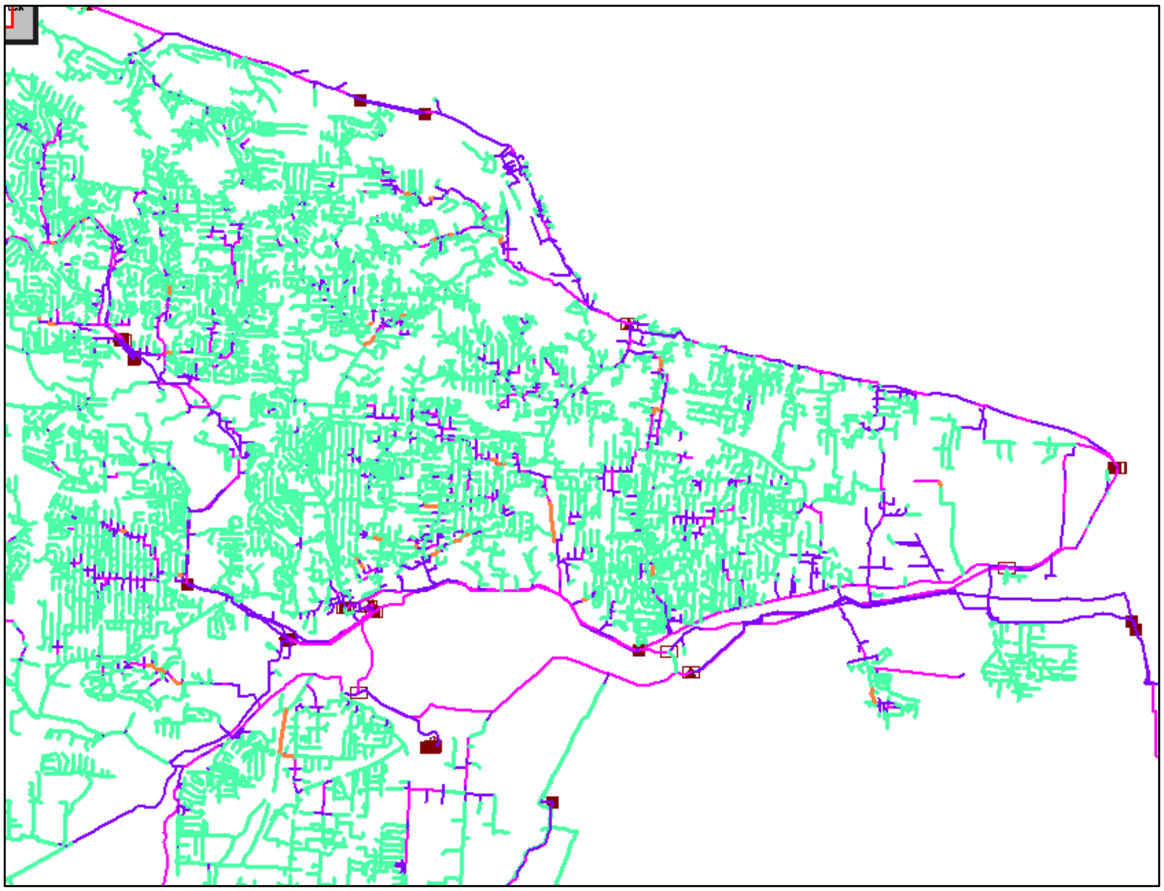


Figure 4.11: System Surcharge Levels (Darker means higher surcharge)

CHAPTER 5

EVALUATION OF ALTERNATIVES

This chapter of the report summarizes the analysis of alternatives that were evaluated to develop an improvement plan to eliminate sanitary sewer overflows. The calibrated hydraulic model described in Chapter 4 was utilized to identify capacity improvements that are localized in nature, as well as to evaluate various improvements to address overflows that are holistic or more overall system related. The evaluations included the entire collection system with the exception of the area tributary to the new Little Maumelle Wastewater Treatment Plant.

The localized improvements are those that are required to eliminate isolated overflows caused by capacity restrictions in a given area and are not related to existing system restrictions in the Arch Street Pump Station, Adams Field WWTP, or major interceptors. The holistic or system related improvements are for those overflows that are caused by multiple deficiencies and must be address by a combination of alternatives. Each is addressed in the following sections.

LOCALIZED IMPROVEMENTS

As discussed in Chapter 4, several capacity deficiencies were identified within the system that contribute to the reported overflows that occur during the design storm event. These localized deficiencies can generally be addressed by upsizing isolated sections of pipe or raising manholes to eliminate specific overflows. These improvements are independent of the alternatives that are discussed later in this chapter.

A total of 24 improvement projects were identified. Table 5-1 provides a summary of these local improvements and includes the approximate lengths, pipe diameters, and capital costs for each project. Figure 5.1 shows the general location of each project. It should be noted that the proposed pipe diameters are based on removing and replacing the existing pipe versus paralleling the pipe.

DEVELOPMENT OF ALTERNATIVES

Based on the results of the existing system hydraulic capacity analysis described in Chapter 4 various capacity deficiencies were identified. Due to the complexity of the LRW system and the interconnectivity between the various interceptors and WWTPs, it was decided to combine the various LRW service basins into four areas for the purpose of identifying alternatives to eliminate these deficiencies. The areas are as follows:

Cantrell Road Pump Station Area
Rock Creek Area
North 60 and South 60/Fource Interceptor Area
Riverfront Area and Adams Field WWTP

Table 5-1

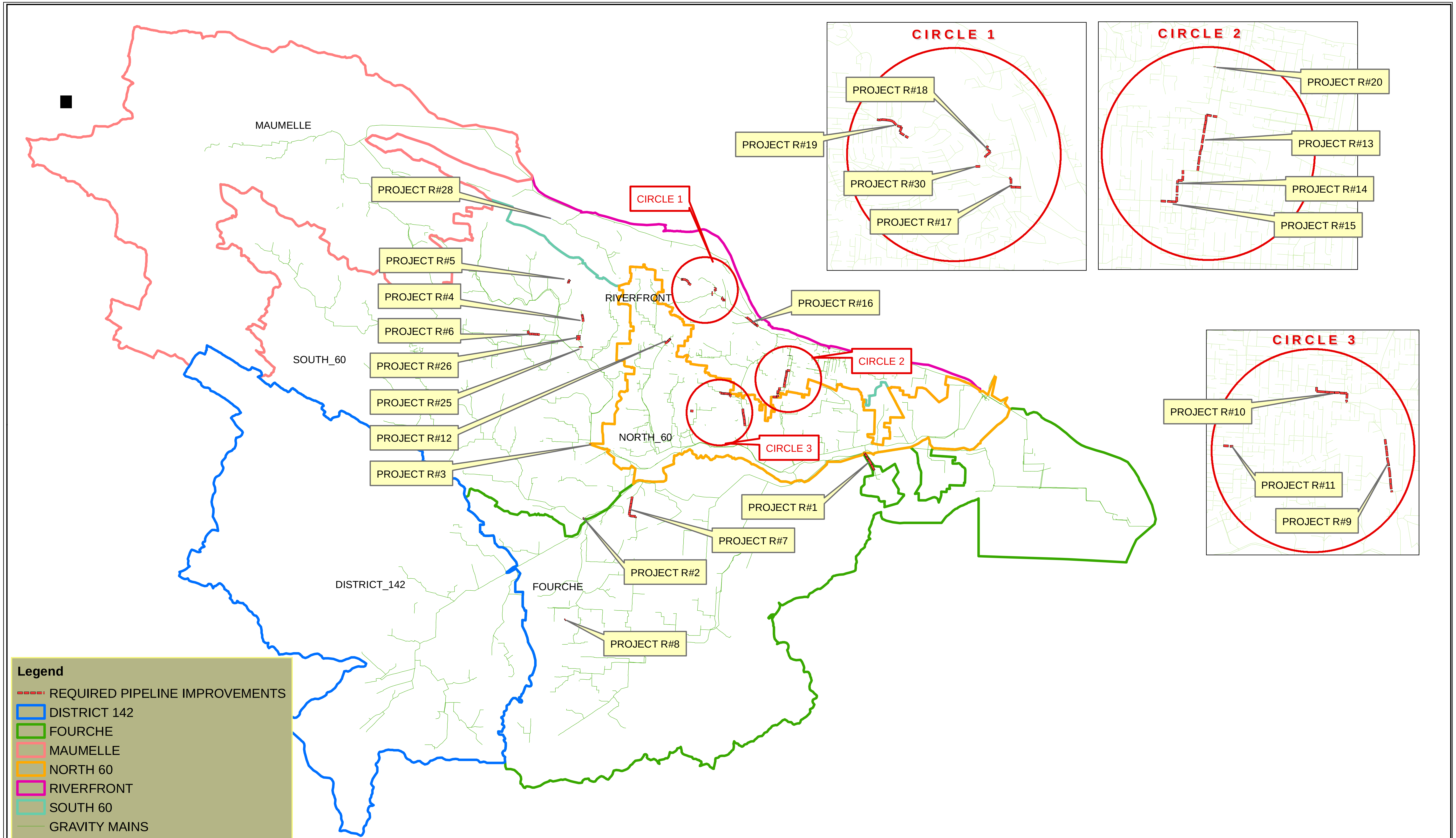
REQUIRED LOCAL IMPROVEMENT PROJECTS

Project No.	Sub-Basin	Grid Nos.	Length (lf)	Pipe Diameter (in)	Estimated Capital Improvement Cost[1/] ($)
R1	Granite Mountain	14K & 14L	2,266	15	569,000
R2	Brodie Creek East / Tall Timbers	2O	171	42	169,000
R3	Lower Coleman Creek	3L	30	60	29,000
R4	Leawood	2F	827	15	251,000
R5	Foreman Lake	1E	455	10	130,000
R6	Natural Resource Complex	0G	1,597	10 & 12	281,000
R7	Meadowcliff	4N & 4O	3,015	15	584,000
R8	Chicot (S. of Baseline)	2S	124	8	24,000
R9	Barton South	9J	1,921	18	471,000
R10	Barton North	8I	1,586	15	320,000
R11	District 84	7J	347	10	64,000
R12	Upper Coleman Creek	6G	828	12	193,000
R13	Rose Creek East	11H & 11L	2,312	18	641,000
R14	Rose Creek East	10I	944	15	230,000
R15	Rose Creek East	10I	811	15	174,000
R16	Allsopp Park South	9F	1,761	30	652,000
R17	Allsopp Park North	8E	715	24	202,000
R18	Allsopp Park North	8E	550	18	156,000
R19	Allsopp Park North	6E & 6D	1,465	12	453,000
R20	Rose Creek East	11G	19	15	7,000
R25[2/]	Leawood	2G	149	18	46,000
R26	Leawood	2G	555	18	133,000
R28	Jimmerson Creek West	0B	396	10	73,000
R30	Allsopp Park North	7E	158	15	45,000
	Total		**23,002**		**5,897,000**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*

2/ *Projects 21, 22, 23, 24, 27, and 29 are included in the required system improvements.*

REQUIRED LOCAL PROJECTS GENERAL LOCATION MAP



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LITTLE ROCK WASTEWATER
SECAP UPDATE
FIGURE 5.1

The overall system model, with the localized improvements incorporated, was used as the base for the analysis. Numerous alternatives were developed to resolve the specific problems within each of the four areas including major facilities such as interceptors, pump stations, and storage facilities. The purpose of developing the alternatives was to better evaluate the overall system issues. Each of these options was presented during a workshop with Little Rock Wastewater personnel, with pro's and con's given for each.

Each of the alternatives were evaluated and either eliminated from further consideration for specific reasons or further developed to consider operational and costing issues. Each of the alternatives that progressed for further consideration was modeled and then evaluated using the metrics of hydraulic performance, elimination of overflows, increase to capacity, and constructability. A discussion of the analysis of alternatives in each area is presented below.

EVALUATION OF ALTERNATIVES

CANTRELL ROAD PUMP STATION AREA

A total of six alternatives to eliminate overflows in the area tributary to the Cantrell Road Pump Station were identified. These alternatives were as follows:

Alternative 1 - Upsize Cantrell Road Pump station to 40 mgd capacity

Alternative 2 - Construct flow EQ storage in lower Cantrell Road area

Alternative 3 - Reduce peak inflow/infiltration

Alternative 4 - Limited reduction of peak inflow/infiltration and construct small storage

Alternative 5 - Construct wet-weather treatment facility

Alternative 6 - Construct flow EQ storage in upper Cantrell Road area

Alternative 1 involved an increase in the capacity of the Cantrell Road Pump Station from 32 mgd to 40 mgd. This would be achieved by replacing the pumps at the station and performing rehabilitation or replacement of the Cantrell Road PS force main. While this option did eliminate many of the upstream overflows, it also increased the volume of overflow occurring downstream in the Riverfront interceptor. This increase in overflow volume is caused by the limited capacity of the Riverfront interceptor, which in some segments has a capacity as low as 30 mgd. Because increasing the capacity of the Cantrell Road Pump Station would also require increasing the capacity of the downstream Riverfront interceptor and potentially the Adams Field WWTP MPS, it was determined that this was not a viable alternative.

Alternative 2 consisted of constructing peak flow storage located at the lower end of the Cantrell Road Interceptor near the Verizon Complex. This location would allow the storage facility to be filled and drained by gravity due to the depth of the Cantrell Interceptor. The model indicated that the storage facility would eliminate all overflows along the Cantrell Interceptor except for those near the Jimmerson outfall. Because of this and concerns of constructability due to land acquisition issues and potential for large optical cables in the Verizon complex area, no further consideration was given to this alternative.

Alternative 3 evaluated various combinations of I/I reduction within the Cantrell Basin. Based on the recorded flow data, Basin 112 was recorded as having a peaking factor of 15.7, and a 32.2 million gallon inflow rate during a projected 1-year/60-minute storm event. Also, Basin 59 recorded a wet-weather peaking factor of 59.3. Through an I/I reduction program, the amount of inflow entering the sewer system could be significantly reduced. Model simulations with I/I volume reductions of 10, 15, and 20 percent in Basin 112 were evaluated. Based on historical projects, a reduction of 15 percent could be realistically achieved. Although the overflows occurring in Basin 112 could be eliminated by this reduction in flow, there was no significant impact on the overflows occurring along the Cantrell interceptor. The volume of I/I reduction required to affect those overflows is most likely beyond what could be achieved

Alternative 4 using a combination option was analyzed. This option would consist of a smaller storage at the lower end of the Cantrell Interceptor and an I/I reduction program in Basin 112. Similar to the previous alternative involving these options, it did not eliminate the overflows farthest upstream near the Jimmerson outfall. In addition, the constructability of the storage remained a concern.

Alternative 5 considered the use of a wet-weather treatment facility along the Cantrell interceptor. During wet-weather events, the system would treat the increased flow and discharge directly into the Arkansas River. This option was never evaluated in the hydraulic model because the Arkansas Department of Environmental Quality (DEQ) has stated that such a system would not meet treatment requirements.

Alternative 6 evaluated the construction of a storage facility farther upstream from the Cantrell Road Pump Station closer to the Jimmerson Outfall. This facility would be located upstream of Rebsamen Park. Similar to Alternative 2, the depth of the Cantrell interceptor allows the storage to fill and drain by gravity. The hydraulic model indicated that this option would eliminate all the overflows along the interceptor except for some minor spills closer to the Cantrell Road Pump Station. A major benefit of this location is the availability of existing Rights-of-Way that can be utilized for construction.

The Cantrell Road Pump Station is not currently programmed to activate all pumps until the interceptor has already reached a surcharged state. By modifying the pump activation levels in the model to prevent this surcharge, a significant amount of existing in-line storage was freed up and could be made available during a major storm event. These reduced operating levels were tested with all the proposed alternatives for the Cantrell Road Pump Station. They were most successful when combined with Alternative 6 (storage at the upper end of Cantrell Interceptor). Since the storage at this location eliminated the majority of overflows, the reduced pump levels were able to eliminate the remaining overflows and therefore reduce the height of surcharge in the interceptor near Cantrell Road Pump Station.

ROCK CREEK AREA

The Rock Creek Basin presently includes some of the most severe overflows in the LRW collection system. Much of the Rock Creek system is also impacted by capacity restrictions downstream. Several alternatives to eliminate overflows were identified and evaluated. A summary of these alternatives are:

Alternative 1 **-** Construct new interceptor (Entire Length)

Alternative 2 **-** Construct peak EQ storage and limited new interceptor

Alternative 3 - Reduce peak inflow/infiltration and construct storage and limited interceptor

Alternative 4 - Construct series of inline storage facilities

Alternative 5 - Construct 72-inch sewer from Rock Creek to Peak Flow Pump Station

Alternative 6 - Construct wet-weather pump station

Alternative 1 evaluated increasing the capacity of the Rock Creek interceptor by re-constructing it over its entire length. A similar option of adding another parallel main was also considered and tested in the model. Both options performed similarly and were able to eliminate the overflows along the Rock Creek interceptor. However, increasing the capacity of the interceptor places significant additional burden on the already overloaded downstream system, including treatment.

Alternative 2 evaluated limited capacity improvements along the interceptor and construction of a storage facility in the Rock Creek / Markham area. This option was successful in eliminating the overflows along the Rock Creek interceptor and causes a manageable overloading of the mains downstream.

Alternative 3 considered the same limited capacity improvements as the second option, but with a reduced size storage facility, plus a reduction in I/I in several contributing basins. As observed in the Cantrell area option, the I/I reduction required to impact a substantial reduction in the size of the required storage was beyond what could be realistically achieved.

Alternative 4 evaluated multiple inline storage facilities to be placed along the Rock Creek interceptor. These included sites near the Rock Creek/Markham area, Rodney Parkham/I-630 area, and two locations east of Reservoir Road. There were multiple issues with this option. First the upstream storage east of Reservoir Road did not detain enough flow to adequately impact downstream flows. This was because of the limited size of the available sites at this location. Secondly, the storage could not be emptied by gravity and would require pump stations to be installed. Finally, the storage east of Reservoir Road would be difficult to construct, due to access issues. The storage in the Rodney Parham /I-630 area was also affected by possible land acquisition issues and the small available tracts of land.

Alternative 5 evaluated the construction of a new 72-inch interceptor main from the end of Rock Creek to the Peak Flow Pump Station. This main would provide relief during wet weather events and would tie into an existing 72-inch diameter stub-out at the South Diversion Structure. While this alternative eliminated the overflows in Boyle Park, it did not have an impact of the overflows occurring at the upper end of the Rock Creek interceptor.

Alternative 6 included the construction of a new pump station and force main from the Rock Creek/Markham area to the Peak Flow Pump Station. While this option relieved overflows downstream of the Henderson Middle School, it did not alleviate the upstream overflows. Also, the extremely long force main (over four miles) presented multiple constructability issues, along both of two considered routes. In addition, major downstream improvements (added capacity at the Peak Flow Pump Station and added storage at the Peak Flow Basins) would still be required.

NORTH 60 AND SOUTH 60 / FOURCHE CREEK INTERCEPTORS

The "heart" of the Little Rock sewer system is in the area where the North 60, South 60, Rock Creek, Brodie Creek, and Fourche Creek interceptors all meet. Thus, all options considered in this location and downstream have a significant impact on the performance of the entire system. The options evaluated included:

Alternative 1 - Construct capacity improvements from outfall of Rock Creek to Peak Flow Pump Station

Alternative 2 - Construct additional storage and upgrade Peak Flow Pump Station

Alternative 3 - Reduce Peak inflow/infiltration

Alternative 4 - Construct wet-weather treatment facility

Alternative 1 considered increasing the capacity of the main from the Rock Creek outfall to the Peak Flow Pump Station. This option was modeled in two configurations. The first proposed constructing a parallel main at a similar elevation to the existing Twin 60's. The second utilized a parallel main set approximately 20 feet below the surface elevation, connecting to the Peak Flow pumping station. After analyzing both options, the 20 foot deep main performed significantly better than a parallel main set to existing elevations. The primary advantage of the deeper main is that it prevents the flow reversal upstream into the Brodie Creek and Fourche Creek interceptors outfall during wet weather events. With the Brodie Creek outfall able to flow freely into the deeper main, this option provides relief for all the overflows in the lower section of Rock Creek and in Brodie Creek all the way back to Hindman Park.

Alternative 2 analyzed the expansion of existing system storage. Three storage alternatives were tested. The first was the expansion of storage at the existing Peak Flow Equalization Basins. The second was construction of new storage along the Brodie Creek outfall behind the old Ford building. The third was construction of a new storage facility at the BFI landfill near Mabelvale Pike, which is currently in the process of being closed. Based on the hydraulic performance of the Brodie Creek outfall, and existing elevations, the storage behind the Ford building was determined to be in the wrong location to adequately remove flow from the system. In addition, the existing Peak Flow Pump station would not be able to be used for filling of the storage.

The first and third storage options under this alternative both require limited upgrades to the exiting Peak Flow Pump System for filling. Both the Mabelvale Pike site and the existing Peak Flow Equalization Basin perform similarly. In order to remove enough flow from the system to either of these storage locations, the Peak Flow Pump Station must be increased in capacity installing a fourth pump. With the fourth pump installed and with the addition of the deep sewer into the pump station, the overflows are eliminated in the lower Rock Creek and Brodie Creek areas.

Alternative 3 analyzed I/I reduction for this area. An I/I reduction program would reduce the amount of contributing runoff from the upstream basins. However, as with the analyses of I/I reduction in other basins, the required percentage of reduction is not realistically obtainable to consider as the sole source of eliminating overflows.

Alternative 4 considered a wet-weather treatment facility. During wet weather events, the system would treat diluted flow and discharge into Fourche Creek or pump over the ridge to the Arkansas River. Like the Cantrell option, the Arkansas DEQ stated that the system would not meet treatment requirements.

Similar to current operations at the Cantrell Road Pump Station, the Arch Street Pump Station is not programmed to be at full capacity until after the Fourche Creek Interceptor has started to surcharge. Thus, with the main already surcharged, any additional flow from I/I causes the Fourche Creek Interceptor to overflow. The model was tested with various pump control levels to assess their impact on the Fourche Creek Interceptor. After several iterations, a set of levels was determined that successfully eliminated the overflows occurring on the Fourche Creek Interceptor. These levels settings are provided later in this Chapter and in Chapter 6.

RIVERFRONT AREA AND ADAMS FIELD WWTP

Due to the density of development and the hydraulic restrictions from the flat interceptor, there are limited options available for system improvements within the Riverfront Basin and the immediate areas around the Adams Field WWTP. Two options were tested in the model: expansion of the storage at the Adams Field WWTP and adjustment of the operating levels at the Adams Field MPS, similar to that proposed for the Cantrell Road and Arch Street Pump Stations. Both were successful in reducing the surcharge levels in the sewer system. By adjusting the operation levels of the Adams Field MPS alone, the predicted overflows near the William Clinton Presidential Library were eliminated. In addition, the modified pump controls produced a system wide reduction in surcharge levels. The preferred operating levels are discussed later in this chapter.

TRANSPORT AND TREAT VERSUS STORAGE ALTERNATIVE

Initial alternative screening included the consideration for transportation and treatment of wet-weather flows to the existing treatment plants. Also considered was the establishment of distributed high rate treatment facilities for excess flows. Neither of these alternatives was carried beyond conceptual development due to complex regulatory and cost issues.

A review of the potential for expansion of the Adams Field and Fourche Creek wastewater treatment plants to treat storm flows in excess of currently planned expansions identified the need to expand the full treatment train process for the intermittent flows. There has not been a successful application of an alternative treatment process permitted in Arkansas for wet-weather flows. The alternative considerations were based on full replication of the treatment processes at each plant for the full wet-weather flows. The expected frequency of events makes the technical viability of biological 'stand by' capacity susceptible to a greater risk of process failure than the current operational modes. However, the key reason to drop plant expansion from detailed consideration was the anticipated cost per gallon of capacity. Recent Arkansas plant construction costs for full secondary plus disinfection treatment plants have been in the $6 to $10 per gallon of capacity range. This would equate to a cost of between $228 million and $380 million for additional capacity at Fourche Creek or Adams Field. In addition, there would be significant cost to upgrade the pump stations and construct interceptors to convey the peak flow to the treatment facilities.

As discussed above, system wet-weather treatment consisting of high rate clarification followed by disinfection was also considered in the initial screening evaluation. This type of treatment has met with significant regulatory opposition in many jurisdictions when proposed for separate sewer systems. There are no wet-weather only alternative treatment facilities on separate sewer systems currently permitted in Arkansas. The likely locations for these distributed treatment systems would also discharge into more sensitive receiving streams than the currently plant NPDES discharges. The probability of permitting anything less than full treatment in a timeframe compatible with project objectives was considered unacceptable. The costs for the proposed high rate treatment facilities was judged to be compatible with the storage options that were carried forward for development and detailed evaluation.

The costs for storage of the wet-weather flows was initially estimated at $1.25 to $2.00 per gallon and presents a lower cost alternative and acceptable likelihood of meeting the project timeline and water quality objectives. The detailed cost analysis and development of alternatives were based on storage of wet-weather flows prior to treatment in the existing wastewater treatment plants.

SYSTEM IMPROVEMENTS

PHILOSOPHY

The primary criteria for choosing alternatives were to eliminate overflows and to maximize the use of existing infrastructure. The sewer system is well equipped in many places to handle large flows from excessive I/I. However a few system bottlenecks cause major hydraulic throughput issues. By eliminating these bottlenecks, the system can perform to its maximum ability during a wet-weather event.

REQUIRED PROJECTS

The following is a list of all projects that must be completed at a minimum and are common to all the recommended alternatives.

- Complete prior SSES recommended improvements
- Complete construction of all mains previously designed but not constructed
- Increase capacity of 23,002 linear feet of sewer main based on local capacity and overflow identification
- Raise elevation of 20 remote manholes
- Continue inflow/infiltration reduction program (specifically in the basins contributing to the Cantrell Road Pump Station, which may reduce the size of the required storage)
- Re-program Adams Field MPS, Arch Street, and Cantrell Road Pump Station operating systems with recommended control levels for wet-weather operations

Based on the results of the alternative analysis, recommended improvements were developed for the four areas discussed. These recommendations are further discussed below. A summary of the projects and their respective costs are presented later in this chapter.

CANTRELL ROAD PUMP STATION AREA

The recommended alternative to eliminate the overflows in the Cantrell Basin is Alternative 6, peak flow storage along with changes to the wet-weather operation levels at Cantrell Road Pump Station. The recommendations are discussed in the following paragraphs.

The first stage involves lowering pump operating levels at the Cantrell Road Pump Station prior to a wet weather event to maximize availability of in-line storage. All four pumps should be operating when the level in the wet well reaches 223 ft, with the "off" level for the pumps lowered to 221 feet. This change in operational setting is necessary only when it is anticipated that a significant wet weather event is imminent and should not result in excessive pump cycling.

In order to eliminate the overflows in the upper Riverfront interceptor, an in-line storage system is recommended. It is recommended that the storage facility be located in the upper Cantrell area along Rebsamen Park Road, within the existing ROW, between the road and railway line (or north of the road), and opposite the golf course.

The proposed configuration of the storage facility was modeled as 4 million gallons, approximately a 2,700 ft long by 20 ft wide by 10 ft deep, covered, rectangular channel, running parallel to the existing sewer. At either end of the storage, and potentially at a midpoint, the storage will connect into the existing sewers with filling weirs / pipes set at the level of the pipe crown. The storage should also have a low level (downstream end) invert-invert connection to drain back into the interceptor.

There are no recommended controls for the storage system as the filling and draining mechanisms are passive. However, LRW may consider making allowances for actuated valves/ sluices into the final design of this structure that could become part of a future system wide SCADA control scheme.

It is also recommend that Little Rock Wastewater continue with their current rehabilitation program in the area to reduce I/I. Final sizing of the proposed storage facility should be determined following post rehabilitation flow monitoring and model recalibration.

ROCK CREEK AREA

Within the Rock Creek Basin, pipeline capacity improvements recommended to eliminate the bottlenecks in the system are minimal. This line work will enable the overall recommended options to operate at maximum capacity. The capacity improvements are included in the recommended projects for the line work to the North 60 and the Grassy Flat Main.

A major confluence of sewer mains occurs in the vicinity of Henderson Middle School. At this location, the 36-inch diameter sewer that serves the recent development in the western portion of the City merges with the parallel 30-inch and 21-inch diameter sewers along Rock Creek. Following the connection of the two main sewers, the 30-inch diameter Rock Creek Main is increased to 42 inches in diameter. At the point of confluence, flows from the western branch comprise approximately 40 percent of the total downstream flows with the remaining 60 percent coming from the northern Rock Creek tributary area.

At this convergence point, underground storage as described in Alternative 2 is recommended to detain flow, which will allow maximizing the available capacity in the Rock Creek Interceptor without causing overflows. The proposed storage system for this area would effectively divert and contain flows from the western branch during wet weather events. The configuration that was modeled for this solution was 7 million gallons, as a 5 barrel 20 ft wide by 10 ft deep, covered, rectangular channels, approximately 940 ft long, running parallel to the existing sewer. Similar to the recommended covered storage in the Cantrell basin, connections will be made at either end of the storage to the existing sewers with filling weirs / pipes set at the level of the pipe crown. The storage should also have a low level (downstream end) invert-invert connection to drain back into the interceptor.

Because of specific site constraints and cost considerations, the recommended layout for the storage is 7 barrels of 12 ft wide by 12 ft deep boxes. Figure 5.2 shows the modeled layout for the proposed wet-weather storage.

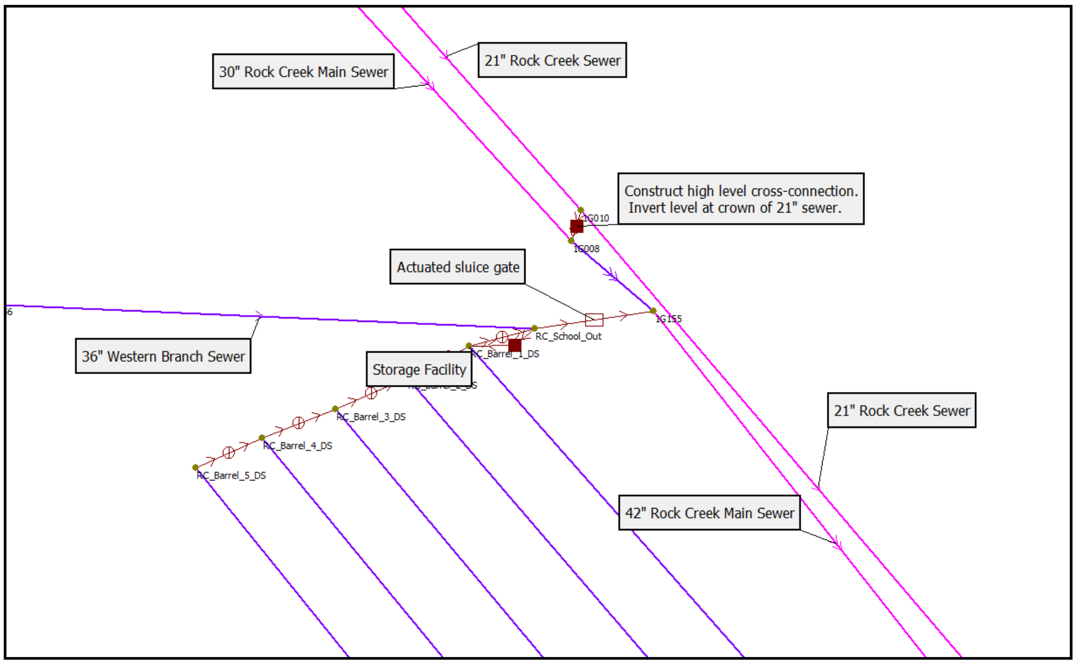


Figure 5.2: Proposed Storage Layout

The philosophy behind the storage is to allow the 21-inch and 42-inch diameter Rock Creek outfall sewers to run at capacity, with excess flows from the western branch sewer restricted by an actuated sluice gate, to be diverted into the storage facility. By locating the storage in this area, it can fill and drain by gravity and will empty once capacity is available in the 42-inch diameter Rock Creek Main sewer. This eliminates the need for a major pump station to drain the structure(s).

The control system developed and tested in the model has two components; a level sensor in manhole 1G155 and a 3 ft wide actuated sluice valve that can divert flows from the western branch sewer to the storage facility. The downstream outlet can be sized to minimize the impact of its contributing flow to the 42-inch diameter main.

The gate position is modulated by an incremental controller every 2 minutes that compares the depth of flow in the outgoing 42-inch diameter pipe with the full pipe depth. The change in sluice gate position dictated by the depth of flow is given in Table 5-2.

Table 5-2

ROCK CREEK STORAGE SLUICE GATE CONTROLS

Depth of flow - Pipe full depth (ft)	Incremental Change to Gate Position (ft)
-2.00	0.10
-1.00	0.05
-0.50	0.00
0.00	0.00
0.50	-0.10
1.00	-0.20

The success of this arrangement hinges on the ability of the storage facility to be on-line and capable of draining, whenever there is capacity in the 42-inch diameter outfall sewer.

Figure 5.3 illustrates the modeled flows through the gate valve during the design storm event. Other than a brief period of approximate 2 hours there is always some flow passing through the gate, with the storage "drain-back" occurring for approximately 36 hours after the storm event.

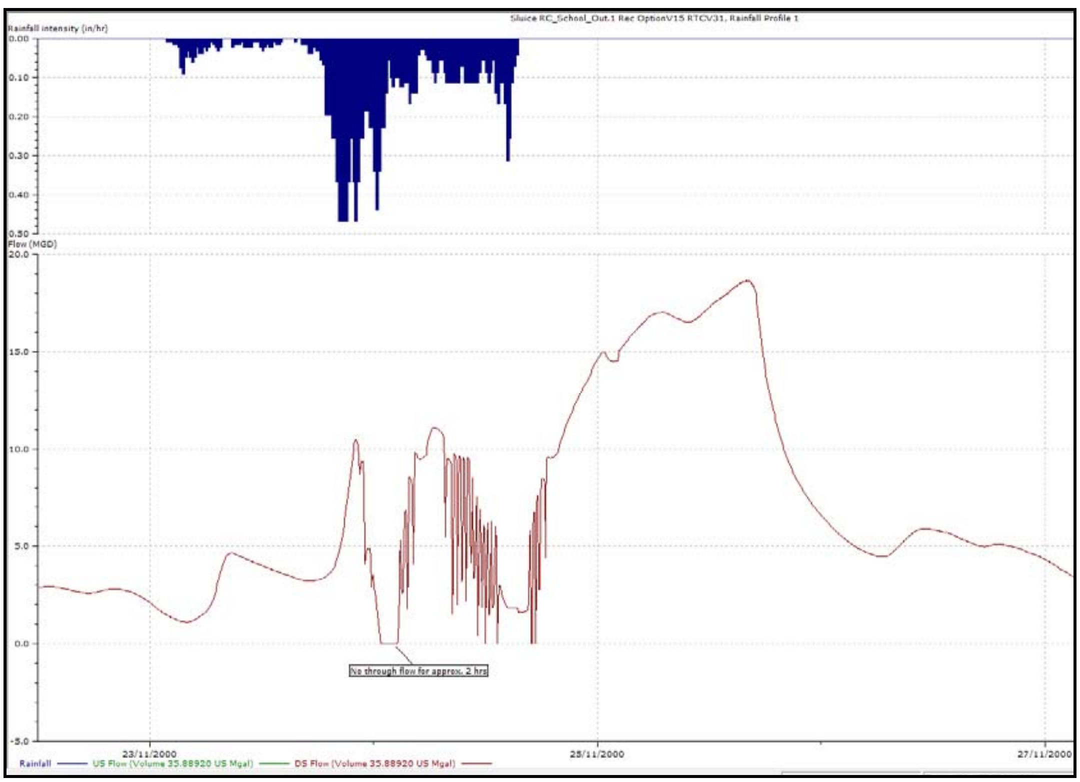


Figure 5.3: Flow Through Gate Valve

NORTH 60 AND SOUTH 60 / FOURCHE INTERCEPTOR AREA

The recommended alternative for this area includes a combination of Alternative 1 and Alternative 2 along with other modifications to the collection system and pump station. A discussion of the recommendations by sub-area or zone is given in the following sections.

Rock Creek / Brodie Creek

To eliminate the hydraulic restrictions at the outfalls of the Rock Creek and Brodie Creek interceptors, a new interceptor is recommended for construction. This project is the Peak Flow Interceptor from 36th Street to Mabelvale Pike. The proposed sewer main is 42-inches in diameter from the Rock Creek outfall to the Brodie Creek outfall and 48-inches from the Brodie Creek outfall to the inlet chamber for the Peak Flow Pumping Station. Figure 5.4 shows an indicative alignment of the proposed sewer.

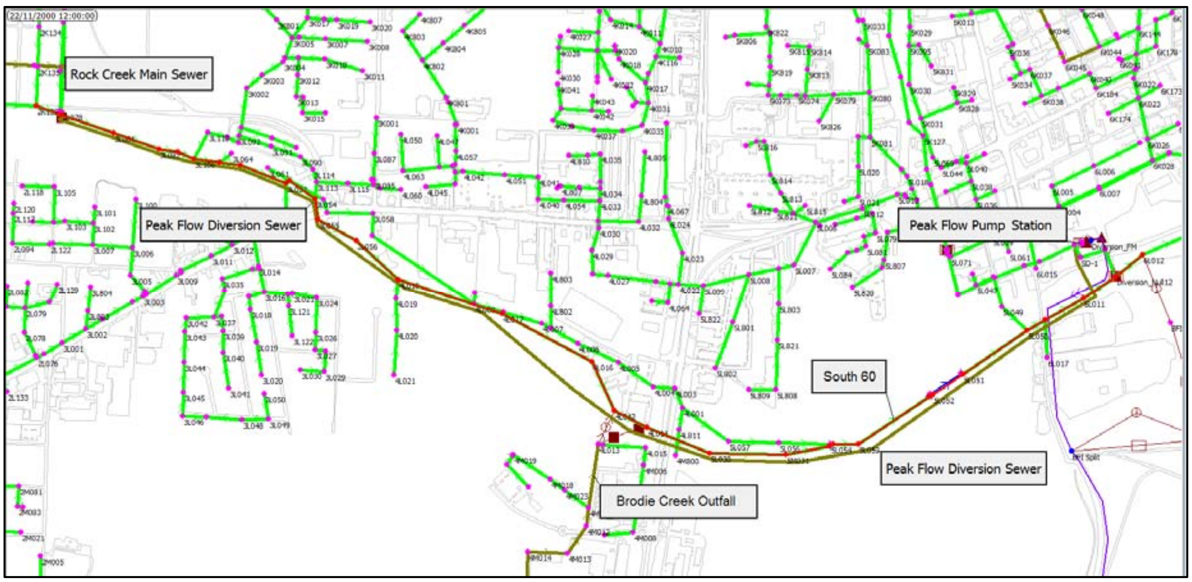


Figure 5.4: Proposed Layout for Peak Flow Interceptor from 36th Street to Mabelvale Pike

The proposed sewer runs approximately parallel to the existing South 60, but should have a vertical alignment approximately 5 feet below the existing sewer to reduce conflicts with branch sewers and enable it to cross under the South 60 near the Peak Flow pumping station.

The primary function of the Peak Flow Interceptor is to convey wet-weather flows in excess of the capacity of the South 60 sewer, directly into the Peak Flow Pump Station. Dry weather flows will remain in the South 60 main. The proposed solution requires a few key connections and control strategies.

The Peak Flow Interceptor will extend as a 42-inch diameter sewer from the Brodie Creek outfall to the outfall of the Rock Creek interceptor. A new junction box with diversion structure is required to facilitate this wet-weather diversion as illustrated in Figure 5.5.

The diversion weir from the South 60 into the Peak Flow Interceptor should be adjustable to provide future flexibility with a default level slightly above the crown of the South 60. This will ensure that the Peak Flow Interceptor is only used when the South 60 is running above capacity.

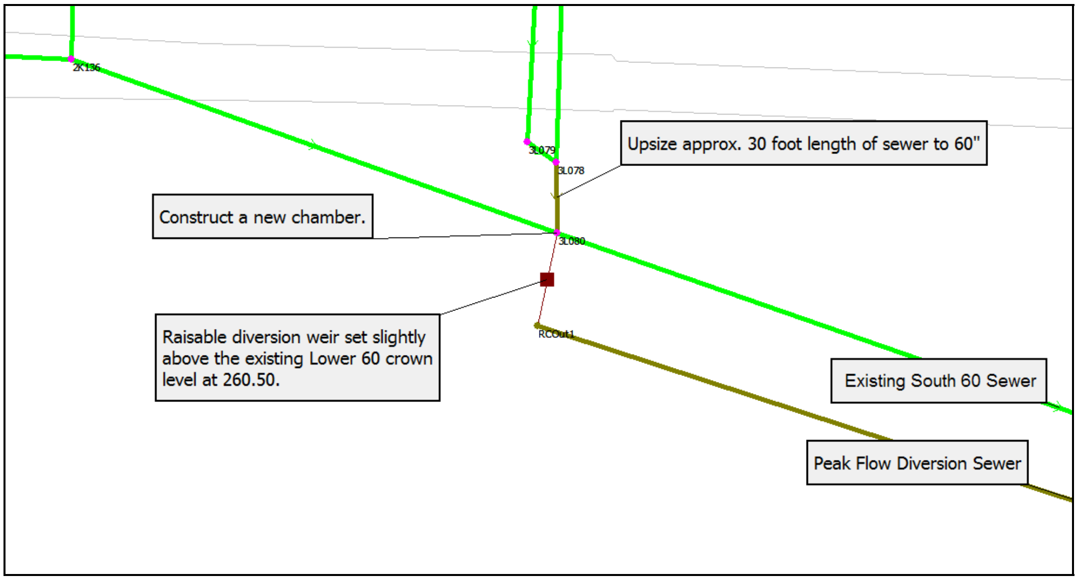


Figure 5.5: Connection of Rock Creek Interceptor

The Brodie Creek/Peak Flow Interceptor connection has a number of functional requirements and resolves a number of existing issues with the existing connection into the South 60. The modeled representation of the connection is shown in Figure 5.6 with the following key functional requirements.

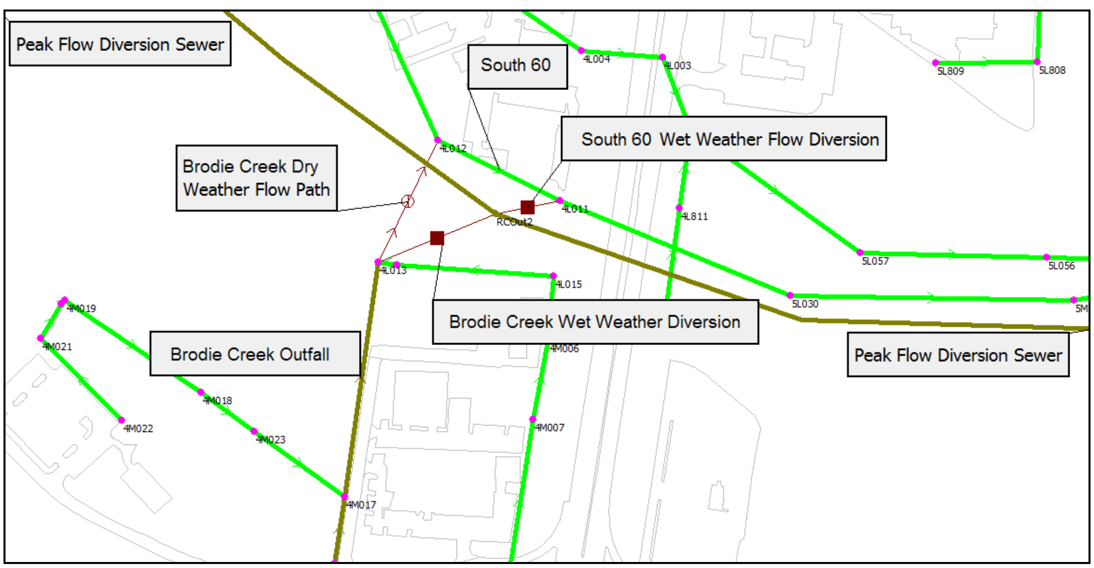


Figure 5.6: Connection of Brodie Creek

Brodie Creek Dry Weather Flow Path

- Peak dry weather flows up to approximately 5 mgd should continue to be discharged from Brodie Creek into the South 60 sewer.

- Once the South 60 starts to surcharge, there should be a mechanism to prevent flows reversing back into the Brodie Creek sewer. This could be provided by an actuated sluice gate at either end of the pipe 4L012 to 4L013, triggered by a level sensor in MH 4L012 on the South 60.

- At present there is a (7 ft x 2 ft) box culvert between MH 4L013 to MH 4L012. The connection at the inlet chamber at 4L013 causes significant maintenance issues (siltation when backflow occurs). One possible solution would be to slip line the existing channel with a new pipe(s) and construct a new chamber at the site of 4L013 that appropriately directs the dry weather flows into the new pipes.

Brodie Creek Wet-Weather Flow Diversion

- Once the South 60 starts to become surcharged and backflow prevention is enabled, flows will start to rise in the Brodie Creek outfall.

- A high level cross connection should be constructed from the Brodie Creek sewer into a chamber on the Peak Flow Interceptor, with an elevation approximately equal to the crown of the 42-inch diameter sewer.

South 60 Wet-Weather Diversion

- Given the proximity of the South 60 to the Peak Flow Interceptor it is prudent to provide high level relief from the South 60 into the Peak Flow Interceptor.

- The South 60 wet-weather diversion should be located at an elevation of 249.0 ft from MH 4L011 to a chamber on the Peak Flow Interceptor.

PEAK FLOW ATTENUATION SYSTEM

The analysis of alternatives determined that there are two viable alternatives for providing peak flow attenuation. The first option, as discussed previously, would be to add a third basin at the existing Peak Flow Attenuation Facility. The second option is to construct a new facility at the BFI landfill location near Mabelvale Pike. Although the cost of constructing the facility at the Mabelvale Pike location is approximately 8 percent more than expanding the existing Peak Flow Facility, the Mabelvale Pike site is recommended. The primary reason for selecting this location is that it requires a smaller storage facility at Adams Field WWTP which will leave land available for future expansion of the treatment plant. Should additional storage be needed in the future, it could be constructed at either the Mabelvale Pike site or Peak Flow Attenuation Facility. Constructing the recommended facility now at the existing Peak Flow Facility would use all of the available land and require the basin at Adams Field WWTP to be over twice as large as required under the Mabelvale Pike location option.

The model simulation discussed in the following paragraphs assumed the facility would be constructed at the Mabelvale Pike location.

The Peak Flow Attenuation System provides relief to the North 60 and the South 60 sewer systems. Flows are diverted into the Peak Flow Pump Station, and then pumped through the Peak Flow force main from which flows can be diverted in any of three directions.

First, flows are diverted into the Fourche Interceptor and subsequently to the Arch Street Pump Station via the Fourche Diversion Valve Vault. Flows in excess of available capacity in the Fourche Interceptor can be diverted into the proposed BFI storage facility. Once the proposed Mabelvale Pike facility is filled, any additional flows are directed to the existing Peak Flow Equalization Storage.

In order to facilitate the removal of flows from the North 60, South 60, and the Peak Flow Interceptor, it is recommended that the Peak Flow Pump Station be expanded with a fourth pump to a capacity of 68 mgd. The current station configuration contains an empty seat for this pump.

The Peak Flow Pump Station receives flow from the North 60 Diversion structure, South 60 Diversion structure and eventually through the Peak Flow Interceptor. The South Diversion structure consists of a chamber on the South 60 interceptor with an actuated diversion weir. Under normal conditions the weir is set at 242.6 ft (i.e. the crown elevation for the outgoing pipe). This ensures that maximum flows are transferred through the South 60 to the Adams Field WWTP.

During major wet weather events, it is necessary to lower the weir to divert flow from the Adams Field WWTP into the Fourche Interceptor and/or peak flow storage facilities. Lowering the weir is also necessary if the surcharge levels become too high in the South 60, upstream of the diversion structure. Figure 5.7 shows the layout of the South Diversion Structure and its connection to the Peak Flow Pump Station.

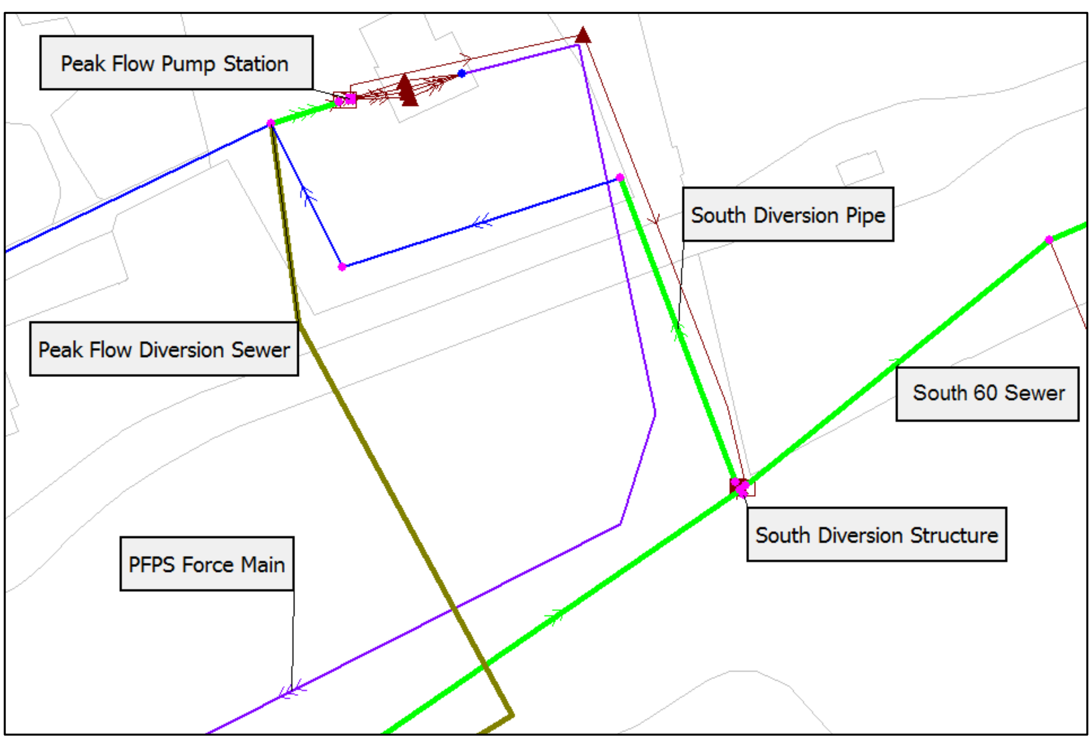


Figure 5.7: South Diversion Structure Connection Layout

The rules and triggers governing the position of the diversion weir are:

- Default position at 242.6 ft, outgoing pipe crown
- If Adams Field 27 mg Storage reaches 15% full drop weir to 240.0 ft
- If Adams Field 27 mg Storage reaches 20% full drop weir to 239.3 ft
- If Adams Field 27 mg Storage reaches 25% full drop weir to 238.6 ft
- If the surcharge level at MH 5L050 is 247.0ft or above, drop weir level to 241.1ft

All flows that have been diverted into the storage facilities drain by gravity to the Fourche Diversion Valve Vault, the Fourche Interceptor, Arch Street PS and onto Fourche Creek WWTP or into the South 60 sewer.

The operation of the Fourche Diversion Valve Vault is a key facility in the operation of the system during and after wet-weather events. The underlying operational philosophy is to maximize the amount of flow that can be diverted into the Fourche Interceptor during and following a major storm event. The valve has been modeled to operate with incremental controls that are updated every 120 seconds. The controls will require depth/pressure sensors to be located upstream of the Fourche Diversion Valve Vault, in the manhole that the Fourche Diversion Valve Vault discharges into, and in MH 2O007.

The position of the valve varies constantly throughout a storm event based upon the following conditions:

- Pressure upstream of the Fourche Diversion Valve Vault: If pressure is sensed upstream of the Fourche Diversion Valve Vault, such as when pumping commences at the Peak Flow Pump Station, the valve opens to 50 percent open. This pumps the first flush, which may contain some septic sewage left in the force main into the Fourche Interceptor and not to storage.

- High surcharge levels at MH 2O007: During the peak of a storm event, the surcharge levels in the Fourche Interceptor translate upstream to the Hindman Park Area. When the surcharge level in MH 2O007 is above 244.2 ft, the position of the Fourche Diversion Valve is governed and modulated by this level. The Fourche Diversion Valve position modulates in 0.1ft increments to maintain a surcharge level of 244.0 ft at MH 2O007.

- Level in the Fourche Diversion Valve Vault Discharge Manhole: If the surcharge level in MH 2O007 is 244 ft or below, the position of the Fourche Diversion Valve Vault gate is dictated by the level in the Fourche Diversion Valve Vault discharge manhole on the Fourche Interceptor. The Fourche Diversion Valve Vault gate will modulate its position to maintain a surcharge level of 241 ft in this manhole.

- Alarm Conditions: If the surcharge level exceeds 248 ft at MH 2O007 or 242.5 ft in the Fourche Diversion Valve Vault discharge manhole, the gate shall be closed.

The proposed Mabelvale Pike storage facility is modeled to be the first of the storage facilities to be filled. This facility should be located north of the BFI landfill. The required size of the storage facility is 51 mg. It is noted that, should this facility be designed to consider future development flows, the storage requirement is 57 mg. Flows from the Peak

Flow Pump Station, in excess of what can be discharged through the Fourche Diversion Valve Vault, will divert into the Mabelvale Pike storage facility. Once the storage is full, a valve will be shut on the inlet which will, in turn, trigger the existing Peak Flow Equalization Storage to be filled.

Also, a discharge line with a check valve should be connected into the Peak Flow force main, which will drain the Mabelvale Pike storage facility, whenever there is capacity in the Fourche Interceptor.

The existing Peak Flow Equalization Storage will start to fill once the Mabelvale Pike storage facility is full. This will be triggered by the opening of a valve downstream of the Fourche Diversion Valve Vault (FDV). The Peak Flow storage will drain to the FDV as capacity becomes available.

There is flexibility within the system, as modeled, which will allow either storage basin to fill or be drained first. The final arrangement will have minimal effect on the system's hydraulic performance.

ARCH STREET PUMP STATION AND FORCE MAINS

The Arch Street Pump Station is currently being upgraded to contain 5 pumps with a 4 pump firm capacity of 45 mgd.

As with the Cantrell Road and Adams Field MPS, it will be necessary to invoke a wet-weather pumping scheme during and following wet weather events to enable maximum conveyance of flow from the Fourche Interceptor into the Arch Street Pump Station. All four duty pumps should be operating at a level of approximately 220 ft to ensure that the hydraulic grade line remains steep enough to drive flow to the pumps.

A valve connection is proposed between the 42-inch diameter Arch Street PS force main and manhole 16K005 on the South 60. In addition to providing general operational flexibility and security, this connection will assist operators in determining the ratio of flows returned from storage that will go to each of the two treatment plants.

The model predicts that up to 24 mgd could be diverted into the Adams Field WWTP system, if an 18 inch valve was installed at this location. The final size of the valve should be determined during design, based on required velocities.

RIVERFRONT AREA AND ADAMS FIELD WWTP

The Adams Field WWTP currently contains 13 mg of equalization storage. It is recommended to expand this storage to a total of 27 mg. This will enable the Adams Field MPS to maintain its peak flow rate of 94 mgd for a longer period of time. By running the Adams MPS at peak capacity for longer, surcharge levels are able to be controlled system wide.

In addition to storage expansion, in order to prevent overflows from the low lying manholes near the William Clinton Presidential Library, the following operational protocols are required:

- The pump operation levels at the Adams Field MPS should be adjusted prior to a wet-weather event, to maintain a level of 223 ft in the Adams Field MPS wet well, until all storage is filled at the plant. This requires the pump station to be pumping 94 mgd prior to the storage basins being filled, at which time it then becomes necessary to cut back the pump station throughput to 60 mgd.

- The peak flow rate from the Cantrell Road Pump Station should be capped at 32.6 mgd. The current pump configuration achieves this peak flow rate. Future engineering at this site needs to ensure that this peak flow rate is maintained, as any increase in flow will accelerate the likelihood and frequency of overflow from manholes near the William Clinton Presidential Library.

CAPITAL IMPROVEMENT COST

DETAILED COST PRESENTATION

Detailed cost estimates for the capital improvements are presented in Appendix C. Exhibits are presented for each improvement with detailed cost spreadsheets presented and summarized for each category.

Data for costs presented herein were taken from current (2010) and recent bid tabulations of similar projects within the Little Rock area. In cases where individual comparable unit prices were not available, cost estimates were derived from manufacturer's quotes and engineering estimates. Each of the unit costs and specific project costs were independently reviewed for verification. Each cost presented herein, includes the construction costs plus a 15 percent contingency and 12 percent engineering design and construction administration fees. Any associated land costs (ROW and/or easements) are also included.

CANTRELL ROAD PUMP STATION AREA

Cantrell Road In-Line Storage – The primary alternative for the Cantrell Basin is the construction of a 4 million gallon in-line storage structure, to be located upstream of the Cantrell Road Pump Station near the outfall of the Jimmerson Creek Basin. The estimated construction cost for this facility is $9,490,000. However, the overall Cantrell Basin recommendation includes a continued SSES program for I/I reduction (ref. Chapter 7), which, with an estimated program cost of $1,359,000 and a rehabilitation construction and design cost of $13,162,000, can reduce the size and cost of the in-line storage facility. The recommended 4 mg of storage is based on no I/I reduction. If I/I reduction is performed, a proportional cost savings will result for the storage project. As recommended, prior to development of detailed plans for the storage project, updates should be made to the hydraulic model of this basin to determine the extent of storage capacity actually needed.

ROCK CREEK AREA

The primary recommended alternative for the Rock Creek Basins is the construction of a 7 mg in-line storage structure to be located in the Rock Creek/Markham area. The estimated construction cost for this facility is $15,995,000. The Rock Creek storage is a firm design and its storage volume is required, irrespective of I/I reductions that may be achieved throughout the basin.

Also, within the Rock Creek Basin is a major recommended line section, identified as the Grassy Flat Main (Required System Upgrade Project No. 27), located in the upper reaches of the basin. The total length of the 18-inch and 30-inch diameter line is 2,220 feet with an estimated cost of $803,000.

ROCK CREEK / BRODIE CREEK CONFLUENCE

The primary required alternative for the Rock Creek / Brodie Creek Confluence is the construction of the new Peak Flow Interceptor (also called the Deep 48 Interceptor). This 10,330 feet long interceptor (Required System Upgrade Project No. 22) will be constructed parallel to the existing South 60 Interceptor and includes both 42-inch and 48-inch diameter lines.

In addition to the interceptor, the other major project within this basin is the re-construction of the connection of the Brodie Creek Interceptor to the Rock Creek Interceptor. This connection frequently operates in a backflow condition and has excessive sedimentation problems. The new 42-inch diameter connection line (Required System Upgrade Project No. 23) will connect to the Peak Flow Interceptor for wet-weather flows, while a new connection, with positive drainage conditions will be slip-lined through the existing box culvert creek crossing.

The capital cost of these two projects is estimated at $10,347,000 and will require new easements and/or ROW.

PEAK FLOW ATTENUATION SYSTEM

The alternative for the Peak Flow Attenuation System is the construction of a new Peak Flow Facility at the Mabelvale Pike site. The additional required storage at the Adams Field WWTP is discussed within the Adams Field WWTP section of this chapter, as its controls are more directly tied to operations at the Adams Field WWTP.

The 51 million gallon storage basin is recommended to be constructed similar to the existing Peak Flow Storage Basin. The estimated construction cost is $49,006,000 which includes associated piping, valves, miscellaneous structures, solar powered surface aeration, grit and chemical facilities, as well as overall SCADA controls to monitor various water levels and control valves throughout the system.

ADAMS FIELD WWTP

The improvement alternative for the Adams Field WWTP is the construction of a new storage basin within the plant property, located at the existing tree farm area. While all of the system wide controls are inter-related, the existing and recommended peak flow basins at the Adams Field WWTP will operate from controls within the Adams Field Main Pump Station.

The 14 million gallon storage basin is recommended to be constructed similar to the existing Peak Flow Storage Basin within the plant property. The estimated construction cost is $12,622,000, which includes associated piping, valves and miscellaneous structures, as well as overall SCADA controls to monitor various water levels and control valves within the plant.

A summary of the improvement plan for system pipeline improvements is given in Table 5-3. A summary of the improvement plan for Peak Flow Attenuation Facilities is provided in Table 5-4. The locations of the improvements, plus the manholes that require raising are shown on Figure 5.8 and on the map included at the back of this report.

Table 5-3

SYSTEM PIPELINE IMPROVEMENT PROJECTS

Project No.	Sub-Basin	Grid Nos.	Length (lf)	Pipe Diameter (in)	Estimated Capital Improvement Cost[1/] ($ Million)
R21	Barrow Addition / South Boyle Park	3K	11	18	0.01
R22	North 60 West (Coleman to QF)	RC	10,329	42 & 48	9.34
R23	Brodie Creek / Rock Creek Conn.	4L	1,500	42	1.01
R24	Henderson JH / North Boyle Park	3I	862	30	0.48
R27	McDermont Elementary	0E, 0F & 1E	2,221	18 & 30	0.80
R29	3M Area	16K	200	48	0.10
	Total		**15,123**		**11.74**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*

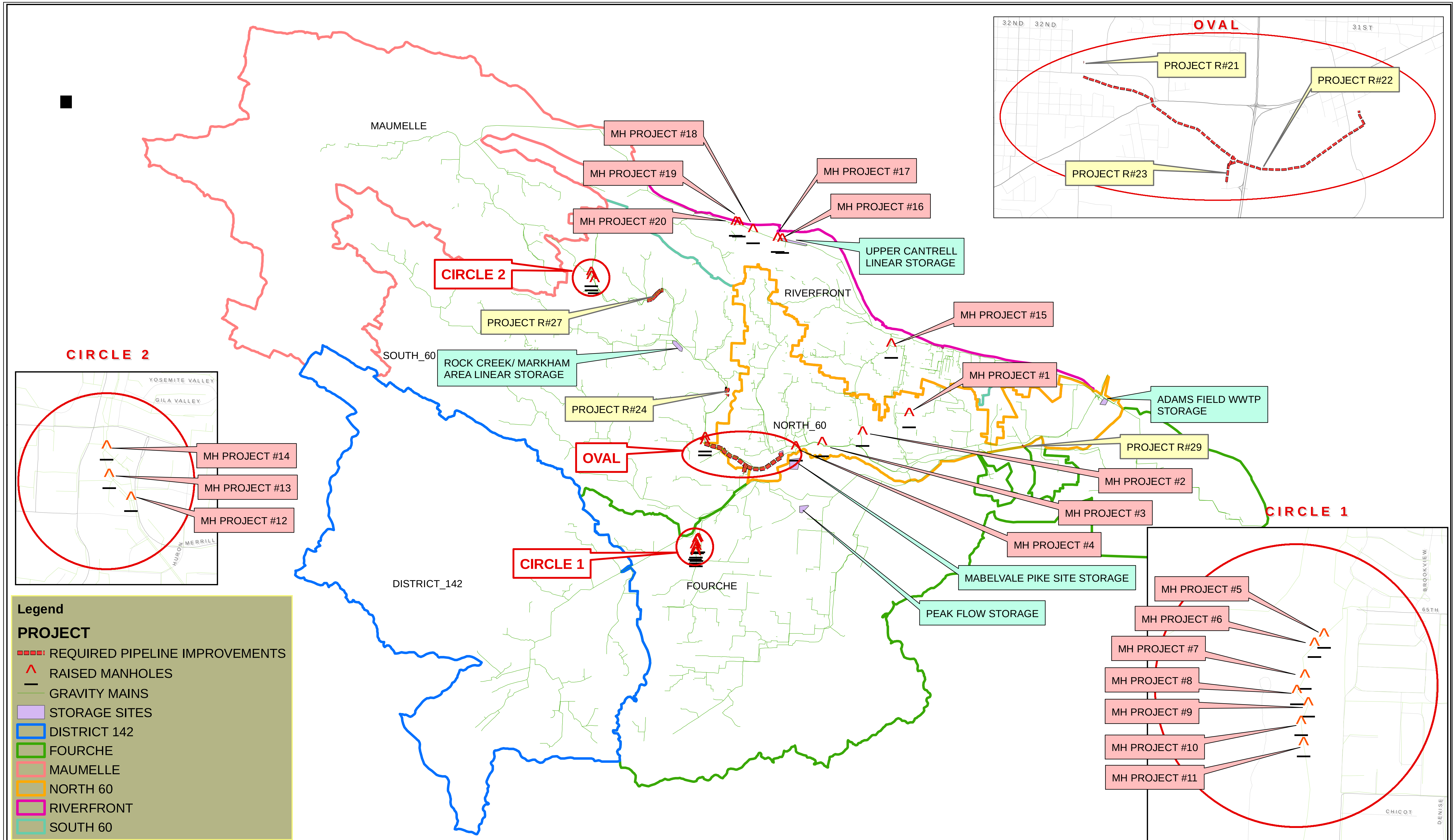
Table 5-4

SUMMARY OF REQUIRED PEAK FLOW STORAGE FACILITIES

Location	Description	Estimated Capital Improvement Cost[1/] ($ Million)
Mabelvale Pike	51 mg Basin Storage	49.01
Adams Field WWTP	14 mg Basin Storage	12.62
Rock Creek	7 mg In-Line Storage	20.49
Cantrell In-Line Storage	4 mg In-Line Storage	12.15
Additional Pump at Peak Flow PS	1-20,560 gpm pump	0.97
Total		**95.24**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*

REQUIRED SYSTEM PROJECTS GENERAL LOCATION MAP



LITTLE ROCK WASTEWATER SECAP UPDATE
FIGURE 5.8

FUTURE FLOWS

The discussion of future flows in Chapter 2 identified two major areas of expected future growth of the City and resulting wastewater flows. These two basins, located west of the current City extensions, were analyzed and modeled for predicted future flows. The required trunk mains to accommodate the flow are 4,900 feet of 12-inch and 5,300 feet of 21-inch diameter main. The estimated capital cost is $3,150,000. The layout and costs are detailed in Appendix C.

UNDOCUMENTED / MODEL PREDICTED OVERFLOWS

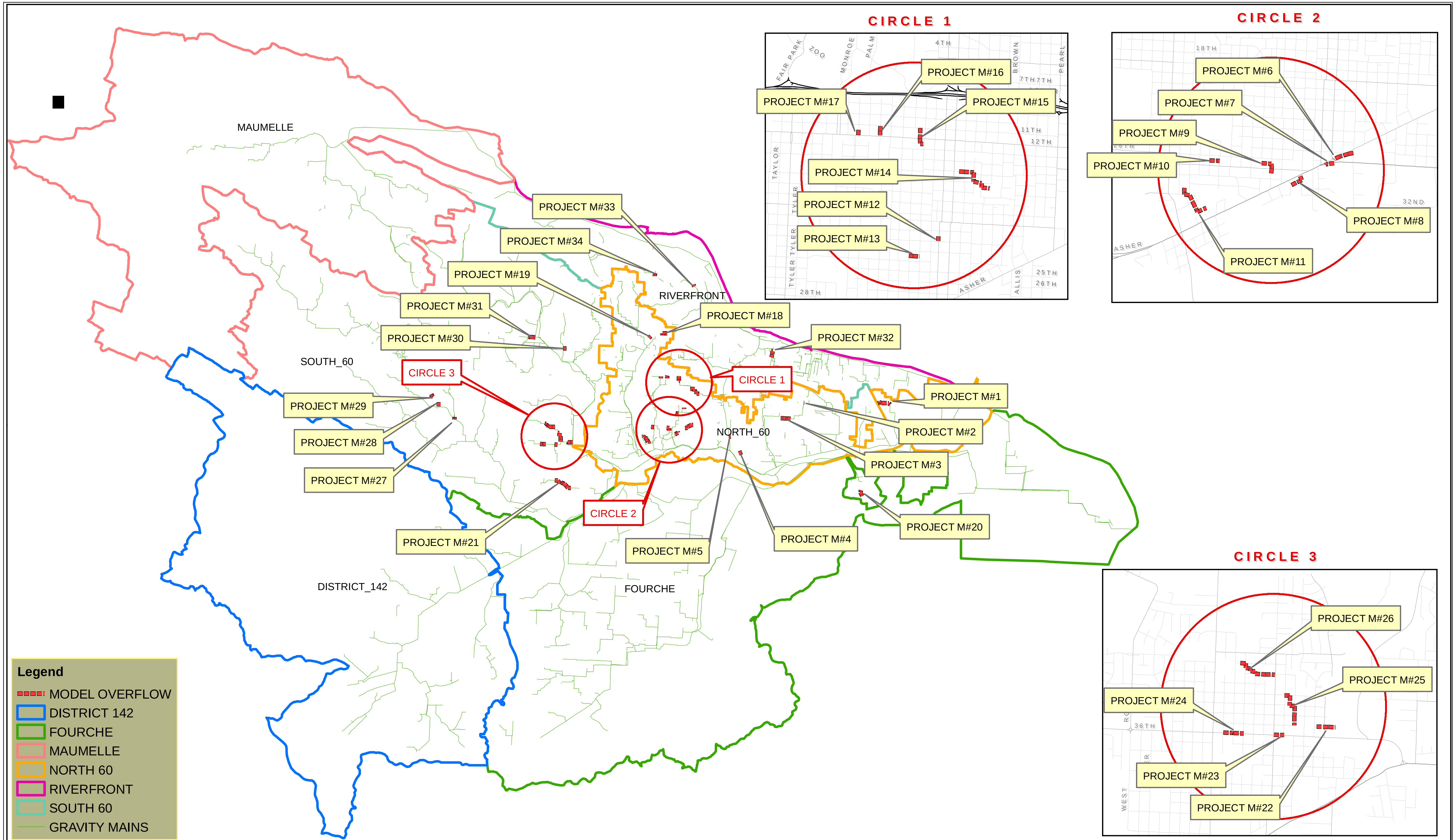
Several manholes that are not known overflow locations were predicted by the model to overflow during the design storm event and require verification by LRW staff. A map outlining the locations of the model predicted overflows can be seen in Figure 5.9. As such, line projects were proposed to eliminate each of these modeled overflows. In all, there are 34 individual projects, totaling 22,100 feet of new 8-inch to 18-inch diameter sewer lines. The estimated capital cost of the improvements is $4,970,000. The individual project layouts and detailed costs estimates are provided in Appendix C. A summary of the improvements required to eliminate these overflows, if verified, is given in Table 5-5.

It is recommended that LRW conduct a site visit of each location to inspect for any evidence of overflow prior to initiating any improvement project. If no evidence is observed it is recommended that LRW visit each location and adjacent manholes during a heavy storm event to determine the level of surcharging and potential for overflows. These overflows may not actually occur and could be predicted by the model because model geometry is different than what actually exist in the system.

CANTRELL ROAD PUMP STATION & FORCE MAIN

The Cantrell Road Pump Station and Force Main were constructed in 1967. The pump station was retrofitted with bar screens and dry pit pumps in 1986. As described in Chapter 3, improvements to these facilities are recommended. The estimated capital cost for the Pump Station improvements is $6,527,000. The force main improvements are estimated to cost $2,614,000. The detailed cost estimates are presented in Appendix C.

UNDOCUMENTED MODEL PREDICTED OVERFLOWS PIPELINE IMPROVEMENTS PROJECTS GENERAL LOCATION MAP



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LITTLE ROCK WASTEWATER
SECAP UPDATE
FIGURE 5.9

Table 5-5

UNDOCUMENTED / MODELED OVERFLOW IMPROVEMENT PROJECTS (IF REQUIRED)

Project No.	Sub-Basin	Grid Nos.	Length (lf)	Pipe Diameter (in)	Estimated Capital Improvement Cost[1/] ($)
M1	North 60 East (L.R. Airport)	15I & 16I	1,423	10	256,071
M2	Quapaw North	12I	22	10	11,254
M3	Quapaw North	11J	1,090	10	247,806
M4	S of Roosevelt (Battery St.)	10K	400	8	63,142
M5	Barton South	9K	217	10	57,805
M6	District 84	7L	667	8 & 10	122,323
M7	District 84	7K	293	10	54,965
M8	District 84	7K	536	10	143,639
M9	District 84	6K	650	8	91,134
M10	District 119	6K	331	8	65,483
M11	District 119	5K & 6K	1,291	8	259,803
M12	District 84	7J	151	8	18,422
M13	District 84	7J	342	10	65,857
M14	Barton North	7I & 8I	1,448	12	342,389
M15	Barton North	7H & 7I	621	10	185,583
M16	District 119	6H	299	8	59,891
M17	District 119	6H	189	10	44,009
M18	Upper Coleman Creek	6G	755	8	153,229
M19	Upper Coleman Creek	6G	276	8	73,059
M20	Granite Mountain	15M	696	10	115,372
M21	Western Hills	2M	2,231	12 & 15	546,128
M22	Barrow Add / South Boyle Park	2K	617	21	198,557
M23	Barrow Add / South Boyle Park	2K	333	15	80,611
M24	Barrow Add / South Boyle Park	1K	659	12	144,004
M25	Barrow Add / South Boyle Park	2K	1,290	15	263,454
M26	Barrow Add / South Boyle Park	1K	1,451	15	325,112
M27	Brodie Creek West / Sandpiper	(-)2K & (-)3K	280	18	82,032
M28	Brodie Creek West / Sandpiper	(-)3J	441	10	101,015
M29	Brodie Creek West / Sandpiper	(-)3J	473	10	99,881
M30	Leawood	2G	386	10	81,043
M31	Natural Resource Complex	1G	796	15	244,527
M32	Rose Creek East	11G	855	15	256,765
M33	Allsopp Park North	7E	191	12	48,096
M34	Allsopp Park North	6D	369	10	71,049
	Total		**22,069**		**4,973,508**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*

CHAPTER 6

CAPITAL IMPROVEMENT PLAN

This chapter presents a summary of the required plan to eliminate sanitary sewer overflows in the Little Rock Wastewater system during the design storm event. The improvement plan was developed in conjunction with LRW and is segregated into required improvements and additional improvements. Each has various components and is discussed in the following paragraphs.

REQUIRED IMPROVEMENTS

The required improvements are those necessary to eliminate the reported overflows that occur during the design storm event and are addressed in the Consent Administrative Order (CAO) and Settlement Agreement. The required improvements include pipeline and peak flow storage facilities as well as operational changes at selected pump stations. Each is addressed below.

PIPELINE IMPROVEMENTS

The required pipeline improvements include localized improvements that eliminate specific overflows within the collection system as well as those that are part of an alternative to eliminate overflows that are more holistic or system in nature caused by downstream restrictions. A summary of the pipeline projects is provided in Table 6-1.

PEAK WET-WEATHER STORAGE FACILITIES

The required plan for the construction of additional peak flow storage facilities includes construction of the new Mabelvale Pike Facility and adding an additional basin at the Adams Field Wastewater Treatment Plant. The plan also includes construction of the Rock Creek and Cantrell Road in-line storage facilities. A summary of each facility is given in Table 6-2.

Table 6-1

REQUIRED PIPELINE SYSTEM IMPROVEMENT PROJECTS

Project No.	Sub-Basin	Grid Nos.	Length (lf)	Proposed Diameter (in)	Estimated Capital Improvement Cost ($)[1/]
R1	Granite Mountain	14K & 14L	2,266	15	569,000
R2	Brodie Creek East / Tall Timbers	2O	171	42	169,000
R3	Lower Coleman Creek	3L	30[2/]	60	29,000
R4	Leawood	2F	827	15	251,000
R5	Foreman Lake	1E	455	10	130,000
R6	Natural Resource Complex	0G	1,597	10-12	281,000
R7	Meadowcliff	4N & 4O	3,015	15	584,000
R8	Chicot (S. of Baseline)	2S	124	8	24,000
R9	Barton South	9J	1,921	18	471,000
R10	Barton North	8I	1,586	15	320,000
R11	District 84	7J	347	10	64,000
R12	Upper Coleman Creek	6G	828	12	193,000
R13	Rose Creek East	11H & 11L	2,312	18	641,000
R14	Rose Creek East	10I	944	15	230,000
R15	Rose Creek East	10I	811	15	174,000
R16	Allsopp Park South	9F	1,761	30	652,000
R17	Allsopp Park North	8E	715	24	202,000
R18	Allsopp Park North	8E	550	18	156,000
R19	Allsopp Park North	6E & 6D	1,465	12	453,000
R20	Rose Creek East	11G	19[3/]	15	7,000
R21	Barrow Addition / South Boyle Park	3K	11[4/]	18	8,000
R22	North 60 West (Coleman to QF)	RC	10,329	42-48	9,337,000
R23	Brodie Creek / Rock Creek Conn.	4L	1,500	42	1,010,000
R24	Henderson JH / North Boyle Park	3I	862	30	483,000
R25	Leawood	2G	149	18	46,000
R26	Leawood	2G	555	18	133,000
R27	McDermott Elem.	0E, 0F & 1E	2,221	18 & 30	803,000
R28	Jimmerson Creek West	0B	396	10	73,000
R29	3M Area	16K	200	48	99,000
R30	Allsopp Park North	7E	158	15	45,000
	Total		**38,125**		**17,637,000**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*
2/ *Interconnection from Rock Creek Interceptor to parallel interceptor.*
3/ *Existing 12-inch diameter sewer located between two 15-inch diameter sewers.*
4/ *Interconnection between Rock Creek parallel interceptors.*

Table 6-2

SUMMARY OF REQUIRED PEAK FLOW STORAGE FACILITIES

Location	Description	Estimated Capital Improvement Cost ($ Million)[1/]
Mabelvale Pike	51 mg Basin Storage	49.01
Adams Field WWTP	14 mg Basin Storage	12.62
Rock Creek	7 mg In-Line Storage	20.49
Cantrell Road In-Line Storage	4 mg In-Line Storage	12.15
Additional Pump at Peak Flow PS	1-20,560 gpm pump	0.97
Total		**95.24**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*

PUMP STATION OPERATION PARAMETERS

The evaluation of alternatives identified the pipeline and storage facility improvements that are required to eliminate the known sanitary sewer overflows that occur during the design storm event. In some cases, the success of these improvements in eliminating the overflows is dependent on making changes to the operational parameters at several pump stations. These changes, basically, require one set of operation parameters for dry-weather periods and a different set for wet-weather periods to allow for maximum utilization of the existing LRW infrastructure. The changes are required at the Adams Field, Arch Street, Cantrell Road, and Peak Flow Pump Stations. A summary of the required operation levels is given in Table 6-3. A detailed discussion of the required changes is included in Chapter 5 of this report.

I/I REDUCTION IN THE CANTRELL BASIN

Peak flow reduction in the area tributary to the Cantrell Road Pump Station may reduce the size of the Cantrell Road In-Line storage facility and also reduce the demand placed on the pump station, downstream interceptors, and treatment facility during wet weather periods. I/I rates in these areas are significant with observed peaking factors being as high as 14 during the flow monitoring period.

REQUIRED IMPROVEMENT PROJECT COST

The total estimated capital cost to implement the required improvement plan is $127.5 million. This consists of $17.8 million for pipeline improvements and $95.2 million for peak flow storage facilities. An additional $14.5 million is included for I/I investigations and sewer rehabilitation in the area tributary to the Cantrell Road Pump Station. The estimated capital cost includes construction cost plus engineering, land acquisition, and 15 percent contingency.

A summary of the estimated capital cost is given in Table 6-4.

Table 6-3

REQUIRED PUMP STATION ON / OFF LEVELS

	Dry-Weather Flow		Wet-Weather Flow	
Location	**ON**	**OFF**	**ON**	**OFF**
Adams Field MPS	Operate as existing		Initiate Stage 7 to maintain wet well level of 223 ft until storage is full, then reduce to Stage 5 for remainder of storm event	
Arch Street PS				
Pump #1	224.5	223.5	220.0	215.0
Pump #2	225.0	224.0	221.0	215.5
Pump #3	225.5	224.5	221.5	216.0
Pump #4	226.5	224.0	222.0	216.5
Pump #5	228.5	225.0	240.0	230.0
Cantrell Road PS				
Pump #1	225.0	219.0	220.0	219.0
Pump #2	227.0	219.5	221.0	220.0
Pump #3	228.0	220.0	222.0	220.5
Pump #4	229.0	221.0	223.0	221.0

Table 6-4

SUMMARY OF ESTIMATED CAPITAL IMPROVEMENT COST

Item	Description	Estimated Capital Cost ($ Million)[1/]
Pipeline Improvements		
36th Street to Mabelvale Pike Outfall	10,330 lf of 42-inch and 48-inch diameter sewer	9.10
Grassy Flat Main	2,220 lf of 18-inch to 30-inch diameter sewer	0.78
Localized Capacity Improvements	23,002 lf of 10-inch diameter sewer to 60-inch diameter sewer	7.67
Raising of Manhole Rim Elevation	20 locations	0.23
Subtotal		17.78
Peak Flow Storage Facilities		
Mabelvale Pike Facility	51 mg	49.01
Adams Field WWTP Basin	14 mg	12.62
Cantrell Road In-line Storage	4	12.15
Rock Creek In-line Storage	7	20.49
Additional Pump at Peak Flow PS	1-20,560 gpm pump	0.97
Subtotal		95.24
Cantrell Basin I/I Reduction		
SSES	SSES in Selected Areas	1.36
Rehabilitation Design / Construction		13.16
Subtotal		14.52
Total		**127.54**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*

OVERFLOW ELIMINATION

The required improvement plan will eliminate all of the reported design storm overflows with the exception of one that could not be replicated in the model. This overflow is addressed later in the additional recommendations section of this chapter.

Each of the known overflows has been associated to a specific improvement project or combination of projects where there is a dependency one to another. A summary of the overflow elimination by project is given in Table 6-5.

ADDITIONAL IMPROVEMENTS

The additional improvements include investigations and if confirmed, elimination of overflows projected by the model to occur during the design storm event. Also included are improvements to the Cantrell Road Pump Station. These are addressed in the following paragraphs.

Table 6-5

PROJECT NUMBER RELATIONSHIP TO VOLUME AND LOCATION OF OVERFLOW

Project Number	Sub-Basin	Overflow Location	Projected Design Storm Overflow Volume Removed (mg)[1/]
1	Granite Mountain	14L026	0.150
2	Brodie Creek East/Tall Timbers	2O025, 2O026	7.083
3 & 21	Lower Coleman Creek and Barrow Add / South Boyle Park	2K143, 3K058, 3K059	12.355
4, 25, & 26	Leawood	2E080, 2E085, 2F115, 2F114	0.484
5	Foreman Lake	1E054	0.012
6	Natural Resource Complex	0G019, 0G025	0.210
7	Meadowcliff	4N080, 4O080	0.054
8	Chicot (S. of Baseline)	2S072	0.053
9 & 10	Barton South and Barton North	8I062	0.360
11	District 84	7J018	0.020
12	Upper Coleman Creek	6G012	0.002
13, 14, 15, & 20	Rose Creek East	10I112, 10I023	0.165
16, 17, 18, 19, & 30	Allsopp Park North and Allsopp Park South	8E049, 8E039, 8F014, 8E050, 7E043, 7E044, 7E055, 7E128, 7E046, 6D036, 6D050, 6D060 6D103, 6E024, 6E023,	0.458
22 (36th St. to Mabelvale Pike Outfall)	North 60 West (Coleman to QF)	4N089, 3N005, 5L066, 5L072, 4M016, 2Q021, 5L050, 4M014, 2K143, 2O026	9.599

1/ *The volume for overflows at each location are only assigned to one project.*

Table 6-5 (Cont.)

PROJECT NUMBER RELATIONSHIP TO VOLUME AND LOCATION OF OVERFLOW

Project Number	Sub-Basin	Overflow Location	Projected Design Storm Overflow Volume Removed (mg)[1/]
23 (Re-design of Junction Box intersection of Brodie Creek and Rock Creek Interceptor)	Brodie Creek / Rock Creek Conn.	4L013, 4M014, 4M016, 4N013, 4N089, 3N055, 3M005, 3N005, 3N007, 3N004	2.342
24	Henderson JH / North Boyle Park	3I036, 3I037, 3I046, 2H017, 2H018, 2H019	7.806
27	McDermott Elem.	0E052, 0E011, 0D034, 0D021, 0D019, 0D104, 0D108	1.086
28	Jimmerson Creek West	0B068, 0B066, 0B065	0.300
29 (Pinch valve connection between Fourche Interceptor and South 60)	3M Area		No associated overflow, critical in flexibility for Arch F.M. to South 60 Connection
Rock Creek In-Line Storage		1G142, 0G085, 0G087, 1G087, 1G008, 1G090, 1G010, 3K058, 3K059, 2H018, 2E080, 2E085, 2F114, 2F115, 2H017, 0G019, 0G025, 3E036, 3I037	1.515
Cantrell Road In-Line Storage		5C007, 4B001, 4B006, 6C036, 6C047, 6C001, 5C097, 8D034, 8D033, 6C048, 5C006	8.886
Peak Storage (Adams Field & Mavelvale Pike)		4N089, 3N005, 5L066, 5L072, 4M016, 2Q021, 5L050, 4M014, 2K143, 20026, 5L071, 3K059, 2O025, 5M031, 3N004, 4N030, 5L024, 2O007, 5L052, 3K058, 4L013, 2M028, 4M003, 2P014, 6L011, 4N031, 2M085, 3M005, 3N055, 2R026	8.027

1/ *The volume for overflows at each location are only assigned to one project.*

Table 6-5 (Cont.)

PROJECT NUMBER RELATIONSHIP TO VOLUME AND LOCATION OF OVERFLOW

Project Number	Sub-Basin	Overflow Location	Projected Design Storm Overflow Volume Removed (mg)[1/]
MH Raising		2P014, 2P012, 6C047, 6C001, 5C006, 5C007, 4B006, 4B001, 6L018, 11J053, 4N031, 4N030	8.235
Arch Street Level Controls		6M002, 6M001, 6N001, 6N003, 6N004, 6N002, 6N008, 6N015, 6N016, 6N018	3.741
Adams Field MPS Level Control		16H003, 16H002, 16H001, 16H069, 5L066, 5L050, 6L018, 6L011, 5L052, 5M031	6.812
Cantrell Road PS Level Control		5C007, 4B001, 4B006, 6C036, 6C047, 5C097, 8D034, 8D033, 6C001, 5C006	0.680
<u>Designed Projects Not Constructed</u>			
Jimmerson Outfall		1B015, 1B017, 1B012, 0B066, 1B026, 1B018, 0B065, 0B068	6.914
Little Maumelle WWTP		-8-A006, -8-B003, -8-A012, -8-B008, -8-A015, -7A065, -7-A008, -8-B007, 7-B001, -7A053, -8-A007, -7A009, -8-B015, -9-B035, -8-B002, -5C009, -5C096, -5D021	8.999
Country Club SSES		8E070, 8F014, 7E055, 8E049, 8E039, 8E050, 6D103, 6D036, 6D060, 6D050	0.393
Jimmerson East SSES		3D065, 3D108, 4C095, 3D119	0.017

1/ *The volume for overflows at each location are only assigned to one project.*

UNDOCUMENTED / MODEL PREDICTED OVERFLOWS

The hydraulic model predicted overflows in several locations that have not been documented. As part of this project, pipeline improvements and estimated capital cost were developed to eliminate these potential overflows. The locations of the predicted overflows are provided in Table 6-6.

A summary of the improvements necessary to eliminate these potential overflows is given in Table 6-7. It is recommended that the LRW conduct a site visit of each location to inspect for any evidence of overflow prior to initiating any improvement project. If no evidence is observed it is recommended that LRW visit each location and adjacent manholes during a heavy storm event to determine the level of surcharging and potential for overflows. These overflows may not actually occur and could be predicted by the model because model geometry is different than what actually exist in the system.

CANTRELL ROAD PUMP STATION

The Cantrell Road Pump Station was constructed in 1967 and was modified with bar screens and two dry pit submersible Flygt pumps in 1986. Two of the four pumps are original pumps while the other two pumps are replacement pumps that were installed in 1986. Two bar screens were also installed in 1986. A portion of the switch gear is original while some was replaced or added in 1986.

It is recommended that the electrical and mechanical components of the station be replaced. It is also recommended that an additional force main be constructed and the existing force main inspected and rehabilitated as required. A summary of the recommended improvements is provided in Table 6-8.

CAPACITY IMPROVEMENTS FOR FUTURE GROWTH

This update to the 2002 SECAP made allowance for future flows by analyzing planning area map, zoning requirements, and land use maps outside of the Little Maumelle WWTP tributary area. Sewer service area boundaries were overlaid on existing aerial photography maps and full buildout populations were projected. It is estimated that the Little Rock population can increase by approximately 13,000 primarily in the southwest portion of the City. As this population develops, additional sewer improvements will be required in the District 142 system. A summary of these future improvements is provided in Table 6-9.

ADDITIONAL INVESTIGATIONS

There are several reported overflow locations that the hydraulic model did not replicate with all but one being Category C overflows. It is recommended that LRW conduct CCTV inspections downstream of these locations to determine if there may be a structural cause to the overflows. Pipe diameter and invert/rim elevations should also be obtained to compare to data in the model. A list of these locations is provided in Table 6-10.

Table 6-6

UNDOCUMENTED / MODEL PREDICTED OVERFLOW LOCATIONS

Manhole Location	Volume Lost (mg)
10K109	0.0058
11G096	0.0424
11J053	0.0142
15I067	0.1240
15I068	0.0876
15I069	0.0029
15M053	0.0140
15M106	0.0486
1K012	0.0520
1K068	0.0009
1K151	0.0348
1L097	0.0551
2G019	0.0277
2K077	0.0284
2K143	0.0175
2M028	0.0026
2M034	0.0289
2M060	0.0279
2M085	0.0080
2M108	0.2930
2M110	0.0001
5K047	0.0041
6D025	0.0272
6G006	0.0072
6G061	0.0014
6H028	0.0230
6H049	0.1632
6K003	0.0030
6K046	0.0012
6K047	0.0040
6K079	0.2451
6K122	0.0249
7E001	0.1105
7H124	0.0355
7I009	0.0065
7I048	0.0023
7I050	0.2062
7J018	0.0200
7J036	0.0056
7J065	0.3413
8I006	0.0614
8I066	0.0061
9K072	0.1321

Table 6-7

UNDOCUMENTED / MODELED OVERFLOW IMPROVEMENT PROJECTS (IF REQUIRED)

Project No.	Sub-Basin	Grid Nos.	Length (lf)	Pipe Diameter (in)	Estimated Capital Improvement Cost ($)[1/]
M1	North 60 East (L.R. Airport)	15I & 16I	1,423	10	256,071
M2	Quapaw North	12I	22	10	11,254
M3	Quapaw North	11J	1,090	10	247,806
M4	S of Roosevelt (Battery St.)	10K	400	8	63,142
M5	Barton South	9K	217	10	57,805
M6	District 84	7L	667	8 & 10	122,323
M7	District 84	7K	293	10	54,965
M8	District 84	7K	536	10	143,639
M9	District 84	6K	650	8	91,134
M10	District 119	6K	331	8	65,483
M11	District 119	5K & 6K	1,291	8	259,803
M12	District 84	7J	151	8	18,422
M13	District 84	7J	342	10	65,857
M14	Barton North	7I & 8I	1,448	12	342,389
M15	Barton North	7H & 7I	621	10	185,583
M16	District 119	6H	299	8	59,891
M17	District 119	6H	189	10	44,009
M18	Upper Coleman Creek	6G	755	8	153,229
M19	Upper Coleman Creek	6G	276	8	73,059
M20	Granite Mountain	15M	696	10	115,372
M21	Western Hills	2M	2,231	12/15	546,128
M22	Barrow Add / South Boyle Park	2K	617	21	198,557
M23	Barrow Add / South Boyle Park	2K	333	15	80,611
M24	Barrow Add / South Boyle Park	1K	659	12	144,004
M25	Barrow Add / South Boyle Park	2K	1,290	15	263,454
M26	Barrow Add / South Boyle Park	1K	1,451	15	325,112
M27	Brodie Creek West / Sandpiper	(-)2K & (-)3K	280	18	82,032
M28	Brodie Creek West / Sandpiper	(-)3J	441	10	101,015
M29	Brodie Creek West / Sandpiper	(-)3J	473	10	99,881
M30	Leawood	2G	386	10	81,043
M31	Natural Resource Complex	1G	796	15	244,527
M32	Rose Creek East	11G	855	15	256,765
M33	Allsopp Park North	7E	191	12	48,096
M34	Allsopp Park North	6D	369	10	71,049
	Total		**22,069**		**4,973,508**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*

Table 6-8

SUMMARY OF
CANTRELL ROAD PUMP STATION IMPROVEMENTS

Item	Description	Estimated Capital Improvement Cost ($ Million)1/
Pump Station Upgrade	Includes Mechanical & Electrical Upgrades	6.60
Force Main	Includes New Force Main and Rehab of Existing Force Main	2.22
Total		**8.82**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*

Table 6-9

SUMMARY OF CAPACITY IMPROVEMENTS
FOR FUTURE GROWTH AREA

Item	Description	Estimated Capital Improvement Cost ($ Million)1/
Pipeline Improvements	10,200 lf of 12-inch to 21-inch diameter sewer	3.15
Total		**3.15**

1/ *Estimated Capital Cost is given in 2010 dollars. No cost escalation is included for future years.*

Table 6-10

REPORTED OVERFLOW LOCATIONS
NOT REPLICATED IN MODEL

Manhole Location	Storm Category
3K022	A
13I014	C
11L025	C
6F072	C
6K152	C
7F057	C
13I007	C
13I005	C

ASSUMPTIONS

The alternatives developed and the improvements recommended in this update are based on the following assumptions:

- LRW will complete construction of all mains previously designed but not constructed
- Complete construction of improvements recommended in previous SSES Reports within the last 10 years
- Re-program Adams Field MPS, Arch Street, and Cantrell Road Pump Stations with the wet-weather operating levels recommended in this report

The hydraulic model utilized during this update was updated to reflect the above assumptions. If any of the improvements are not implemented, it may have an impact on the success of the overflow elimination plan.

CHAPTER 7

INFLOW/INFILTRATION REDUCTION MEASURES

This chapter of the report presents an outline for Little Rock Wastewater to continue to implement a corrective maintenance program to address problems related to inflow and infiltration (I/I). The improvement plan outlined in Chapter 6 removes all overflows for the design storm discussed without any credit for reduction in I/I, except in the area tributary to the Cantrell Road Pump Station. However, over time I/I related maintenance problems will increase and an abatement program to remove I/I is recommended.

REDUCTION PLAN

The reduction plan prioritizes basins with the highest amounts of I/I and the ability to impact the capacity improvement plan by reducing the volume of storage required or possibly the size of trunk line improvements. Targeted areas would undertake comprehensive manhole inspection, smoke testing of sewer mains, dyed water flooding to confirm cross connections or main line defects, and selected closed-circuit television (CCTV) inspection warranted from field inspections. The program should also address private defects, as recent sanitary sewer evaluation studies have indicated that over 50 percent of I/I can be attributed to private sewer laterals, cleanouts, or illegal tie-ins to the public sewer system.

Typically, a comprehensive evaluation of a sanitary system can identify 50 to 60 percent of monitored rain induced I/I. In addition, if the recommended rehabilitation is completed that was identified during the field investigations, a 30 to 50 percent reduction in the peak monitored inflow rate can be seen during post flow monitoring activities.

Little Rock Wastewater has undertaken several sanitary sewer evaluation studies since the original SECAP in 2002. The areas studied were prioritized as those with the highest amounts of I/I entering the system from the City-wide flow monitoring conducted in 2000. Based on the 2009 flow data Table 7-1 lists the basins which experience the highest amounts of inflow per 1,000 linear feet of pipe, in a prioritized order. In addition, the basins in which RJN has conducted evaluation studies since 2004 have been denoted, as well as the basins that have undergone some rehabilitation measures since 2002. Figure 7.1 graphically depicts the service area of Little Rock Wastewater and those areas experiencing the greatest impact from inflow.

It is recommended that LRW continue their I/I Reduction Program in each of the basins that have a unit inflow ratio greater than 12,000 gpd/1,000 lf of sewer.

Table 7-1

WET WEATHER REACTION OF BASINS

Meter Basin	Basin Peak 1-Year/60-Minute Inflow per 1,000 lf (gpd/1000 lf)	Basin Average Daily Dry-Weather Flow (mgd)	Cumulative Peak Wet-Weather Flow (mgd)	Peaking Factor
L126	115,791	0.388	3.961	10.21
L112	110,765	2.414	21.850	9.05
L010R	107,488	6.881	46.202	6.71
L002	91,543	0.118	1.712	14.51
L125 (Country Club)[1/]	88,897	0.312	5.609	17.98
L023	86,830	0.133	2.798	21.04
L028 (Barton South)[1/]	82,583	0.295	7.345	24.90
L063	82,155	0.329	6.003	18.25
L034	80,040	1.276	26.734	20.95
L037	79,447	0.336	2.811	8.37
L100	76,558	0.373	18.619	49.92
L007 (Leawood)[1/]	75,398	0.326	5.616	17.23
L122 & L123	74,069	14.137	60.221	4.26
L029 (Barton North)[1/]	70,050	0.184	4.300	23.37
L012	64,552	0.179	3.086	17.24
L010	61,910	0.462	7.951	17.21
L036	59,501	0.161	1.980	12.30
L003 (Echo Valley)[1/]	57,272	0.541	6.621	12.24
L004 (Natural Resources)[2/]	56,428	0.188	2.434	12.95
L001	56,021	1.393	14.871	10.68
L025 (Swaggerty Creek)[1/]	54,496	0.365	9.121	24.99
L014 (Pleasant Valley)[1/]	53,486	0.549	4.581	8.34
L032	52,149	2.511	11.621	4.63
L035	51,009	0.267	2.278	8.53
L107	50,851	0.622	6.021	9.68
L058 (East Jimmerson)[2/]	49,375	0.769	11.823	15.37
L062 (Granite Mountain)[1/]	48,600	0.125	1.319	10.55
L006	41,652	0.089	2.729	30.66
L105	39,542	1.779	13.682	7.69
L059 (West Jimmerson)[1/]	38,824	0.706	6.453	9.14
L109	37,452	0.222	2.940	13.24
L117	36,263	0.827	45.148	54.59

1/ *SSES Work has been performed; rehabilitation has not been completed.*

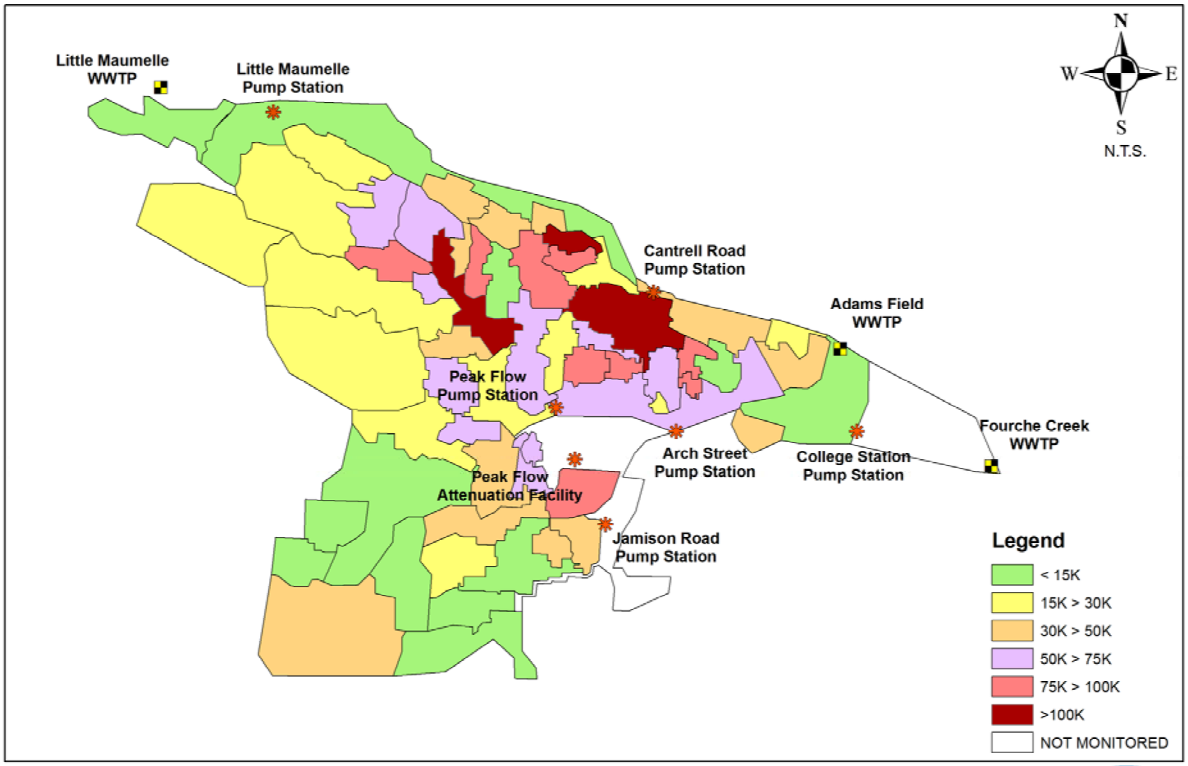
2/ *SSES Work has been performed; rehabilitation work has been completed or is underway.*

Table 7-1 (Cont.)

WET WEATHER REACTION OF BASINS

Meter Basin	Basin Peak 1-Year/60-Minute Inflow per 1,000 lf (gpd/1000 lf)	Basin Average Daily Dry-Weather Flow (mgd)	Cumulative Peak Wet-Weather Flow (mgd)	Peaking Factor
L009	36,049	0.312	3.553	11.39
L044	32,174	0.974	4.693	4.82
L057	31,328	0.095	3.394	35.73
L004R	31,043	1.558	23.223	14.91
L005	30,229	2.579	13.831	5.36
L050	30,209	8.109	54.221	6.69
L055 (Allsopp)[2/]	29,342	1.311	7.351	5.61
L043	29,227	0.455	16.507	36.28
L026 (Swaggerty Creek)[1/]	28,307	0.056	1.220	21.79
L101	27,855	0.433	2.055	4.75
L015 (Pleasant Valley)[1/]	27,593	0.258	1.487	5.76
L031	26,363	0.102	3.752	36.78
L118	25,604	0.244	5.099	20.90
L002R	25,384	8.032	44.382	5.53
L008R	24,805	3.685	18.351	4.98
L009R	24,044	10.221	26.884	2.63
L113	23,673	0.518	5.299	10.23
L102	22,205	1.120	9.693	8.65
L114 (Upper Hinson)[2/]	19,246	0.989	12.911	13.05
L018	18,424	1.422	14.161	9.96
L124	14,842	1.132	10.759	9.50
L120	13,365	0.321	3.284	10.23
L116	12,505	0.311	1.411	4.54
L111	10,976	0.112	2.313	20.65
L108	10,330	0.384	4.919	12.81
L110	10,259	0.259	2.597	10.03
L103	9,593	1.791	14.221	7.94
L021 (Bond)	9,477	0.626	2.398	3.83
L005R	8,414	3.503	10.921	3.12
L008	7,850	0.062	1.641	26.47
L003R	7,094	12.983	35.497	2.73
L119	4,223	0.218	1.092	5.01

1/ *SSES Work has been performed; rehabilitation has not been completed.*
2/ *SSES Work has been performed; rehabilitation work has been completed or is underway.*



BASIN PEAK 1 YEAR / 60 MINUTE INFLOW PER 1000 LF
FIGURE 7.1

An area that is recommended for I/I reduction at the initiation of the improvement program are the basins conveying their flow to the Cantrell Road Pump Station and in particular Basin 112. Basin 112 consists primarily of commercial and government facilities, such as hospitals and the state capital. The sewer flow generated in this area drains to the Cantrell Road Pump Station and the wet-weather peaking factor is 9.05. Other basins have higher peaking factors, however 19.436 million gallons of wet-weather flow volume occurs in this basin for a typical one-year storm event. This area also exhibits an inflow ratio of 110,765 gallons of inflow per one thousand feet of pipe during a one-year storm event. The response to rainfall is quick, which most likely means there are direct or indirect connections to the storm sewer, a high number of illegal roof drains, or other larger sources of inflow. However, the recommended rehabilitation has not been completed. In addition, the interceptor parallel to Rebsman Park that leads to Cantrell Road Pump Station is located in a swampy area and would be susceptible to large amounts of I/I. This interceptor and the lines leading into it have not been studied and are recommended for an evaluation study. These areas impact the amount of future wet-weather storage needed at Cantrell Road Pump Station and affect the areas downstream of the lift station along the Riverfront near the William Clinton Presidential Library.

Several basins contributing flow to the Cantrell Road Pump Station have had complete evaluation studies, such as East Jimmerson Creek, Jimmerson Creek, Allsopp, and Country Club Basins.

REHABILITATION DESIGN AND CONSTRUCTION

It is recommended that during the rehabilitation design phase LRW utilize design measures to mitigate migration of I/I from a repaired section of sewer to another section of unrepaired sewer. One effective measure is to look at all identified I/I defects in a pipeline or manhole and select a rehabilitation method that fits each component. Another very effective measure is to utilize clay dams at key points in the rehabilitation program. The clay dams will prevent the migration of I/I from one section of pipeline to another. It is also recommended that effectiveness testing of each repair be completed as part of the construction program. Effectiveness testing may include vacuum testing and dyed water flooding of new or rehabilitated pipelines. The effectiveness testing is a method of confirming that the I/I removal was successful and that migration did not occur. Post-rehabilitation flow monitoring should also be completed to verify the actual level of I/I reduction.