Report on

Integrated
2012
Wastewater
Utilities
Plan

Project No. 54793

2012

Report on

Integrated 2012 Wastewater Utilities Plan

City of Lawrence, Kansas

Project No. 54793

2012

July 25, 2012

Mr. David Wagner
Utilities Director
Department of Utilities
720 West 3rd Street
Lawrence, KS 66044-0708

Re: Wastewater Facilities Master Plan
City of Lawrence, Kansas
B&M Project No. 54793

Dear Mr. Wagner:

Burns & McDonnell in association with BG Consultants is pleased to submit our report titled Wastewater Facilities Master Plan in accordance with our engineering services agreement with the City of Lawrence. The report consists of two volumes as follows:

1. Executive Summary
2. Master Plan Report

This volume is the detailed master plan report. The report consists of technical memoranda completed at various stages of the plan development. The key recommendations of the plan are as follows:

- Implement an 8 year infiltration/inflow reduction program within a targeted area of the collection system that includes the oldest parts of the system close to the Kansas River Wastewater Treatment Plant (WWTP), with the objective of reducing peak wet weather infiltration inflow rates by approximately 19 MGD.
- Construct gravity sewers, relief sewers, and pumping station and force main capacity expansions needed to convey peak flow rates occurring during wet weather periods.
- Construct a new pumping station and force mains to divert a portion of dry and wet weather flows to a new Wakarusa WWTP. Final planning for these facilities should begin by the time the utility service area population is 96,000 so they are in operation before flows to the Kansas River WWTP reach its design capacity. An initial dry weather flow capacity for the new Wakarusa WWTP of 2 MGD would be sufficient for handling flow rates forecast to occur through year 2030.
- Complete a program of clay pipe and brick manhole replacement to insure the long term integrity of the collection system.
- Plan and budget for additions to the collection system that are necessary for extending service to areas outside the existing utility service area as new development occurs.

Burns & McDonnell
SINCE 1898

- Plan and budget for improvements to the Kansas River WWTP that will be necessary for meeting new regulatory requirements such as nutrient (nitrogen and phosphorus) removal.

This master plan was developed to be a living document, subject to revision as dictated by the timing and direction of future regulatory actions, and actual patterns of future growth and development. The collection system computer hydraulic model prepared for this master plan is one tool that may be used to assess future conditions that may differ from those assumed by the master plan.

We sincerely appreciate the assistance and direction received from your staff, including Mike Lawless, P. E. and Philip Ciesielski, P. E., throughout the development of this master plan. We would be pleased to assist you with implementing the recommendations of this plan. Thank you for this opportunity to serve the City of Lawrence.

Sincerely,

Stephen A. Yonker, P. E.
Project Manager

Jeffrey J. Keller, P. E.
Project Review Engineer

John P. Mitchell, P. E.
Project Principal

SAY/say

INDEX AND CERTIFICATION

Report on
Wastewater Facilities Master Plan
City of Lawrence, Kansas

Project 54793

Report Index

Certification

I hereby certify, as a Professional Engineer in the state of Kansas, that the information in this document was assembled under my direct personal charge. This report is not intended or represented to be suitable for reuse by the City of Lawrence, Kansas or others without specific verification or adaptation by the Engineer. This certification is made in accordance with the provisions of the laws and rules of the Kansas under Kansas Administrative Code.

JEFFREY J. KELLER LICENSED 19668 7/18/12 KANSAS PROFESSIONAL ENGINEER

Jeffrey J. Keller, P.E. (insert state & license)
Date: 7/18/2012
(Reproductions are not valid unless signed, dated, and embossed with Engineer's seal)

Technical Memorandum No. 1
Initial Services

Wastewater Facilities Master Plan

for

Lawrence, Kansas

City of Lawrence, Kansas

BMcD Project No. 54793

City P.O. 07629

Burns & McDonnell Engineering Company, Inc.
9400 Ward Parkway
Kansas City, MO 64114

City of Lawrence, Kansas

Wastewater Facilities Master Plan Technical Memorandum No. 1

Table of Contents

List of Tables

List of Figures

* = follows page number

Appendices

Wastewater Master Plan List of Abbreviations

BOD	Biochemical Oxygen Demand
CIPP	Cured-in-Place Pipe
DWF	Dry Weather Flow
DWI	Dry Weather Infiltration
ft	Foot
GIS	Geographical Information System
gpd	Gallons per Day
IDM	Inch-Diameter Mile
I/I	Infiltration/Inflow
in	Inch
KDHE	Kansas Department of Health and Environment
KRWWTP	Kansas River Wastewater Treatment Plant
MG	Million Gallons
MGD	Million Gallons per Day
PF	Peaking Factor
PVC	Poly-vinyl Chloride Pipe
ppd	Pounds Per Day
RDII	Rainfall-Derived Infiltration/Inflow
RTK	Rainfall, Time to Peak, and Recession Response Parameters
SSOAP	Sanitary Sewer Overflow Analysis Planning Program
TAZ	Traffic Analysis Zone
TM	Technical Memorandum
VCP	Vitrified Clay Pipe
WWTP	Wastewater Treatment Plant
Wakarusa	Wakarusa Wastewater Treatment Plant

A. Introduction

Technical Memorandum No. 1 is a summary of initial services completed in partial fulfillment of the Lawrence, Kansas Wastewater Facilities Master Plan. The goals of the initial services were to establish the project team (kick-off meeting), obtain the necessary data for performing the master plan, the selection of the collection system modeling software, and the population forecast to be used for the master plan

B. Kick-Off Meeting

The kick-off meeting, commencing the Lawrence, Kansas Wastewater Facilities Master Plan project, was held on December 10, 2009, at the wastewater treatment plant. The agenda is provided in Appendix 1-A. Attendees included representatives from the City of Lawrence (City), BG Consultants (BG) and Burns & McDonnell Engineering (BMcD) and are listed below:

Representative	Org	Contact #	Contact Email
Mike Lawless	City	785-423-3306	mlawless@ci.lawrence.ks.us
Philip Ciesielski	City	785-423-7114	pciesielski@ci.lawrence.ks.us
John Bertrand	City	785-764-6136	jbertrand@ci.lawrence.ks.us
Clint Miller	City	785-832-7827	cmiller@ci.lawrence.ks.us
David R. Guntert	City	785-832-3158	dguntert@lawrence.ks.us
Dave Wagner	City	785-832-7800	dwagner@ci.lawrence.ks.us
Mark Hegeman	City	785-423-3380	mhegeman@ci.lawrence.ks.us
David Hamby	BG	785-749-4474	davidh@bgcons.com
Jeff Keller	BMcD	816-822-4371	jkeller@burnsmcd.com

C. City of Lawrence Data

A preliminary data request was made including the items outlined in the master plan engineering services agreement. Data and information received as a result of this request includes asset data for building the model, relevant background reports related to the Kansas River Wastewater Treatment Plant and the wastewater collection system, available rain gauge and flow meter data, and pertinent GIS information necessary for an intelligent model. A complete listing of data received and utilized by this master plan is provided in Appendix 1-B.

D. Wastewater Collection System Model Selection

The selection of modeling software was narrowed to two modeling vendors whose products met the City's minimum criteria, which are:

- Commercially available, non-proprietary.
- Provide dynamic (flow versus time) analysis.
- Interface with GIS.
- Availability of both water distribution system and wastewater collection system modeling software from the same vendor.

Bentley Systems and MWH Soft were selected vendors to demonstrate their respective software to the Utilities and Public Works Department staff and Burns & McDonnell. Ultimately, the City selected Bentley Systems after the demonstration and verifying references. City memorandum dated April 19, 2010 is included in Appendix 1-C for further detail of the software selection process.

E. Study Area Description and Population

1. Study Area Description

The master plan study area as delineated by the City of Lawrence Planning Department is shown on Figure 1.1. A memorandum dated April 27, 2010 from City staff to the City Commission concerning the plan study area is included in Appendix 1-D. The planning area boundary is identical to the Planning Department Urban Growth Area 2030 Boundary, with the exception of the North Lawrence area north of the Kansas River where the master planning boundary lies inside of the Urban Growth Area 2030 Boundary. The current wastewater utility service area shown on Figure 1.1 includes some areas immediately adjacent to sewered areas that are considered to be readily served by minor extensions of the existing collection system. In addition to the current wastewater utility service area, the master planning area includes unsewered areas south of the Wakarusa River and the site of the future Wakarusa Treatment Plant.

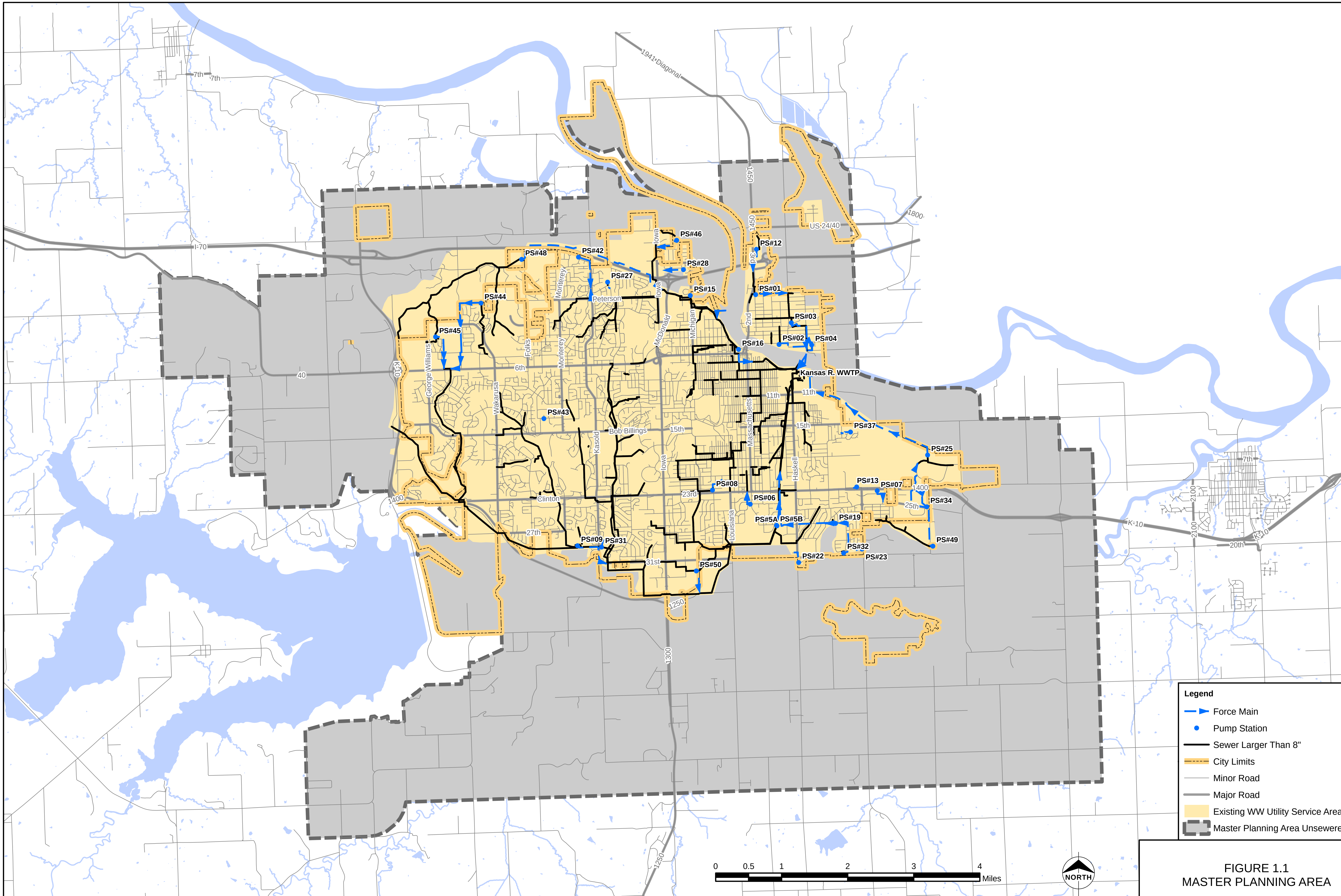


FIGURE 1.1
MASTER PLANNING AREA

2. Existing Population and Population Forecast

The City of Lawrence Planning Department developed the estimate of existing population and the forecast of future population to be used by this master plan for the year 2030 planning period. A memorandum dated April 27, 2010 from City staff to the City Commission concerning existing and forecast population is included in Appendix 1-D. A public comment process began in May 2010, whereby the City requested input from various stakeholders including organizations, individuals, schools and other entities as follows:

- Chamber of Commerce - talked to reps there and emailed the information. No comment returned.
- Perry - Lecompton Schools - voice mail to Dr. Yoder, who was out - emailed information again. No comment returned.
- Lawrence Public Schools - Rick Doll - emailed information to rdoll@usd497.org. No comment returned.
- Gould Evans - left message and emailed again to Steve Clark. No comment returned.
- Land Plan Engineering - talked to Tim Herndon and emailed again. No comment returned.
- Paul Werner Architects - left message. Call not returned.
- Peridian Group - talked to Lance Johnson and emailed again. No comment returned.
- PEC - left message for Jim Martin. No return call received. Attended study session.
- Treanor Architects - left message for Mike Treanor. No return call received.
- Bartlett and West - left message for Stan Meyers. No return call received.
- KU/Facility Operations - left message for Jim Modig and sent email again. Received email with comments.
- Lawrence Association of Neighborhoods - Gwen Klingenberg - wrong number posted on their web site. Sent email again. No comment returned.
- Lawrence Home Builders Association - talked to Bobbie Flory and sent again. No comment returned.
- Kansas Water Office - Talked to Cathy Tucker - Vogel, who is in KDHE now. She called me back and emailed to her again. She also was going to show it around her office. No comment returned.

- KU Civil Engineering - Craig Adams - left message. He called back. He is very interested in water quality, but not so much so in the planning and infrastructure. He welcomes opportunities for KUCE and Lawrence Utilities to work on projects together.
- Rod Geisler - KDHE - talked to him. He did not have comments but commended us for doing master planning.
- First Management - Talked to Robert Green and emailed him a copy. No comments returned.
- Gene Fritzel Construction - Left message and emailed to him. No comments received.
- Steve Schwada - left message and sent email with link. No comments received.
- Michael Stultz - left message and sent email with link. No comments received.

The University of Kansas Facility Operations forwarded to the City 25-year water use projections for the Main and West campuses which is included in Appendix 1-E, along with meeting notes from a follow-up meeting between City staff and University of Kansas representatives.

Table 1.1 summarizes existing and future population within the current service area and within the extensions of the service area forecast for 2020 and 2030, and within the planning area.

Table 1.1
Wastewater Utility Service Area and Master Planning Area
Population Forecasts

Year	Utility Service Area	Master Planning Area
2010	92,727	94,564
2020	106,667	113,051
2030	119,529	129,176
Buildout	251,971	251,971

New development is forecast to occur inside and outside of the current utility service area in accordance with the 2020 and 2030 service area population projection as shown on Figure 1.2.

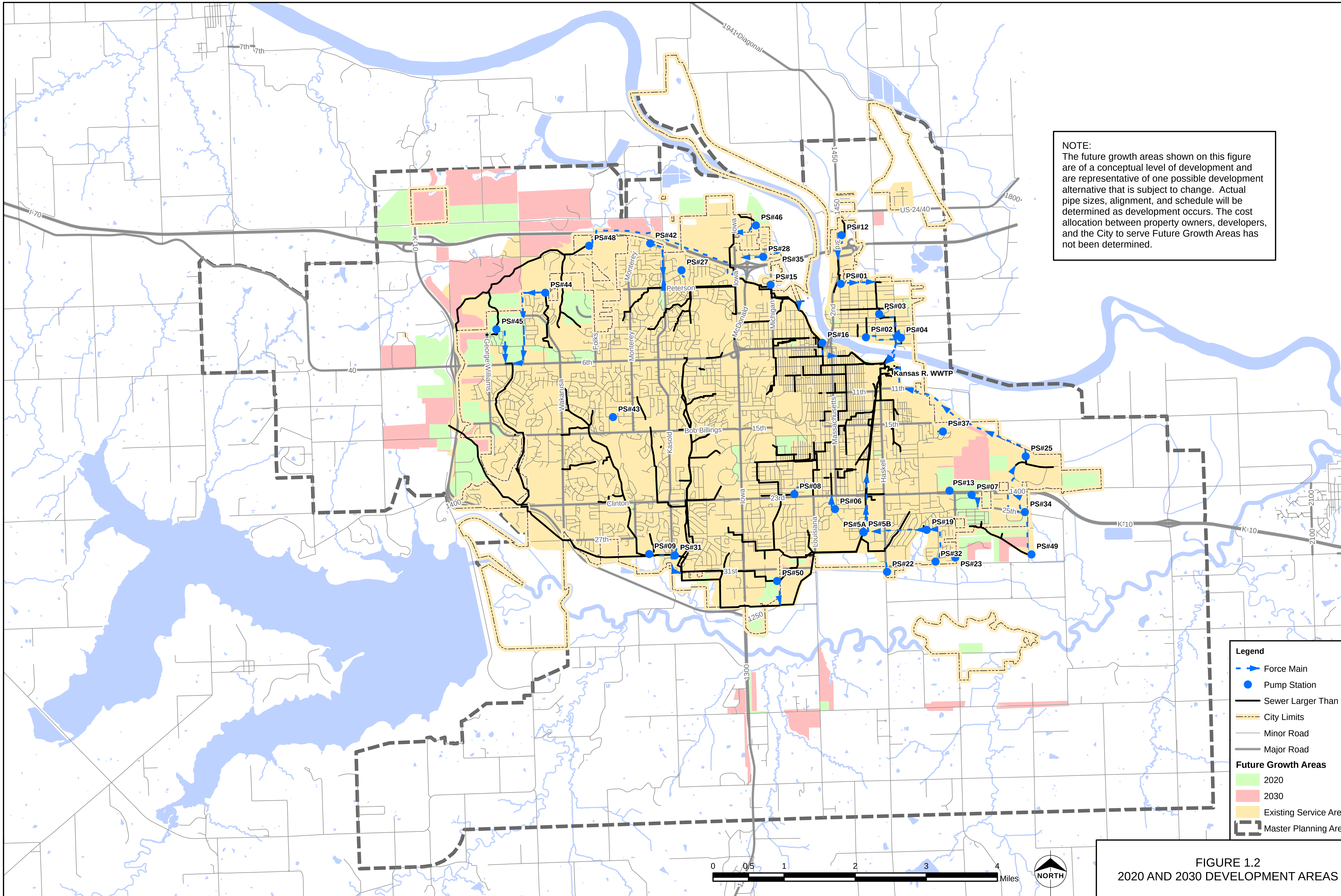


FIGURE 1.2
2020 AND 2030 DEVELOPMENT AREAS

Appendix 1-A
Kick-off Meeting Agenda

AGENDA

KICK-OFF MEETING

Wednesday, December 10, 2009
10:00 a.m. at WWTP Conference Room

1. Project Objectives
2. City Expectations
3. Project Team
4. Schedule
5. Project Control Plan
6. Review Data Request / City Information
   a. Data needs memorandum
   b. Mapping and GIS
   c. Flow and Rainfall Data
   d. Facility Data
   e. Planning Information
   f. Plans to secure any missing information
7. Software Selection
   a. Dec. 16th – Bentley Software (SewerGEMS & WaterGEMS)
   b. Dec. 17th – MWH-Soft (InfoSewer/SWMM & InfoWater, Suite)
8. Public Relations, Developing Community Support, and Involving Stakeholders
9. Service Area definition
10. Use of SharePoint web site
11. Flow and Rainfall Monitoring Data

Appendix 1-B
City Data

MEMORANDUM
Lawrence, Kansas Wastewater Facilities Master Plan
BMcD Project 54793
City P.O. 072629

Date: December 8, 2009

To: Mike Lawless
From: Jon Gray, Jeff Keller
Re: Data Request

Data to be provided by the City for the Master Plan are described in the contract scope of work in task no. 5. These items are listed below with additional text or comment shown in italics.

5.1. Copy of any correspondence with KDHE related to the Project. *(Note: There may be no pertinent correspondence at this time.)*

5.2 Any other required information and prompt review of draft technical memoranda.

5.2. Assistance by placing at CONSULTANT's disposal all available information pertinent to the assignment, including previous reports and any other data relative thereto. CONSULTANT shall rely on information made available by the CITY as accurate without independent verification.

5.3. Electronic copy of necessary GIS Data for the Project. *GIS data may be provided in the form of an ESRI Personal Geodatabase file.. After the initial data delivery, it may be more convenient to provide additional information in ESRI shapefiles for Burns & McDonnell to add to the project Geodatabase.*

5.3.1 Sanitary Sewer System Facilities information required for the hydraulic model, including manholes, sewer lines, lift stations and wet wells, force mains (merged to single lines), and outlet to the WWTP. All entities will have unique ID's and be attributed with their data. CITY will attribute lift station records with data required for the pumps and wet-well.

5.3.1.1. *Additional sewer facilities information not in the GIS including pump curves for all pumps, and lift station layout drawings for any complex stations.*

5.3.2 Maintenance Management History records (GBA Master Series) for sewer maintenance for the past 5 years. CITY will extract history records to GIS format and attribute manhole and sewer pipe records with the total number of events by type of maintenance.

5.3.3 Planning information including Existing and Future Land Use information, City Limits and Growth Area Boundaries, drainage basins and sub-basins, and TAZ or other population projection areas attributed with available past and projected populations. CITY will overlay planning information layers and attribute sub-basins with total land use by major use type, existing and projected populations, and total acreage.

5.3.4 Base mapping information for use in preparing report exhibits and layout of new sewers including but not limited to roads and highways, waterways, urban boundaries, 2006 elevation contours (2-foot), and NRCS soils.

5.3.5 Other base GIS information in CITY's files that may be deemed necessary during the Project.

5.4. Electronic copy of all of the City's rainfall and flow monitoring data files stored on the Marsh McBirney web site. *There should be one file for each rain gage and for each flow monitor.*

5.5. CITYs most recent population growth and development projections and assist CONSULTANT in developing a range of realistic growth scenarios. CITY will provide existing ultimate build-out population projection from planned land use and population densities. CITY will review projected growth as developed through Public Participation by sub-area to refine the projected timing and location of growth, according to potential agreed-on development needs, development timing, available utilities, and other influences, and provide projected growth in GIS format.

5.6. Electronic Maintenance Management History records (GBA Master Series) for sewer maintenance for the past 5 years in GIS format, to include total counts of sewer maintenance events by type for each pipe and manhole.

5.7. Operating records for the wastewater treatment plant and lift stations. *Plant records to include each day's total and peak influent flows since 2003, and lift station records to include each day's daily total pumpage or running hours (depending on availability), and daily peak pumping rate, for 2008 and 2009 to date.*

5.8. Purchase of one copy of selected software for use on the Project

5.9. One copy of paper sewer atlas map set. One copy of any other sewer maps that may be useful.

5.10. One copy of any prior studies and reports pertinent to this project.

.

Appendix 1-C
Model Software Purchase Memorandum

Memorandum
City of Lawrence
Utilities Department

TO:	**David L. Corliss – City Manager** **Cynthia Boecker - Assistant City Manager** **Diane Stoddard - Assistant City Manager**
FROM:	**Mike Lawless – Asst. Director of Utilities**
CC:	**Dave Wagner – Director of Utilities** **Philip Ciesielski – Asst. Director of Utilities** **Beth Frailey Krishtalka – Management Analyst**
Date:	**April 19, 2010**
RE:	**Agenda Item – Purchase wastewater and water modeling software from Bentley Systems, Inc.**

Please include the following item on the City Commission Agenda for consideration at the April 27, 2010 meeting:

Authorize Staff to purchase 1 license each of wastewater and water modeling software from Bentley Systems, Inc.

Project Description

The Wastewater Master Plan scope of services included the City selecting and purchasing modeling software to be used for this project as well as, the Water Master Plan. Selection of the software vendors to provide demonstrations started with establishing base criteria for the software and vendors to meet. The criteria included the following:

1. Commercially available
2. Vendor could supply dynamic sewer and water system models
3. GIS Interface

Only two software vendors met the criteria: Bentley Systems and MWH Soft. Each vendor was invited to present a demonstration of the software to Utilities and Public Works Department staff as well as, Burns & McDonnell. Utilities GIS data for wastewater and water was provided to the vendors. Each vendor was provided 4 hours to demonstrate the capabilities and functionality of their products.

The demonstrations of each software were very comparable. Staff considered a number of different criteria in the evaluation of the software, some of which are listed below.

• Hydraulic Features	Data Structure
• GIS Interface	Multiple Platform
• Additional Modules	Output Presentation
• Virtual Server Compatible	VFD Capabilities
• Local Users	Skeletonized Data Retention
• Initial Cost	Maintenance Cost

After considerable discussion and checking references, staff concluded that the Bentley Systems software will provide the best and most cost effective solution for water and sewer system modeling for the City of Lawrence.

	Initial Cost	5 Year Cost (Purchase & Maintenance)
	1 License Each Water & Sewer	1 License Each Water & Sewer
MWH	$ 42,000	$ 81,000
Bentley	$ 35,350	$ 59,350

Project Funding:
Funding for the sewer modeling software was included in the 2009 Capital Improvement Program and funding for the water modeling software was included in the 2010 Capital Improvement Program.

Action Request: Authorize staff to purchase 1 license each of wastewater and water modeling software from Bentley Systems, Inc.

Thank you for your assistance. Please advise if you have any questions.

Appendix 1-D
Planning Area Boundary and Population Growth Forecast Memorandum

Memorandum
City of Lawrence

TO: **David L. Corliss, City Manager**

FROM: **Mike Lawless, Assistant Director, Utilities**
Scott McCullough, Director, Planning and Development Services

CC: **Cynthia Wagner, Assistant City Manager**
Diane Stoddard, Assistant City Manager
Dave Wagner, Utilities Director

Date: **For April 27, 2010 City Commission Meeting**

RE: **Utilities Master Planning Growth Projections**

On November 11, 2009 notice to proceed was issued for an engineering services contract for the Wastewater Master Plan (Plan). The Plan will provide an evaluation of the wastewater collection and treatment systems for improvements to serve potential development planned through the year 2030. The Plan will use existing population for 2010 and population projections for 2020 and 2030 as the input data for the design years. The Plan will provide flow/development triggers for the construction of system improvements.

Plan Boundary

To develop the flow projections for the design years, a defined boundary with the population estimates and distribution of the population within the boundary are needed. Utilities and Planning staff have met several times to discuss the planning boundaries of the project as well as the logistics of preparing the underlying data needed for the population and growth projections for the design years. Several adopted sector plans, including the Southeast Area Plan, the K-10 & Farmer's Turnpike Plan, the West of K-10 Plan, and the Northeast Sector Plan that is currently underway, have guided development of an appropriate boundary for the Plan. In addition to the planning boundaries, the drainage basins are physical boundaries that also effect development and the results of the Plan.

A map of the current Urban Growth Area, basin boundaries, and sector plans is provided to show how these boundaries overlay each other. As a result of these overlays and staff discussions, a logical and justifiable boundary is proposed for the Wastewater Master Plan as shown on the map. This planning boundary, the population projections, and distribution of the population will allow distribution of the basin flows needed for the project. While this boundary is logical based on the discussion above there is always the possibility that a development request could be made outside of the planning area.

Population Projections

Horizon 2020 sets out three population projections using July 1st 2000 Census data of 80,508 for the city of Lawrence: Low, Medium, and High.

Population Projections from Horizon 2020			
Horizon 2020 Projections	**2010**	**2020**	**2030**
Low	88,961	100,076	111,191
Medium	95,178	110,406	125,635
High	99,013	122,394	151,296

Population projection methods primarily rely on trend data and the most accurate projections can only be completed every decade after the Census Bureau releases the Decennial Census Data. Staff will release new population projections after the 2010 Census numbers are calculated and disclosed for public use.

Planning Staff has analyzed the effects that short and long-term growth trends would have on the population projections. Given recent population trends, staff is of the opinion that Lawrence is between the Low and Medium population projections from Horizon 2020 and the department currently projects Lawrence to reach between 112,000 and 126,000 people in 2030. A 2030 population of approximately 125,000 for Lawrence is used to build the growth scenarios for the Wastewater Master Plan.

Population Distribution - Future Development Trends and Growth Areas

To determine appropriate distribution of the 2030 population, staff used existing data and made assumptions about the amount of residential dwelling unit inventory the city of Lawrence currently has and where the likely growth will occur based on historic patterns and identified opportunities and constraints. The following exercise will assist the consultants as they embark on the Plan update.

Staff used census population data, building permit trend data and information from meetings with owners and consultants on specific properties over the last few years to make assumptions about the number of dwelling units that are approved for construction or could be available with the appropriate land use approvals granted and infrastructure extended. Staff concludes that there are approximately 5,100 approved or potentially approved residential units available in the city limits currently. Please see map for locations of approved and not yet, but potentially, approved residential units.

A range of population growth, based on *Horizon 2020* projections, was used to draw conclusions as to the number of years of current or potential residential inventory currently within the city. The data does not differentiate between single-family, duplex, and multi-family structures and so any one of these types of residential units may be more or less under-represented in the exercise.

Build Out Table			
Population / Year	Assumed Persons / Unit	Units Occupied / Year	Years to Build Out Approved and Potentially Approved Inventory of 5,100 units (City Only)
LOW - 1,000 / year	2.3 persons / unit	435	11.7 years
MED. - 1,500 / year	2.3 persons / unit	650	7.8 years

Notes

1. The numbers in the table are approximations and have been rounded for ease of computation. Alignment with the Residential Inventory Analysis, authored by Roger Zalneraitis, is not possible since that memo tracks "lots" and this memo uses "units" (several units can be constructed on a single lot in some instances – duplexes, triplexes, multi-dwelling).
2. There is an assumed potential for approximately 5,100 dwelling units in the city limits including available lots and assuming densities on unplatted parcels that could be served.
   a. These units could serve a population of 11,700 new residents
3. Of the 5,100 units, there are 1,335 lots currently vacant with infrastructure available to serve them.
4. There are several infill and fringe areas that are in the concept stage and the anticipated number of units is currently unknown. The following areas were not assigned a unit count but are on the development radar – the area east of The Exchange Apartments, several lots downtown, N. Lawrence redevelopment near Johnny's Tavern, mixed use potential near the Oread Hotel, several fringe areas, etc. These areas were not included in the total unit count used in the calculations in the table and so the actual potential for units in the table may be low.

Discussion

The table and exercise above concludes that there is approximately a decade's worth of existing and potential residential inventory of building sites within the city limits assuming current absorption rates; however, adequate infrastructure may not be in place to serve all of these areas at this time. This is an overly simplistic view, however, because it does not differentiate between housing types, a level of detail that could be investigated if the commission desires but may not be necessary for the purposes of the Wastewater Master Plan update.

This exercise begs several questions about growth and its impact on infrastructure – roads, sewer, water, and even outside providers – electric, gas, cable, cellular, etc.

1. Is there currently an appropriate amount of residential inventory for the community?

   Historically, the market has dictated the level of residential inventory in the community and the city has not established a certain level of "healthy" residential inventory. It is good to track the current inventory over time to understand the historic rates, but it is assumed that the inventory will cycle through periods of growth.

2. What is the design capacity of the current wastewater treatment plant? When must the City begin construction of the Wakarusa Water Reclamation Facility (WWRF)?

   The Utilities Department reports that the design population equivalent that can be served by the wastewater treatment plant located on East 8th Street is 100,000. However, for a number of reasons the City should not wait until that number is reached to begin construction of the WWRF. The Utilities Department believes the WWRF should be completed at a population equivalent of 98,000. The WWRF's design and construction is estimated to take up to five years to complete. The 2008 population estimate for the city was determined to be 90,866. The upcoming recommendations from the Wastewater Master Plan will be very important in determining the timing and scope of the necessary WWRF.

3. Assuming a 10-year inventory of residential locations, it is still appropriate to plan for future growth. Where will growth likely occur given the opportunities and constraints specific to this community?

   Providing sewer and water are only two components of setting a framework for growth. Other opportunities and constraints to development include the following:

   - West of K-10 – Expected **High** rate of growth. Growth in this area aligns with the historic growth pattern of Lawrence and would take advantage of K-10 and I-70 access and inclusion in the Lawrence school district. The West of K-10 Plan established a policy for not permitting development for a large portion of this area until a financing plan and a commitment to construct an interchange at 15th Street/Bob Billings Parkway is established.
   - K-10 and Farmer's Turnpike Sector Plan area – Expected **Medium** rate of growth. The area north of I-70 along Farmer's Turnpike has been planned for significant employment center growth. Demand for residential growth would need to be high to develop some portions of this area with sewers given the makeup of the watersheds. Residential growth in this area is not expected to occur at a high rate.
   - Northeast Sector Plan area – Expected **Low** rate of growth. The Grant Township area is an area currently undergoing sector planning to determine the level of future urbanization. Historically, this has been a very slow growth area as it is constrained by floodplain and other elements that make it less desirable to urbanize.
   - East – Expected **Low** rate of growth. Challenging topography, limited highway access, floodplain, and moving too far downstream of the treatment facilities all constrain development to the east along K-10 Highway. Urbanizing within the Southeast Sector Plan area is anticipated, but developing east of this plan's boundaries may not be feasible.
   - South – Expected **High** rate of growth. South of the Wakarusa River, opportunities exist to take advantage of a new treatment facility and the Highway 59 improvements. The area is within the Lawrence school district

and staff believes this could be a significant growth area if the market demands it after the new treatment facility is constructed. Sector planning this area is included on the long range work plan for the department.

- Infill – Expected **Low** rate of growth. While there is opportunity to develop and redevelop certain areas of the community, this will not play a significant role in the long term growth projections for Lawrence. Infill and higher density redevelopment is considered the most efficient use of existing infrastructure but would only provide a small fraction of inventory needed to support the anticipated growth over the coming decades.

While capacity issues can be resolved with the new WWRF, decisions about where to establish water, sewer and road infrastructure will need to be made within the next 10 years. The Utilities Department indicates that once the WWRF is online, projects will continually need to be *balanced* in order to optimize the system. Growth decisions also impact other public services – street maintenance, police, fire, solid waste, and other general government services that must expand to keep up with the growth demand.

Plan Scenarios

Staff believes that the consultant should develop wastewater infrastructure solutions to serve the following three (3) scenarios:

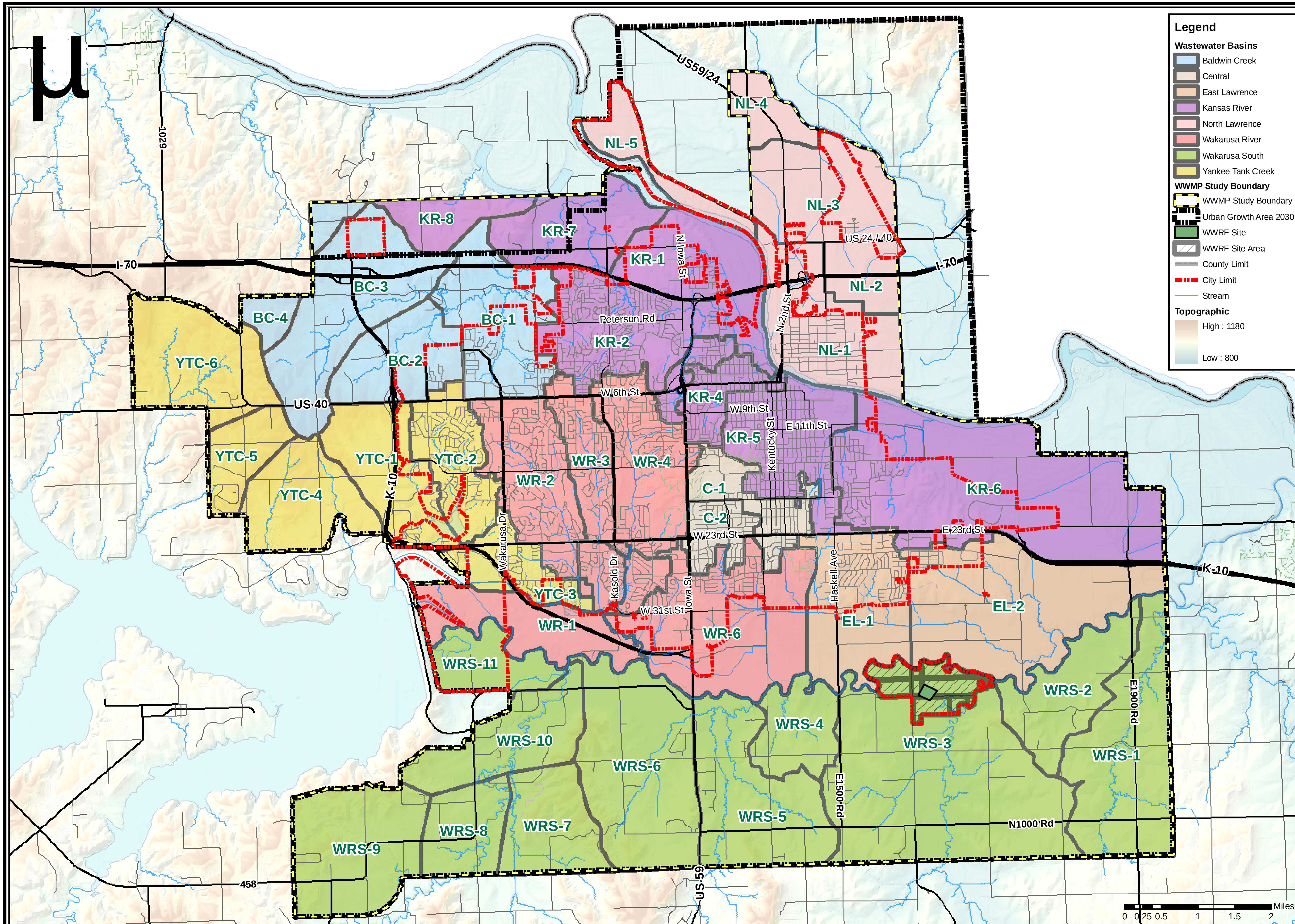
a) **Scenario 2020:** Using the approved sector plans and other assumptions about future growth, disburse the projected 2020 population within the Wastewater Master Plan boundary.
b) **Scenario 2030:** Using the approved sector plans and other assumptions about future growth, disburse the projected 2030 population within the Wastewater Master Plan boundary.
c) **Scenario Build-out:** Using the approved sector plans and other assumptions about future growth, populate the entire Wastewater Master Plan boundary.

Planning Process

It is appropriate for this report and attached maps to be provided to the Planning Commission, County Commission, School Districts, and other stakeholders for review and comment. Input from the stakeholders and general public will be solicited through the meetings staff will hold with the City Commission, Planning Commission, County Commission, and School Districts. Staff can complete the majority of the information sharing in May, 2010. Results of the input and comments will be summarized and a report of the results will be presented for City Commission approval at the end of the information-sharing process in order to provide staff the direction to implement the planning process.

Action Requested:

Receive report and direct staff as appropriate.



City of Lawrence UTILITIES

2010 Wastewater Master Plan
Study Boundary & Basins

DISCLAIMER NOTICE

The data is provided "as is" without warranty or any representation of accuracy, timeliness or completeness. The burden for determining accuracy, completeness, timeliness, merchantability and fitness for or the appropriateness for use rests solely on the requester. The City of Lawrence makes no warranties, express or implied, as to the use of the Data. There are no implied warranties of merchantability or fitness for a particular purpose. The requester acknowledges and accepts the limitations of the Data, including the fact that the Data is dynamic and is in a constant state of maintenance, correction and update.

WWMP_Basins&Boundary.mxd - Cjm 3/29/10

Residential Development Project Sites, January 2010

Memorandum
City of Lawrence
City Manager's Office

TO: David L. Corliss, City Manager
CC: Diane Stoddard, Assistant City Manager
FROM: Roger Zalneraitis, Economic Development Coordinator/Planner
DATE: January 27, 2010
RE: Update to Residential Inventory Analysis

This memo provides an update to the available residential lot inventory conducted on January 30th, 2009. The update finds that based on current market conditions, there is sufficient inventory to meet 8 to 14 years of demand for new single family residential housing. This represents an increase from last year and is almost exclusively caused by deteriorating housing market conditions.

Previous Report

The residential lot inventory of January, 2009 found that there were about 4,400 lots platted from 1997 to the end of 2008. Of those, approximately 1,000 lots remained available for construction, and a little over 1,400 lots remained available for construction throughout the City. In 2008, there were 141 single family residential permits issued for new construction. As a result, the available lots represented up to 11 years of available inventory for the community.

Inventory Update

From 1999 to the end of 2009, there were approximately 4,087 residential lots platted in Lawrence. The change from the previous analysis suggests that about 300 lots were platted in 1997 and 1998. Perhaps as a result of the recession, there were very few new plats filed in 2009. The majority of new plats were replats of existing subdivisions.

Of the 4,087 lots available at the end of 2009, about 761 of them remained available for development:

Table 1

Residential Inventory as of December 31st, 2009

Lots Platted After January 1, 1999

	Lots	Area (Acres)	Average Lot Size
No Infrastructure, No Dwelling Units	547	137.6	0.25
Infrastructure, No Dwelling Units	761	223.3	0.29
Infrastructure and Dwelling Units	2,779	787.7	0.28
Total Lots	4,087	1,148.7	0.28

Note: The increase in area from 2008 resulted from lots that previously had no acreage recorded within the GIS database.

Additionally, almost 550 lots platted in the last 10 years still have no sewer or water (infrastructure). It is unclear at this time when these lots may receive infrastructure, as the recession has slowed demand for additional housing. The 761 lots with infrastructure represent almost 20% of the total stock of newly platted lots.

Across the City as a whole, there are 1,335 available lots for development (this includes the 761 lots in recently platted subdivisions). This represents a decrease of about 90 available lots since last year. The decrease in available lots resulted because of new building permits and limited new plats over the course of the year.

New Residential Construction
About 141 residential building permits were issued last year:

Table 2
New Residential Permits in 2009

Type	Total	Platted, 1999-2009	Units
Single Family	110	91	110
Duplex	16	13	32
Apartment	15	15	172
Total	141	119	314

source: GIS and Development Services Permit Report

There were 110 single family residential permits issued (compared to 141 single family residential permits in 2008), 16 duplex permits issued (representing 32 units of new construction), and 15 apartment permits issued. However, all 15 apartment building permits were issued to the same site and represent 172 units in 15 new buildings at the one site. In total, 314 new units of residential housing were built. The vast majority of these new units were built on lots that were platted between 1999 and 2009. Three duplex permits and 19 single family housing permits were issued on lots that were platted prior to 1999. Therefore, it is reasonable to expect that new residential building permits will be issued on recently or soon-to-be platted properties in the City.

A map of the residential lots platted from 1999 to 2009 and the building permits that were issued in 2009 is available in the Appendix to this report.

Residential Building Lot Inventory
The residential lots listed in Table 1 are predominantly single family residential units. Therefore, this analysis will use them as a proxy for all available single family residential units in the City.

With approximately 761 single family residential lots available in areas platted from 1999 to 2009, in current market conditions this represents over 8 years of market demand. In other words, the market demand of 91 new single family residential units in newly-constructed subdivisions could be maintained for that time period. However, we also saw in Table 1 that almost 2,800 lots platted since 1999 have housing on them. The historic rate of demand is thus about 252 units per year. At that rate of development, the 761 single family lots would accommodate about 3 years of growth.

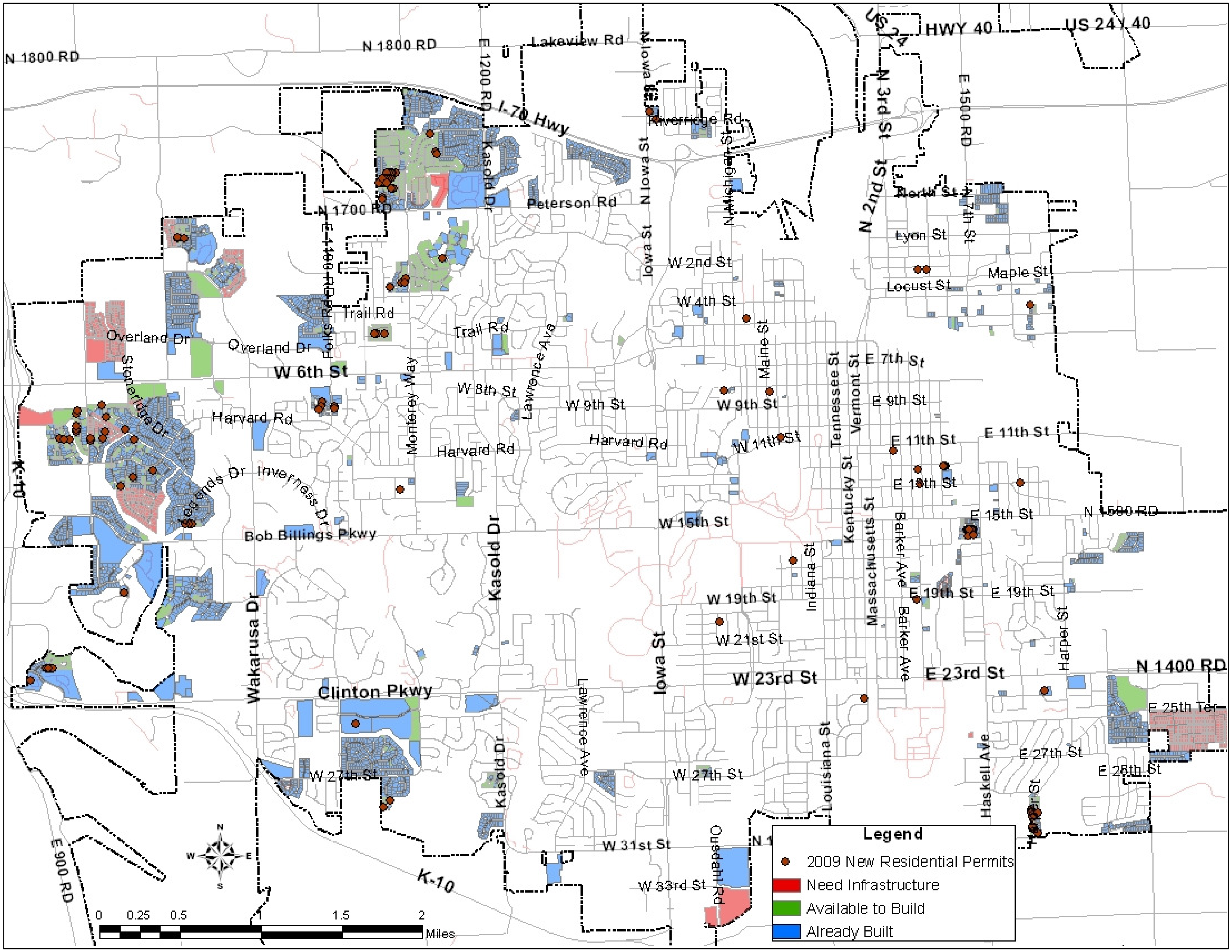
There are a total of 1,335 available residential lots in the City. This implies that there are 574 additional single family residential lots available in older subdivisions. As noted, 18 permits were issued in these subdivisions last year for single family residences. Therefore, under current market conditions these 574 lots could accommodate far more than 20 years of growth.

Finally, there are 547 residential units that are platted but have no infrastructure. If these are added to the 761 available single family residential units, the inventory rises from being able to accommodate 8 years of current demand to being able to accommodate over 14 years of current demand. Under historic demand scenarios, there would be a little more than 5 years of inventory available for single residential family housing.

At the end of 2008, we estimated that existing and potential inventory (lots that do not yet have infrastructure) could accommodate between 5 and 11 years worth of demand. There now appears to be between 5 and 14 years of demand. Additionally, there has been a slight decline in available lot inventory over this period. The fact that demand appears to be slightly greater now is a reflection of deteriorated housing market conditions rather than new supply coming online.

APPENDIX

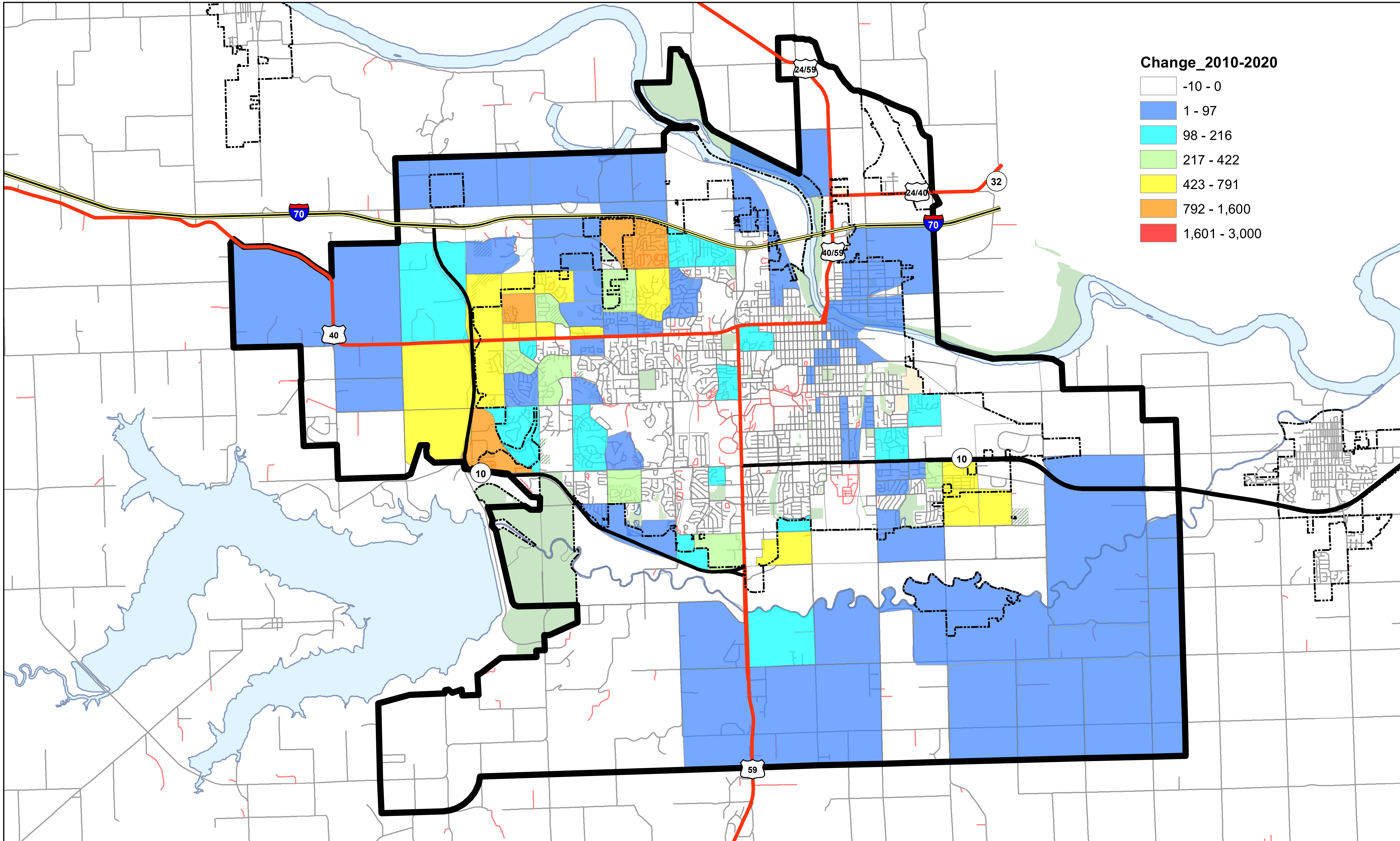
Lawrence Residential Lots Platted Between 1999 and 2009, and Residential Building Permits Issued 2009



Estimated Population Change Between 2010 and 2020 by TAZ for City Wastewater Master Plan

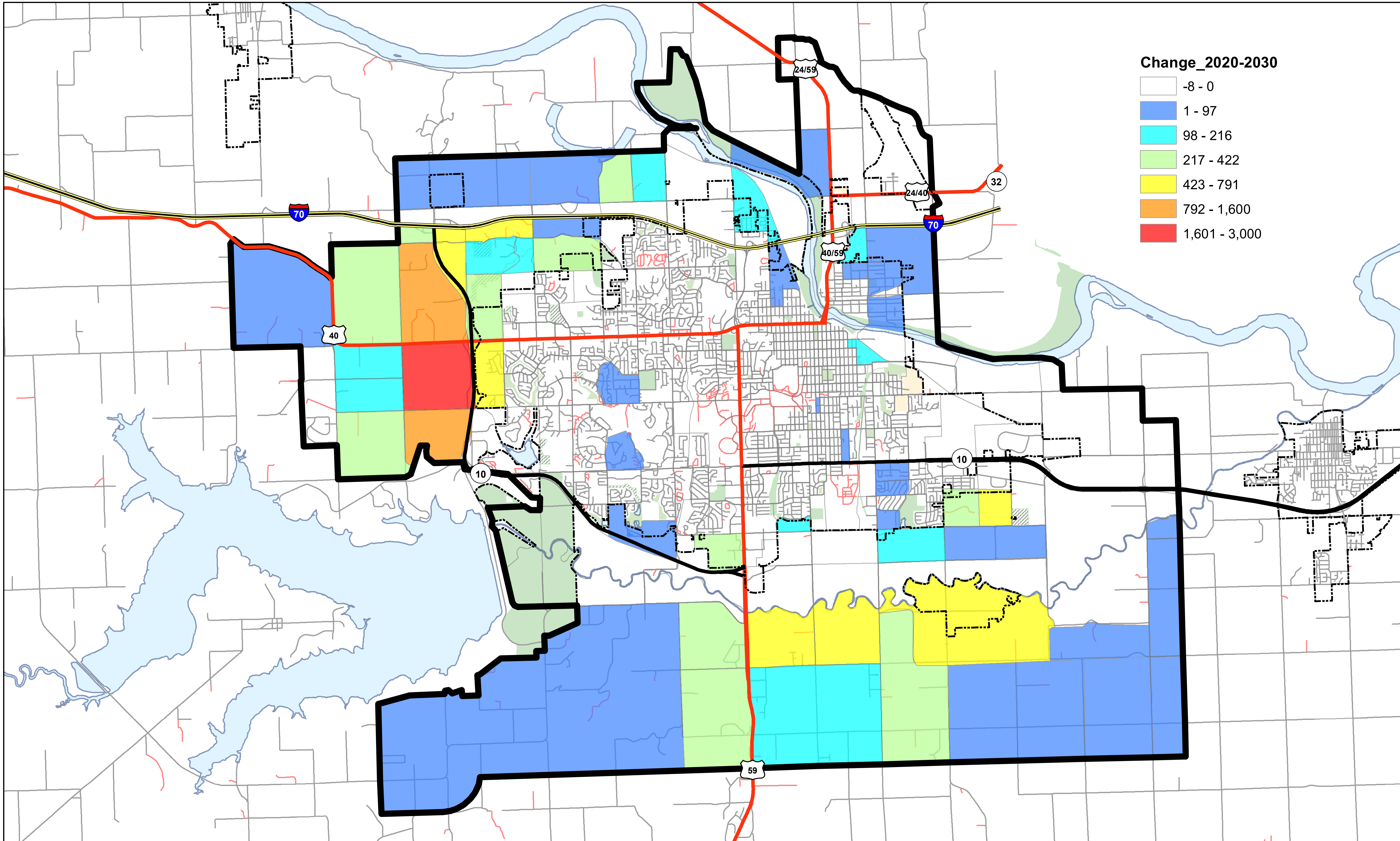
0 1,750 3,500 7,000 10,500 14,000 Feet

1 inch = 7,000 feet

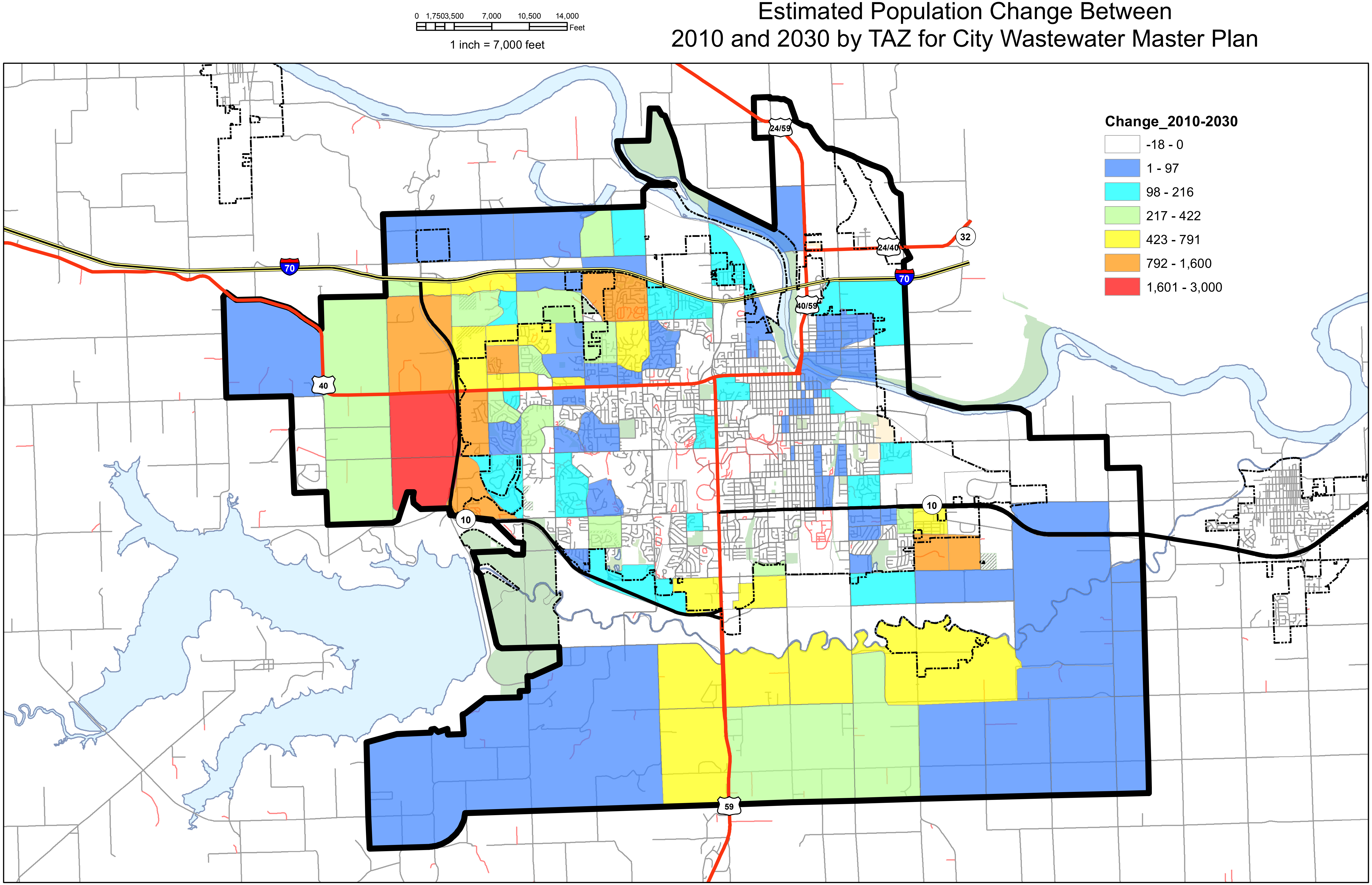


0 1,7503,500 7,000 10,500 14,000
Feet
1 inch = 7,000 feet

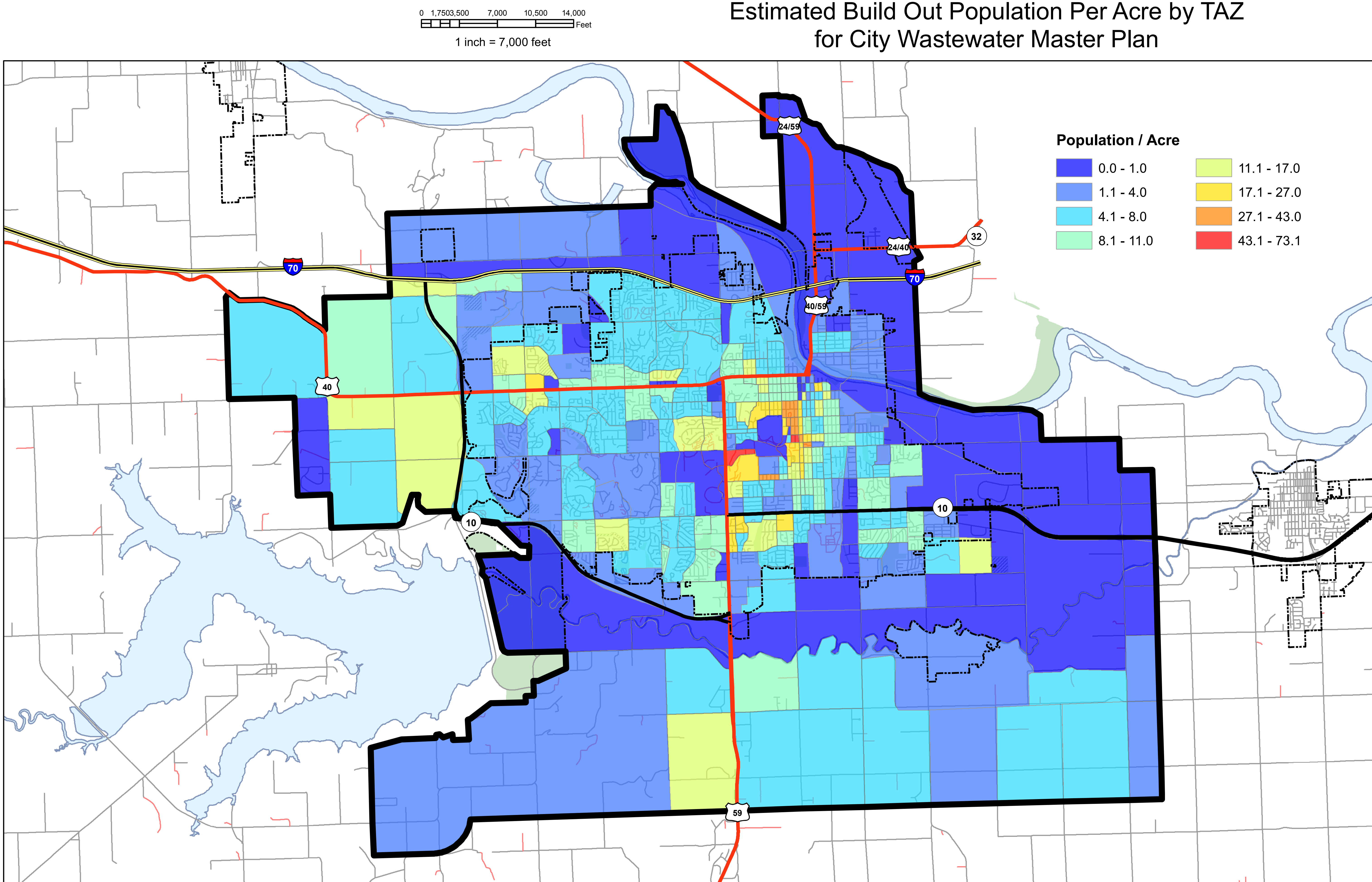
Estimated Population Change Between 2020 and 2030 by TAZ for City Wastewater Master Plan



Estimated Population Change Between 2010 and 2030 by TAZ for City Wastewater Master Plan



Estimated Build Out Population Per Acre by TAZ for City Wastewater Master Plan



Appendix 1-E
University of Kansas Water Use Projections

THE UNIVERSITY OF KANSAS
Facilities Operations

To: Doug Riat

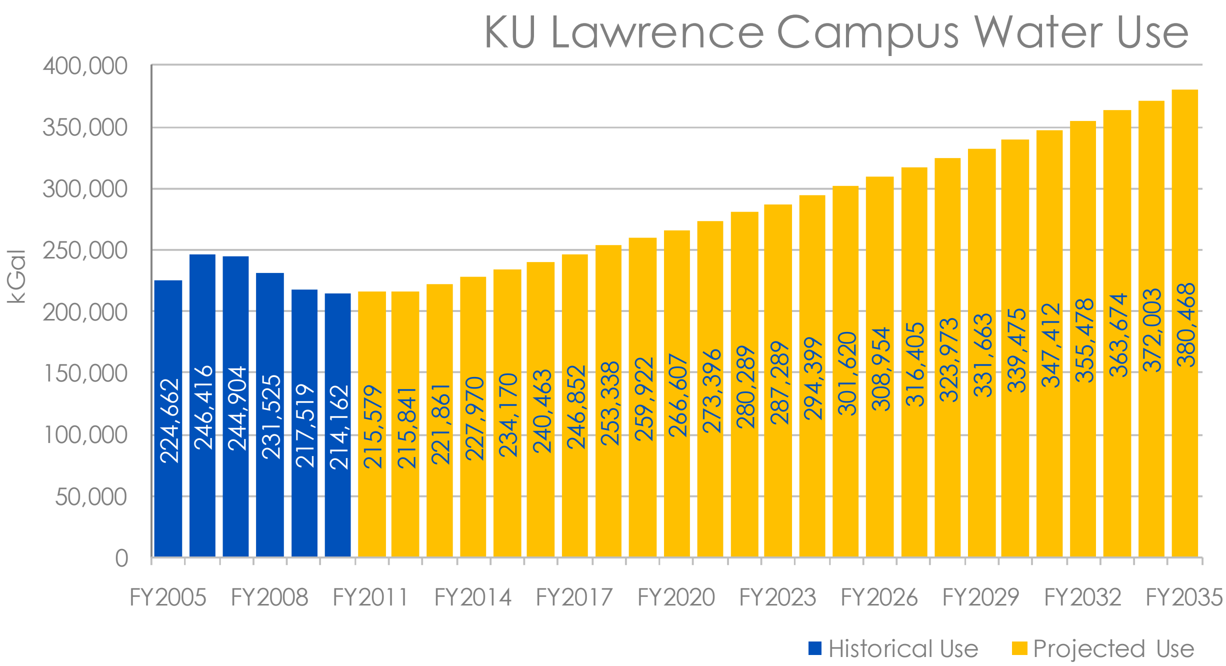
From: Scott McVey

Date: July 27th, 2010

Re: 25 year water use projections for KU's Main and West Campus

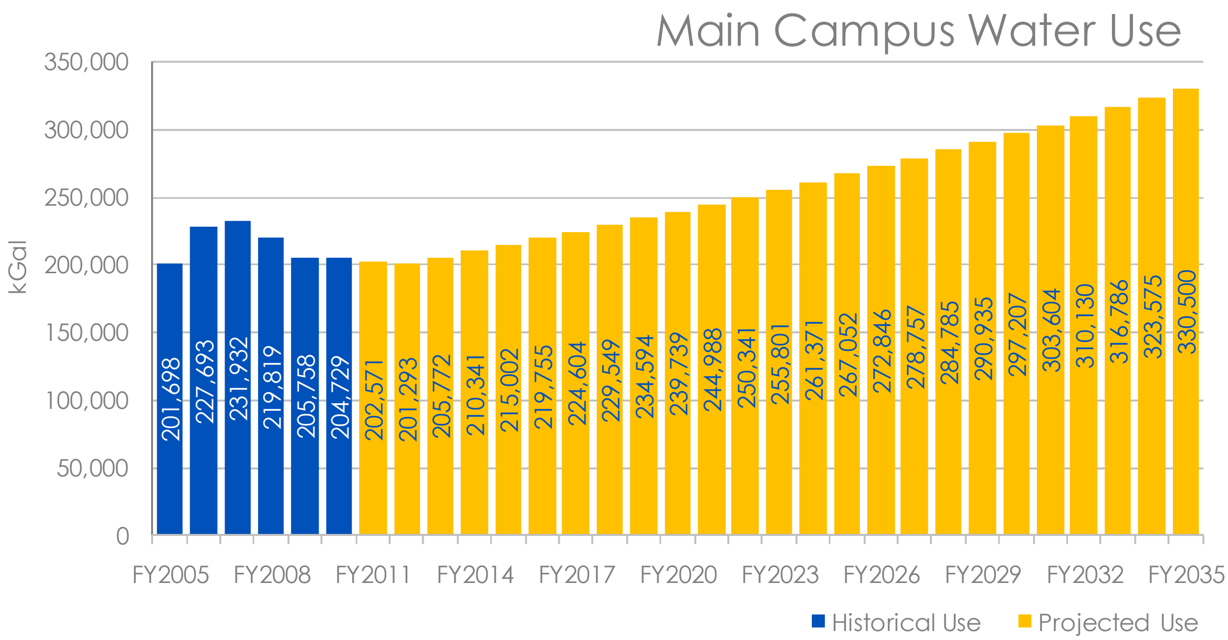
---

This memo summarizes the study of KU's projected water use over the next 25 years. The combined Main and West Campus water usage is expected to increase by 77 percent in 25 years, or 166 million gallons as shown in the following chart:



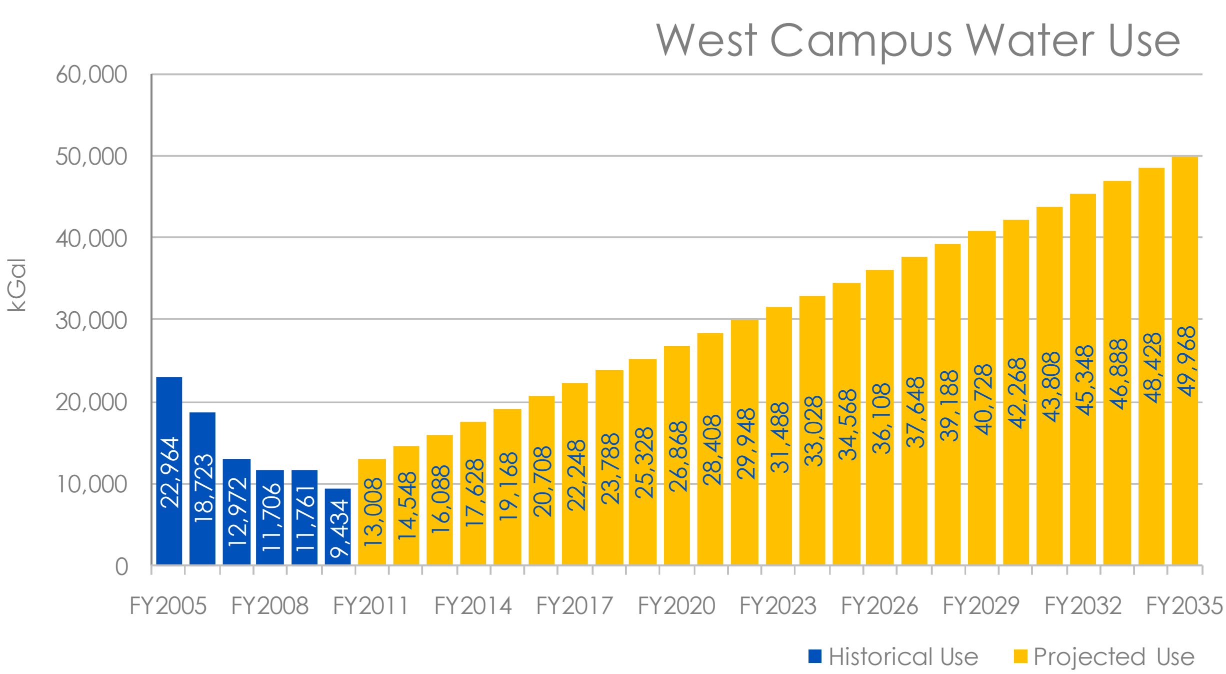
Main Campus water use projections

Modest growth in student population on Main campus is expected. It is assumed that water use will increase proportional to student population growth on Main Campus at a rate of 2 percent per year. The following chart shows KU's Main Campus water use for the past five years and projections for annual water use over the next 25 years. Additionally, it is anticipated that a recent water conservation project on campus will result in a short-term decline in water use. In 25 years it is projected that water use on Main Campus will be approximately 330,500,000 gallons, a 61 percent increase over FY2010's usage.



West Campus water use projections

KU's West Campus has the largest potential for water consumption increases due to the large amount of undeveloped land that will facilitate new buildings. It is projected that KU will expand new building area by approximately 50,000 square foot per year over the next 25 years. On average, west campus research buildings use approximately 31 gallons of water per square foot annually. As a result of this growth it is projected that water use on West Campus will be approximately 50,000,000 gallons, a 400 percent increase over current usage by year 2035.



Lawrence Wastewater Master Plan
Meeting Notes

Date: October 6, 2010
Location: BG Consultants office
Attendees: Mike Lawless, Jim Modig, Doug Riat, Leigh Myers , David Hamby

Topics Discussed:

At the beginning of the meeting there was a short introduction of the master plan process and using population projections to determine flow. The City appreciates the input from KU and for meeting with them to help understand the water use projections and what they represent.

Doug explained the thoughts behind the July 27, 2010 memo that was prepared by Scott McVey and sent to the City of Lawrence staff showing the 25 year water use projections.

Doug shared that KU did not expect a significant enrollment growth either long term or short term.

Doug said that the projected use numbers presented in the memo for Main Campus may be an overestimation. The 2% growth rate used is likely high. A 1% growth rate is more likely.

Doug explained that new building growth on the Main Campus will be limited as there are only a few locations for new buildings. The new buildings will be more efficient than most of the existing buildings.

Jim and Doug stated that they felt comfortable with the West Campus water use projections. They have had extensive growth in the last few years and expect that growth to continue for the foreseeable future. They said that the growth will mainly be in the area to the north of the Shenk Fields. A building could be built near the Lied Center but no building is planned for the Fields Area or west of the existing creek.

The amount of water associated with irrigation on the West Campus was discussed. If KU Staff could isolate this amount they would provide it to the City.

It was decided that the water use information is not appropriate for the master plan build out scenario but is really an addition to the 2020 and 2030 population information.

Technical Memorandum No. 2
Existing Wastewater Collection System Evaluation

Wastewater Facilities Master Plan

for

Lawrence, Kansas

City of Lawrence, Kansas

BMcD Project No. 54793

City P.O. 07629

Burns & McDonnell Engineering Company, Inc.
9400 Ward Parkway
Kansas City, MO 64114

City of Lawrence, Kansas

Wastewater Facilities Master Plan Technical Memorandum No. 2

Table of Contents

List of Tables

List of Figures

* = follows page number

Appendices

A. Introduction

Technical Memorandum (TM) No. 2 is a summary of an evaluation of the existing wastewater collection system completed in partial fulfillment of the Lawrence, Kansas Wastewater Facilities Master Plan. The goals for this TM were to:

- Document the components of the existing system.
- Delineate system drainage basins that are useful for system analysis.
- Document the wastewater flow and rainfall monitoring program performed by the City of Lawrence in support of this master plan.
- Analyze existing wastewater flow components including both dry and wet weather flow components such as wastewater flow, dry weather infiltration, and wet weather derived infiltration and inflow by drainage areas tributary to the wastewater flow meters installed for the City's wastewater flow metering program.
- Compare estimated levels of rainfall derived infiltration and inflow (RDII) within drainage areas and rank them on the basis of RDII level.
- Develop a computer hydraulic model of the existing collection system calibrated for both dry and wet weather flow conditions on the basis of the wastewater flow and rainfall monitoring program performed by the City.
- Run the computer hydraulic model to simulate what flows would occur during a design storm wet weather event.
- Based on the existing system modeled design storm event, identify any system deficiencies which require corrective action to reduce or eliminate all sanitary sewer overflows.
- Determine recommended corrective measures required to address existing system deficiencies. In some cases, alternatives may be compared to arrive at the best solution for the City's needs.

B. Existing System Description

1. Drainage Basins

For purposes of system evaluation, the master plan study area is divided into a total of eight major drainage basins which are further subdivided into sub-basins. The drainage basins and sub-basins generally follow natural watershed and sub-watershed boundaries, which also correspond to the configuration of gravity sewers, pumping stations and force mains. Drainage basins and sub-basins are shown on Figure 2.1 and are identified below in Table 2.1.

Table 2.1
Drainage Basins

Drainage Basin Name	Drainage Basin Designation	Number of Sub-Basins
Baldwin Creek	BC	7
Central	C	4
East Lawrence	EL	4
Kansas River	KR	12
North Lawrence	NL	4
Wakarusa River	WR	8
Wakarusa River South	WRS	11
Yankee Tank Creek	YTC	6

Parts or all of seven of the drainage basins are currently served by the City's wastewater utility, while one, the Wakarusa River South drainage basin, has no service at this time.

2. Drainage Basin Descriptions

a. Baldwin Creek Basin

The Baldwin Creek Basin lies within northwest Lawrence and includes seven sub-basins. The drainage basin straddles Interstate 70, is bounded by W 6th Street and US 40 on the south, E 700 on the west, and E 1100 Road to the east. Natural drainage is generally to the north and east toward the Kansas River. Due to limited development in this drainage basin, pumping stations convey flows east to gravity sewers in the Kansas River basin and south to gravity sewers in the Yankee Tank Creek and Wakarusa River Basins.

b. Central Basin

The Central Basin includes four sub-basin and contains most of the main University of Kansas campus. The Alabama Street Pumping Station (PS-8) receives the majority of flow from this basin for transfer to the Kansas River Basin. Wet weather peak flows that may exceed PS-8 capacity are diverted to the Wakarusa River Basin via an 15-inch gravity relief sewer.

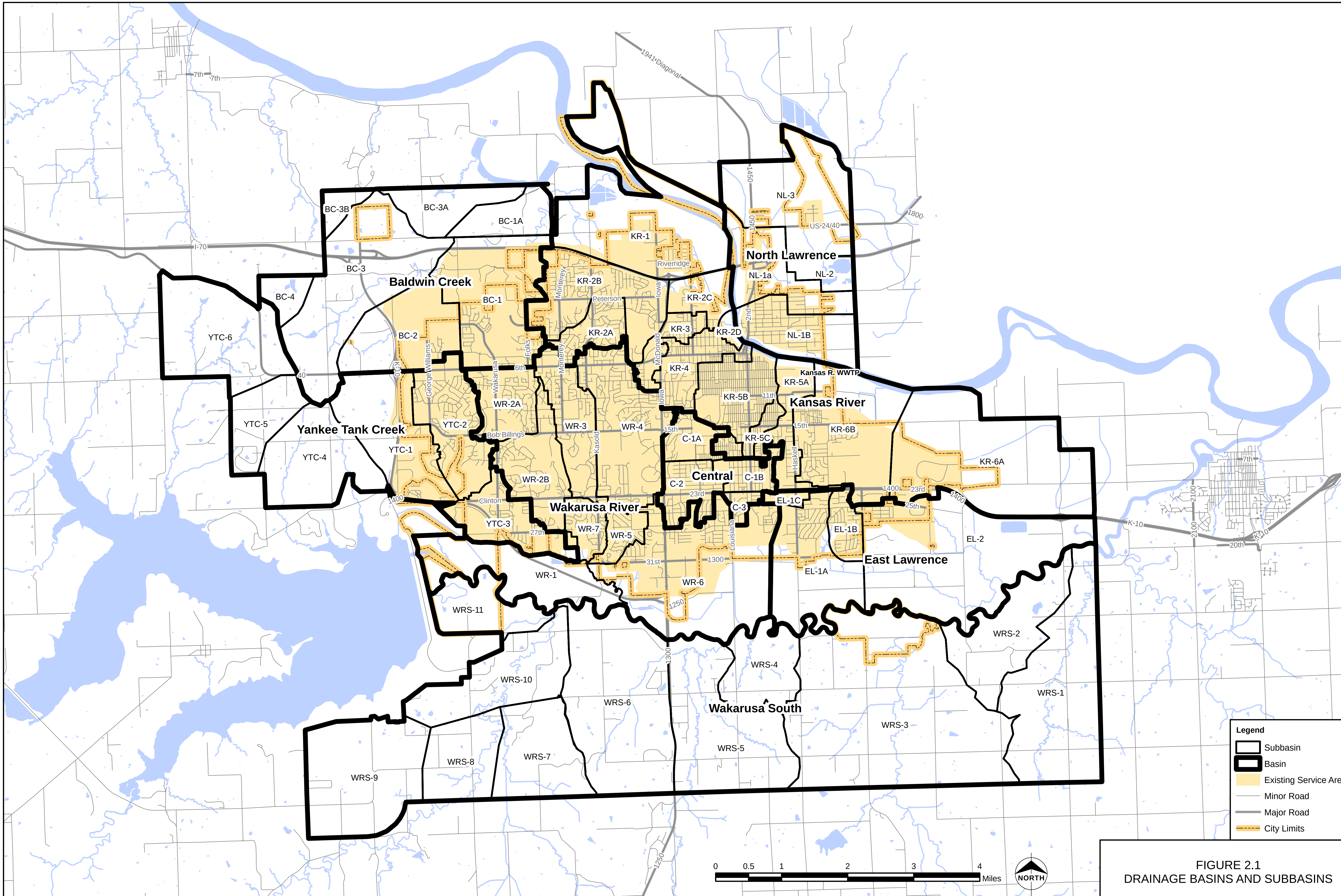


FIGURE 2.1
DRAINAGE BASINS AND SUBBASINS

c. East Lawrence Basin

The East Lawrence Basin is generally bounded by K-10 to the north, the Wakarusa River to the south, Haskell Avenue to the west and E 1900 Road to the east. It includes four sub-basins. Pumping stations transfer basin flows west and north to the Kansas River Basin.

d. Kansas River Basin

The Kansas River Basin includes downtown Lawrence and adjacent areas to the southeast and northwest of downtown. There are 12 sub-basins. All other drainage basins convey their flows to the Kansas River Basin since the KRWWTP is located in this basin. It includes the oldest parts of the City's collection system. Because of these factors, it is a critical part of the City's wastewater collection system. The majority of flows are conveyed to the KRWWTP influent pumping station via the Kentucky Street Pumping Station (PS-16) and gravity interceptors.

e. North Lawrence Basin

The North Lawrence Basin includes the only sewered areas north of the Kansas River and has four sub-basins. Collected flows are pumped south across the Kansas River to the KRWWTP.

f. Wakarusa River Basin

The Wakarusa River Basin includes eight sub-basins. It includes south Lawrence and a portion of west Lawrence to approximately Wakarusa Drive. It lies generally south of 6th Street and west of Iowa Street. Its southern boundary is the Wakarusa River. Major pumping stations in this drainage basin include the Four Seasons Pumping Station (PS-9) and its 6.25 MG total volume wet weather peak flow storage basins, and the Wakarusa Pumping Stations (PS-5A and PS-5B), which convey basin flows north to the Kansas River Basin.

g. Wakarusa River South Basin

The Wakarusa River South Basin includes eleven sub-basins. It lies generally south of the Wakarusa River and bounded to the west, south and east by the master planning area boundary. The Wakarusa River South Basin has no wastewater utility service at this time.

h. Yankee Tank Creek Basin

The Yankee Tank Creek basin lies west of the Wakarusa River Basin and south and west of the Baldwin Creek Basin, extending as far west as East 550 Road. It includes six sub-basins (YTC-1 through YTC-6). Clinton Lake is directly south of the Yankee Tank Creek Basin. Currently, only three sub-basins, YTC-1, YTC-2 and YTC-3 are served by the City's wastewater utility. Flows are conveyed by gravity to the Four Seasons Pumping Station where they are pumped east to the Wakarusa River Basin.

3. Collection System and Treatment Facilities

a. General

The existing collection system and treatment facilities include a network of gravity sewers, pumping stations and force mains, peak flow storage basins, and a wastewater treatment plant. A map of the existing collection system and treatment facilities is shown in Figure 2.2.

b. Gravity Sewers

Gravity sewer sizes range from 6-inch to 48-inch diameters. The oldest sewers, dating as early as 1916, are vitrified clay pipe. More recently, reinforced concrete, composite, and plastic (PVC) have been used. Older manholes are constructed of brick and mortar, while newer manholes are precast concrete.
Table 2.2 summarizes existing sewers and other data for each of the drainage basins.

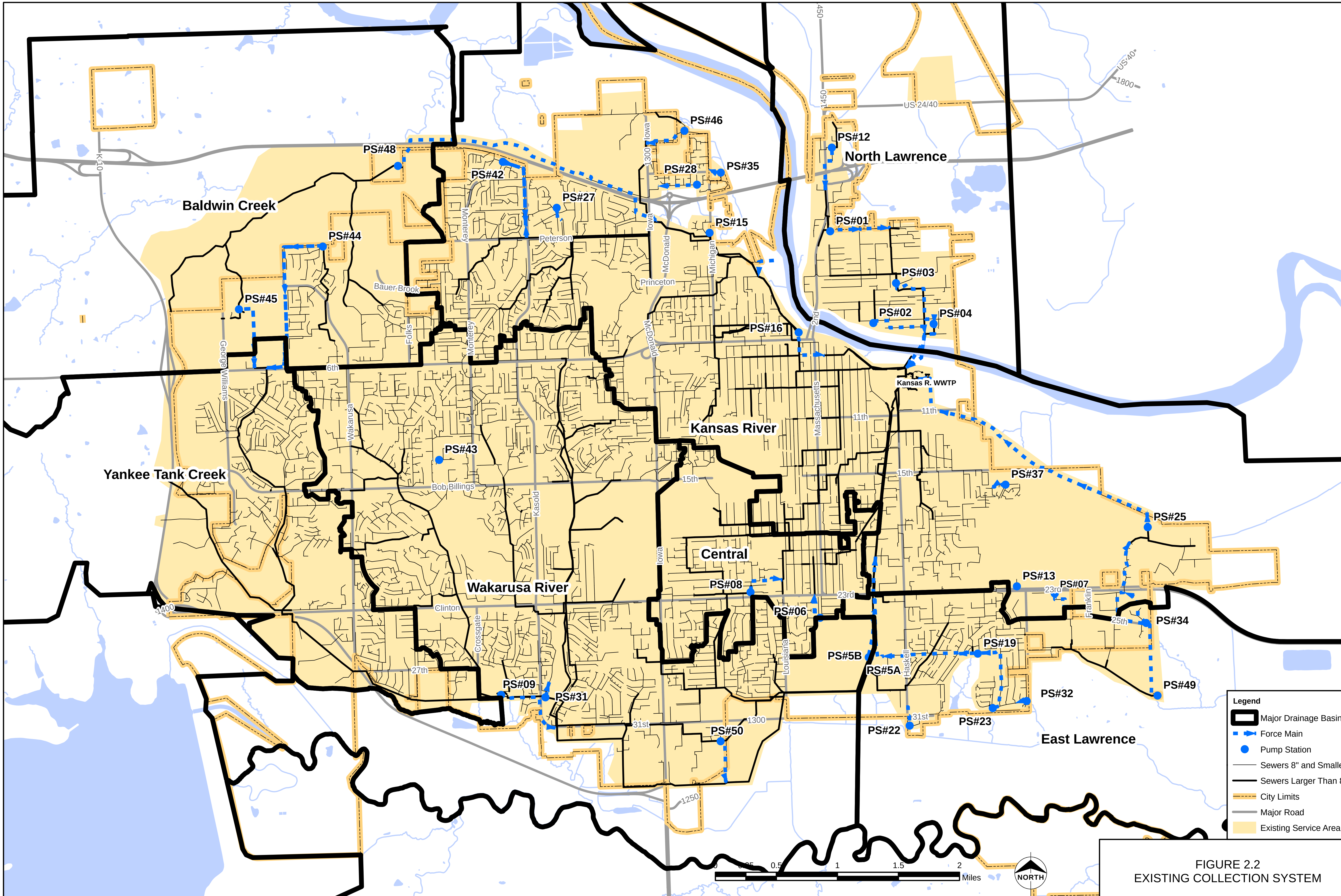


FIGURE 2.2
EXISTING COLLECTION SYSTEM

Table 2.2
Existing Sewers

Gravity Sewers					Manholes		
Basin	Average Age (Years)	Length (Miles)	Material (>10%)		Number	Material (>10%)	
Baldwin Creek	9.5	16.8	PVC	37.8%	407	Conc	100%
			Truss	44.3%			
			VCP	11.3%			
Central	52.3	31.2	VCP	88.9%	786	Brick	61%
						Conc	37%
East Lawrence	23.2	21.9	PVC	20.9%	570	Brick	7%
			Truss	46.8%		Conc	93%
			VCP	31.2%			
Kansas River	37.4	139.9	PVC	10.8%	3366	Brick	39%
			VCP	71.6%		Conc	59%
North Lawrence	47.9	16.0	Truss	10.9%	354	Brick	43%
			VCP	80.7%		Conc	54%
Wakarusa River	32.1	135.8	PVC	15.5%	3611	Brick	14%
			Truss	42.1%		Conc	85%
			VCP	41.0%			
Yankee Tank Creek	12.6	44.3	PVC	30.8%	1084	Conc	100%
			Truss	60.3%			

c. Pumping Stations and Force Mains

The collection system includes 33 wastewater pumping stations as needed to convey wastewater from one drainage basin to another or at locations where it is not practical to extend gravity sewers. Table 2.3 summarizes pumping station and force main information by drainage sub-basin.

Table 2.3
Pumping Station and Force Main Summary

Sub-basin	Pump Station No.	Total Peak Capacity-MGD (1)	Firm Peak Capacity-MGD (2)	Force Main No. 1		Force Main No. 2	
				Size (in)	Length (ft)	Size (in)	Length (ft)
North Lawrence Basin							
NL-1	PS_01	1.97	1.86	10/8	2,648	NA	NA
NL-1	PS_02	0.62	0.54	6	2,933	NA	NA
NL-1	PS_03	3.24	2.90	12/6	3,492	NA	NA
NL-1	PS_04	4.82	3.56	18	2,712	8	2,595
NL-1	PS_12	0.46	0.39	6	2,175	NA	NA
Wakarusa River Basin							
WR-6	PS_5a	3.05	2.77	12	4,519	NA	NA
WR-6	PS_5b	16.59	12.72	24	2,088	NA	NA
WR-2	PS_09	7.12	3.91	20	3,499	NA	NA
WR-2	PS_09 WW	4.74	0.00	20	412	NA	NA
WR-2	PS_31	0.17	0.16	4	686	NA	NA
WR-3	PS_43	0.13	0.08	2	267	NA	NA
WR-6	PS_50	0.80	0.61	6	1,995	NA	NA
Central Basin							
C-3	PS_06	2.29	1.80	8	1,299	8	1,243
C-2	PS_08	3.04	2.87	10	1,999	NA	NA
Baldwin Creek Basin							
BC-1	PS_44	3.72	1.86	10	7,716	10	7,716
BC-2	PS_45	1.60	0.79	8	3,192	8	3,192
BC-1	PS_48	6.56	6.03	16/24	13,665	NA	NA
Kansas River Basin							
KR-6	PS_07	0.34	0.30	4	1,359	NA	NA
KR-6	PS_13	0.16	0.15	4	349	NA	NA
KR-2	PS_15	0.34	0.29	4	342	NA	NA
KR-4	PS_16	19.52	17.41	24	2,124	NA	NA
KR-6	PS_25	4.40	3.63	12	13,743	8/10	13,114
KR-2	PS_27	0.74	0.53	6	651	NA	NA
KR-1	PS_28	0.13	0.12	4	1,681	NA	NA
KR-1	PS_35	0.13	0.11	4	653	NA	NA
KR-6	PS_37	0.17	0.11	4	914	NA	NA
KR-2	PS_42	1.26	0.55	8	3,950	6	3,950
KR-1	PS_46	1.57	1.23	8	1,937	NA	NA
East Lawrence Basin							
EL-1	PS_19	2.60	1.93	12	7,319	NA	NA
EL-1	PS_22	0.13	0.12	4	1,139	NA	NA
EL-1	PS_23	0.08	0.05	4	508	NA	NA
EL-1	PS_32	0.79	0.69	6	3,576	NA	NA
EL-2	PS_34	0.13	0.09	4	1,860	NA	NA
EL-2	PS_49	2.63	1.78	12	8,439	NA	NA

(1) Capacity based on all pumps in service using all force main(s).

(2) Capacity based on largest pump out of service using all force main(s).

d. Wet Weather Storage Basins

Three wet weather storage basins having a total storage volume of 6.25 MG are located adjacent to the Four Seasons Pumping Station (PS-9). PS-9 includes two pumps used for pumping peak wet weather flows that exceed the capacity of the dry weather pumps to the storage basins. Stored flows are returned to the pumping station and pumped with other dry weather flows to the wastewater treatment plant after flows return to normal rates.

e. Wastewater Treatment Plant

All wastewater flows are now conveyed to the City's Kansas River Wastewater Treatment Plant (KRWWTP) for treatment. Treated wastewater is discharged to the Kansas River. The KRWWTP has a permitted annual average flow rate of 12.5 MGD. The treatment plant provides secondary biological treatment and disinfection of flows up to a peak flow rate of 25 MGD. During wet weather, flows rates that exceed 25 MGD are pumped to a high rate wet weather treatment facility or an on-site peak flow storage basin. These facilities have a firm peak flow rate capacity of 40 MGD, providing a total firm peak flow rate capacity of 65 MGD.

Wastewater flows will exceed the capacities of the KRWWTP at some time during the master planning period. The City has purchased land south of the Wakarusa River, which is to be the site of a new wastewater treatment plant.

C. Flow Monitoring Program

The City of Lawrence has conducted a flow monitoring program for about 5 years, beginning in September of 2006. There are 33 flow meters and 8 rain gauges used in the program. The locations of the flow meters are shown on Figure 2.3.

The City has leased the flow metering and rainfall monitoring equipment from Marsh McBirney. The start and stop dates for some flow meters do not span the entire length of the 5-year flow monitoring period. Three flow meters were moved part way through the monitoring period, others started after the beginning of the monitoring period, and most meters experienced periods when the flow meter was active, but did not record data at some time during the program. An attempt was made to utilize all available data, so there were 36 meter locations examined. A chart showing active dates by flow meter is shown in Table 2.4.

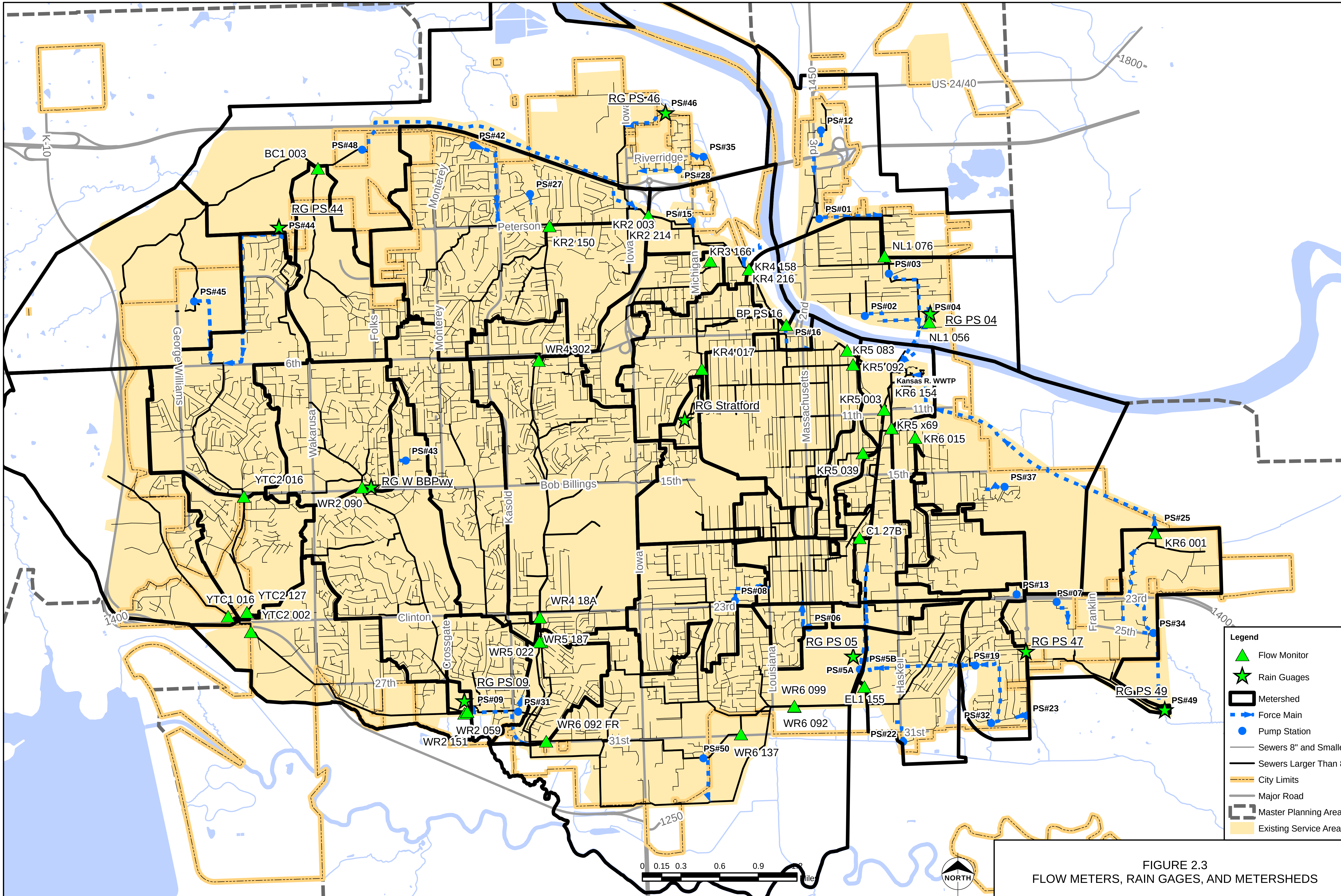


FIGURE 2.3
FLOW METERS, RAIN GAGES, AND METERSHEDS

Table 2.4
Flow Meter Coverage

Transition date (no overlap is assumed)

9/22/2006		9/26/2007	4/22/2008	8/8/2008	6/17/2009	PRESENT
		BC1 003	9/26/07 TO PRESENT			
			C1 27B	4/22/08 TO PRESENT		
EL1 155	9/22/06 TO PRESENT					
KR2 003	9/22/06 TO PRESENT					
				KR2 150	8/10//2008 TO PRESENT	
KR2 214	9/25/06 TO PRESENT					
				KR3 166	8/05/08 TO PRESENT	
KR4 017	9/26 TO 8/8/08					
KR4 158	9/22/06 TO PRESENT					
				KR4 171	8/12/08 TO PRESENT	
KR4 216	9/21/06 TO PRESENT					
KR5 003	ALL					
					KR5 039	6/17/09 TO PRESENT
KR5 083	9/26/06 TO PRESENT					
KR5 092	ALL					
KR5 x69	9/20/06 TO 4/22/08					
KR6 001	ALL					
KR6 015	ALL	big gap from 6/23/08 to 11/4/08				
KR6 154	9/22/06 TO PRESENT					
NL1 056	ALL					
NL1 076	ALL					
WR2 059	ALL					
WR2 090	ALL	GAPS				
WR2 151	ALL					
			WR4 18A	4/22/08 TO PRESENT		
WR4 302	9/26/06 TO 4/22/08					
WR5 022	9/26/06 TO 4/22/08					
WR5 187	ALL	gaps: 5/18/07 + , 10/4/2008				
WR6 092	9/22/06 to PRESENT					
WR6 092 FR	9/26/06 TO PRESENT					
WR6 099	9/20/06 TO PRESENT					
			WR6 137	4/22/08 TO PRESENT		
YTC1 016	ALL	BIG FLOW at 7/6/09				
YTC2 016	9/25/06 TO 8/8/08					
YTC2 002	9/26/06 TO 8/8/08					
				YTC2 127	8/10/08 to PRESENT	

The sanitary sewer network was partitioned for analysis into networks tributary to flow meters. Each of the tributary networks, or "metersheds", was given a shortened name derived from the flow meter. For example, the pipe network tributary to the meter KR2_214_HALLMRK was named the KR2 214 metershed (as seen in Table 2.4). The metersheds used for the analysis in this report were created by following the flow path through the sewer network according to pipe invert elevations. The metersheds were compared and found to be different from the study areas in the 2003 Master Plan Report and different from the areas created by the City in subsequent analysis. Further comparative analysis between the three study areas was not done due to these differences.

In several instances, there are adjacent metersheds interconnected upstream of their flow meters by what are termed as "crossover" gravity sewers by this TM. A typical situation is two gravity sewers exiting from a single manhole, with one of the sewers conveying flow further downstream in the metershed, and the second conveying flow to an adjacent metershed. Often, the two exiting gravity sewers are installed at the same invert elevation.

There are a total of 31 such crossover gravity sewers that were referred to the City during TM2 development for investigation and verification of their conditions before the flow analysis proceeded further. Crossover sewers are installed to maximize system capacity. This number of crossover sewers, however, is unusual for a collection system of this size and makes it much more problematic to analyze the system without adding significantly more flow metering locations.

A schematic of these crossover sewers is provided in Figure 2.4, showing their relationships with the metersheds and flow meters. The crossover sewers are shown with their pipe size, slope, and whether each pipe's invert elevation is the same or higher than other pipes leaving the manhole at the upstream end of the crossover sewer.

Where these crossover conditions occur, it is likely that some portion of the upstream flow from one metershed is being diverted to the adjacent metershed. The actual diversion of flows by these crossover sewers is unknown and likely varies with different wet weather events. This raises uncertainties about the analysis of flows. As an example, there are cases where a downstream flow meter in a metershed records less flow than an upstream flow meter in the same metershed. In other cases, the dry weather metered flow rate is outside of reasonable expectations based on the population and land use that exists in the metershed. Both of these circumstances can be explained by one or more crossover sewers located in the system between the two meters that is diverting a portion of the flow to an adjacent metershed.

The first analysis step taken was the insertion of the average base flow rate observed at each flow meter into a spreadsheet and subtracting the flow components from the upstream metershed from those of the immediately downstream metershed. This step identified a number of inconsistencies in the flow meter data. The inconsistencies all occur at flow meters, which are, in one way or another, affected by the crossover gravity sewers. This makes it likely they are due to the unknown amounts of flow diversions that occur as a result of the crossover gravity sewers rather than inaccuracies in the flow meter data.

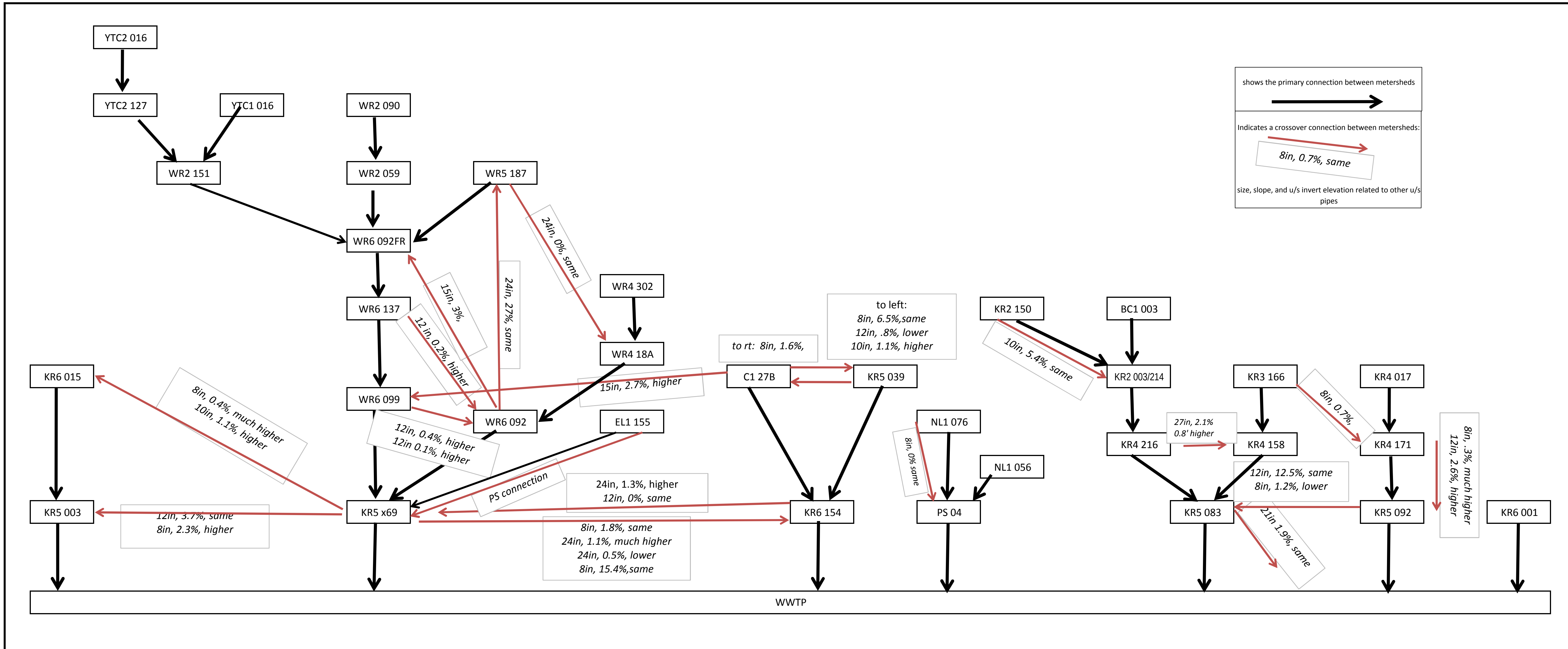


Figure 2.4
Flow Monitoring Diagram
Original Schematic

Each of these cases was analyzed to determine a reasonable approach for the subsequent detailed analysis of flows by metershed. In some cases, it was necessary to combine two or more flow meters into a single metershed or eliminate separate metersheds where their flow meters yielded a negative result due to the subtraction process. Table 2.5 summarizes those flow meters which needed to be eliminated from the flow component analysis and the reason each was eliminated. The net effect of the eliminated flow meters was a reduction in the total number of flow meters that are used for evaluation from 36 to 17, with a corresponding reduction in the number of separate metersheds that may be analyzed for flow components from 36 to 16.

Table 2.5
Flow Meter Drainage Area Elimination and Adjustments

FLOW METER	Action and Rationale
KR2 003	Data combined because of crossover connection immediately upstream of meter.
KR2 214	
YTC2 002	Data not used: this meter is immediately downstream of YTC1 016 and YTC2 127.
WR6 092FR	Data not used: flow is higher than expected according to area and population. Likely affected by upstream crossovers.
WR6 137	Data not used: flow is higher than expected according to area and population. Likely affected by upstream crossovers.
WR6 092	Data not used: negative interior flow.
KR6 x69	Data not used: negative interior flow.
C1 27B	Data not used: uneven distribution of base flow for similar areas, likely affected by upstream crossovers.
KR5 039	
KR4 216	Data not used: negative interior flow.
KR5 083	Data not used: negative interior flow when KR4 216 is accounted for. Likely affected by upstream crossovers.
KR4 171	Data not used: negative interior flow. Likely caused by crossovers both into and out of this metershed.
NL1 076	Data not used: uneven distribution of base flow, likely affected by upstream crossover.
NL1 056	Data combined with PS 03 and PS 04: flow is much smaller than it should be without scaling to PS 04 quantities.
KR5 003	Data not used: very high interior flow, likely affected by upstream crossover.
WR5 022	Data not used: this flow meter was an early term meter, immediately downstream of WR4 18A. Not used because coverage with very similar to u/s meter coverage.
KR2 150	Data not used: there is a crossover conduit adjacent to the flow meter with greater slope that appears to take more flow during low flow conditions than the flow meter conduit.
KR5 092	Data not used: this metershed loses substantial amounts through two crossovers on the north boundary. More leaves the metershed than goes through the meter. There are substantial differences in ratios between high and low flow models.
KR4 158	Data not used: there is very large inflow from a crossover that splits the flow that comes out of KR2 003/214.
WR5 187	Data not used: this metershed loses through one crossover and gains through another, very near the flow monitor. It gains a lot more than it loses. A lot of what comes out of WR4 18A crosses over and goes thru this flow monitor.
WR6 099	Data not used: this metershed has minimal gains and losses from crossovers, and is bracketed with meters at both the upstream and downstream boundaries. However, the difference between the two meters yields daily negative flows.
KR6 154	Data not used: large flows enter this metershed from the south under high flow conditions, and then just above the flow monitor a substantial amount enters from the east.
KR3 166	Data is scaled: at a midway point about 1/3 of the available flow leaves the metershed. This amounts to about 10% of the total metershed flow. This flow meter is scaled to 110%.
WR4 18A	Data is scaled: this shed gains from a crossover with WR5 187 near the meter. The distribution is fairly consistent between high and low flow conditions, a scaling of 75% compensates for the gain.

As a result of this analysis, the schematic of meters and crossover sewers shown in Figure 2.4 is modified to the revised schematic shown in Figure 2.5.

D. Flow Components

U.S. EPA's Sanitary Sewer Overflow Analysis Planning (SSOAP) toolbox program was used to analyze the flow monitoring data and determine various wastewater flow components. The following sections present the analysis for each flow component.

1. Dry Weather Flow

Dry weather flow (DWF) is defined as flow that is not influenced by wet weather conditions. Previous master plans referred to DWF as average daily dry weather flow. It includes wastewater flow (WWF) discharged by utility customers, which is considered to be equal to winter quarter metered water usage. It also includes dry weather infiltration (DWI), which is groundwater that enters the sewer system through system defects such as defective service line connections, broken sewer pipe, and manhole defects. DWI occurs at a nearly constant rate year-round and is not influenced by wet weather conditions.

The SSOAP program uses the flow meter raw data to develop flow component statistics. Because this program is in its first iteration, these statistics were verified for many of the metersheds using spreadsheet analysis. This check was done by choosing dry periods from the rain gauge information and finding the average and maximum flows for each meter. This check found that the SSOAP DWFs were reliable. The SSOAP analysis develops separate weekday and weekend statistics, and these were combined into a single statistic to characterize the base flow for each metershed (2/7*weekend + 5/7* weekday) with an average daily DWF.

Representative examples of the diurnal curves calculated by the SSOAP program during dry weather are shown below in Figures 2.6 and 2.7.

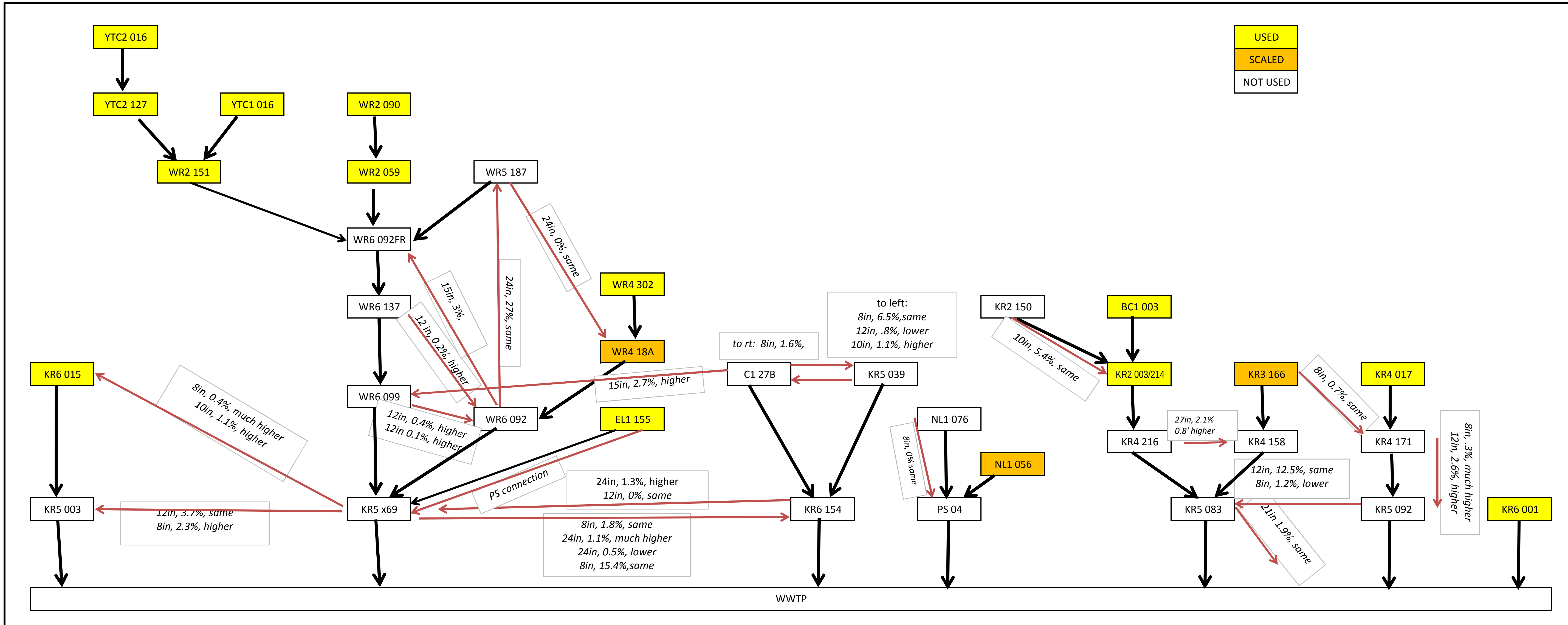


Figure 2.5
Flow Monitoring Diagram
Final Schematic

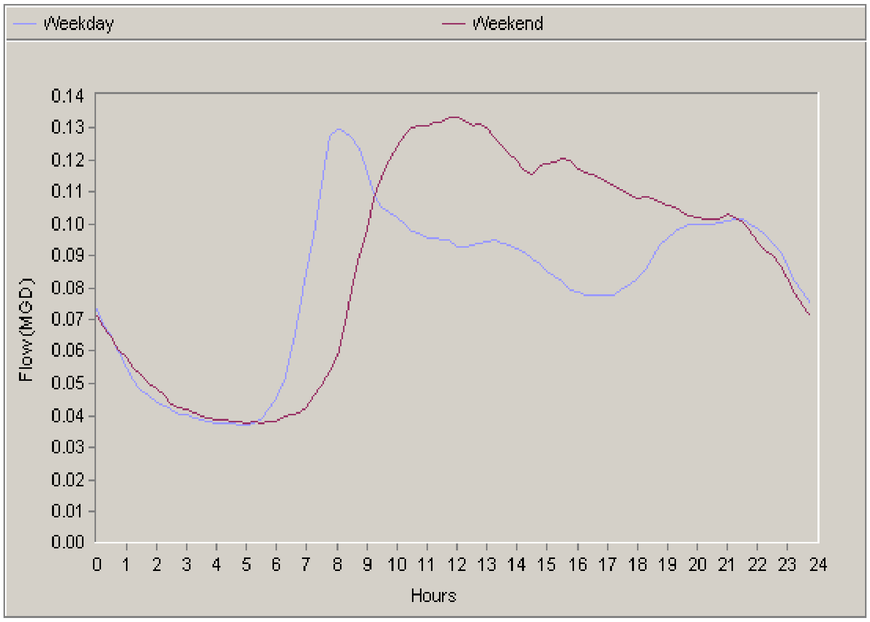


Figure 2.6. Diurnal Curves - Flow Meter BC1 003

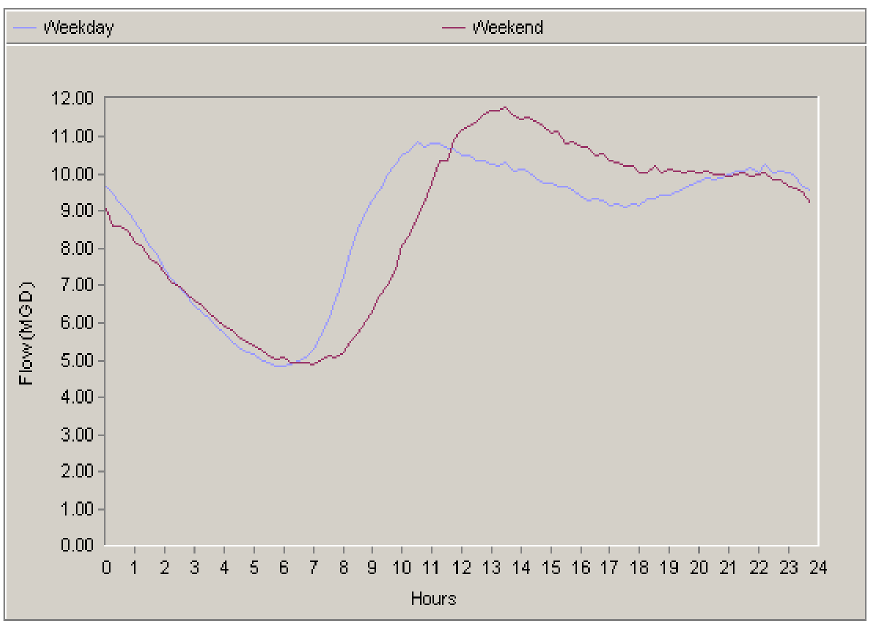


Figure 2.7. Diurnal Curves - Wastewater Treatment Plant

The flow meters and values are shown below in Table 2.6. The last three columns are calculated as previously explained.

Table 2.6
Diurnal Curve Values

Flow Meter	Weekday			Weekend			Week		
	Max (MGD)	Ave (MGD)	Min (MGD)	Max (MGD)	Ave (MGD)	Min (MGD)	Max (MGD)	Ave (MGD)	Min (MGD)
BC1_003	0.130	0.081	0.037	0.133	0.088	0.038	0.131	0.083	0.037
EL1_155	0.444	0.310	0.201	0.456	0.331	0.196	0.448	0.316	0.199
KR2 003/214	1.226	0.808	0.414	1.232	0.844	0.410	1.228	0.818	0.413
KR3_166	0.302	0.230	0.109	0.317	0.233	0.122	0.306	0.231	0.113
KR4_017	0.138	0.097	0.046	0.146	0.102	0.054	0.140	0.099	0.048
KR6_001	0.082	0.054	0.030	0.039	0.033	0.026	0.070	0.048	0.029
KR6_015	0.286	0.212	0.098	0.330	0.229	0.098	0.298	0.217	0.098
NL1_056	0.120	0.090	0.053	0.127	0.090	0.045	0.122	0.090	0.051
WR2_059	0.904	0.613	0.330	0.907	0.643	0.341	0.905	0.622	0.333
WR2_090	0.597	0.428	0.239	0.635	0.464	0.269	0.608	0.439	0.247
WR2_151	0.662	0.420	0.179	0.687	0.444	0.179	0.669	0.427	0.179
WR4_18A	0.969	0.767	0.408	1.151	0.858	0.462	1.021	0.793	0.423
WR3_302	0.376	0.245	0.111	0.414	0.271	0.128	0.387	0.252	0.116
YTC1_016	0.086	0.051	0.021	0.083	0.056	0.023	0.085	0.053	0.022
YTC2_016	0.345	0.170	0.053	0.347	0.193	0.058	0.346	0.176	0.054
YTC1_127	0.397	0.222	0.092	0.383	0.247	0.095	0.393	0.229	0.093
WWTP	10.810	8.675	4.817	11.810	8.671	4.863	11.096	8.674	4.830

Diurnal curves for flow meters are provided in Appendix 2-A.

In Table 2.7 below, the DWF results for each utilized metershed are shown. These are cumulative DWF values for the total tributary system for each meter, and the interior metershed DWF calculated by subtraction of upstream metershed DWFs. The DWF per person per day is shown in the fourth column for the interior metersheds and in last column for the complete metersheds. These were important statistics when combined with the land use information in evaluating the validity of the flow meter data.

Table 2.7
Dry Weather Flow by Metershed and Interior Metershed

(a)	(b)	(c)	(d)	(e)	(f)	(g)
Meter	Interior Metershed Population	Interior Metershed DWF- MGD	Interior Metershed DWF per person gpcpd	Metershed Population	Metershed DWF - MGD	Metershed DWF per person - gpcpd
	From TAZ-Metershed GIS Intersection	Difference of Metershed(s)	DWF / Population	From TAZ-Metershed GIS Intersection	From Table 2.3	DWF / Population
BC1 003				1,133	0.083	82
EL1 155				2,140	0.316	207
KR2 003/214	9,130	0.735	81	10,265	0.818	80
KR3 166				1,795	0.2539	141
KR4 017				783	0.099	126
KR6 001				246	0.048	196
KR6 015				1,424	0.217	152
NL1 056				1,838	0.17	92
WR2 059	4,566	0.183	40	7,345	0.622	85
WR2 090				2,779	0.439	158
WR2 151	2,719	0.145	53	7,763	0.427	85
WR4 18A	8,033	0.541	67	9,208	0.5925	64
WR4 302				1,175	0.252	214
YTC1 016				1,042	0.053	51
YTC2 016				3,289	0.176	53
YTC2 127	713	0.053	74	4,002	0.229	57
WWTP				92,727	8.674	94

Column (g) of Table 2.7 provides a measure of per person DWF as determined from the analyzed flow meter data and estimated population served upstream of each flow meter. This provides a check of the reasonableness of the flow meter data. The per person DWF values shown in column (g) are considered to be reasonable and therefore support a conclusion that the flow data is reasonable.

A comparison was made between SSOAP analyzed DWFs of a group of seven flow meters termed "first tier meters" and DWFs recorded at the KRWWTP. First tier meters are those meters located at points in the collection system where there is no downstream meter between them and the KRWWTP. They include the following meters:

KR5 003, KR5 x69, KR5 083, KR5 092, KR6 001, KR6 154, PS#04

The locations of the first tier meters are shown on Figure 2.8.

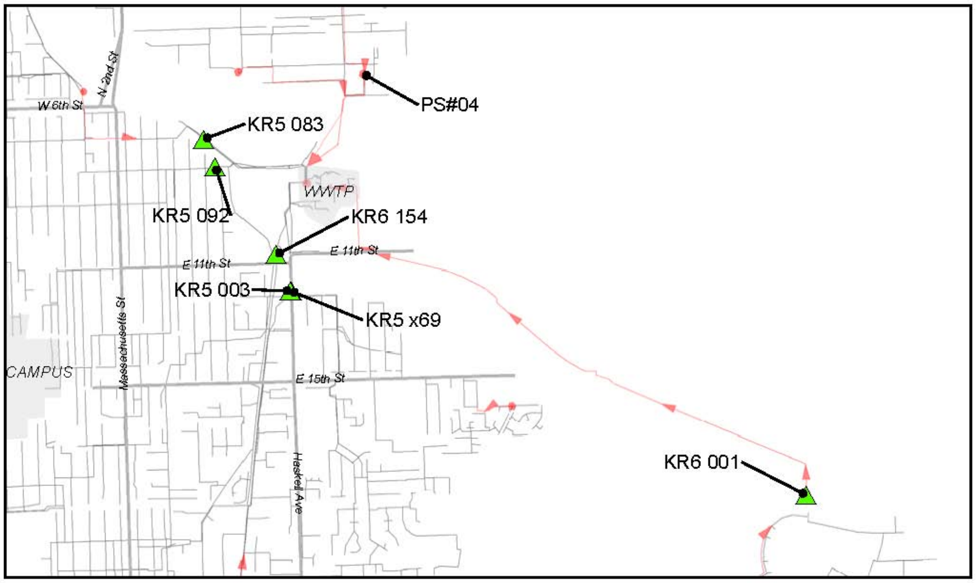


Figure 2.8. First Tier Flow Meters

Initially, this continuity check indicated there was an unacceptable discrepancy between the sum of first tier meter flows and the KRWWTP metered flows, with first tier metered flows being much less than KRWWTP metered flows. As a result of this check, it was subsequently determined that meter KR6 154 was not providing accurate data due to conditions at the meter location. Actual flows were field measured and a correction was made for this flow meter. A means was also determined to estimate flows through a 21-inch sewer parallel to the KR5 083 sewer that was not metered by any of the first tier meters using flow meter data from upstream Pump Station 16. Finally, a correction of flows received from the North Lawrence Drainage Basin was made using operating data for Pumping Station 4. With these corrections, it was possible to arrive at a reasonable agreement between the first tier meter flows and KRWWTP metered flows.

2. Rainfall Derived Infiltration and Inflow

Rainfall derived infiltration and inflow (RDII) is additional groundwater infiltration that is associated with higher than normal groundwater immediately following a wet weather event, plus inflow of stormwater runoff into the sewer system through sources such as leaking private sewer laterals, building roof drains and foundation drains connected to the sewer, and other sewer and manhole defects. Levels of RDII in a collection system can vary significantly depending on rainfall intensity and duration, and the condition of the system. The relative condition of collection system sub-basins can be measured in part by RDII levels that occur during comparable wet weather events.

RDII flows are estimated using the SSOAP toolbox program. For the SSOAP analysis, a rain gauge file was created for each flow meter that apportioned the influence of nearby rain gauges. The inverse of the square of the distance from the centroid of each metershed was used to calculate the influence of each rain gauge. Figure 2.9 shows an example of this procedure graphically for flow meter YTC2 016 and Table 2.8 lists the composite rain gauge information by flow meter.

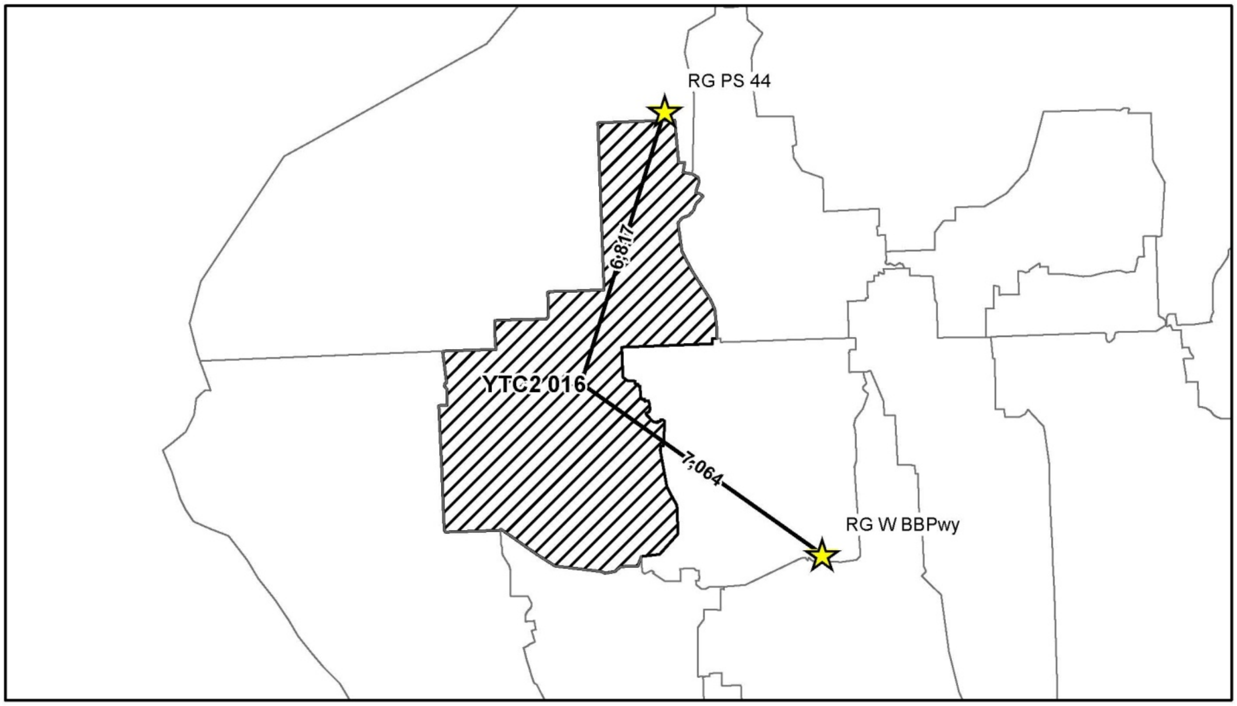


Figure 2.9. Rain Gage Apportioning Example

Table 2.8

Rain Gage Allocation to Metersheds

		RAIN GAGE							
		RG PS 44	RG WBBPwy	RG PS 46	RG PS 05	Stratford	RG PS 09	RG PS 49	RG PS 04
FLOW METER	BC1 003 PS48	86%	14%						
	EL1 155 PS5				85%			15%	
	KR2 003/214	53%	14%	23%		10%			
	KR3 166 2ND		7%	18%		66%			9%
	KR4 017					100%			
	KR6 001 EHBP				14%			86%	
	KR6 015 1221				63%			17%	20%
	NL 1 056 PS4			18%		13%			69%
	WR2 059 PS9		99%				1%		
	WR2 090 QUAI	11%	89%						
	WR2 151 PS9	20%	63%				17%		
	WR4 18A 2301		27%			56%	17%		
	WR4 302	20%	34%			46%			
	YTC1 016 LK	33%	48%				19%		
	YTC2 016	52%	48%						
	YTC2-127	31%	69%						

Rainfall events were then examined in SSOAP, where the system response could be compared visually with the rain gauge data, RDII flow, and the constructed DWF curves. Based on the rain gauge data, there were approximately 250 events available for analysis for most of the flow meters. An initial set of event choices used every event that was sufficiently isolated from previous and successive events. This set of about 60 events for each meter yielded some outliers and unexpected results, and these events were sorted such that the storms that produced the most RDII for a given rainfall depth were plotted. Ultimately, a fairly narrow range of storm durations and storm intensities was used to develop RDII peaking factors.

These plots were used to draw regression lines so that the peaking factors for one- and two-inch storms could be found with a fair amount of confidence. After the final sort there were about 12 events plotted for each meter with average storm duration of 7.22 hours. An example of a final set of storms and peaking factors is shown below in Figure 2.10 for meter EL1 155.

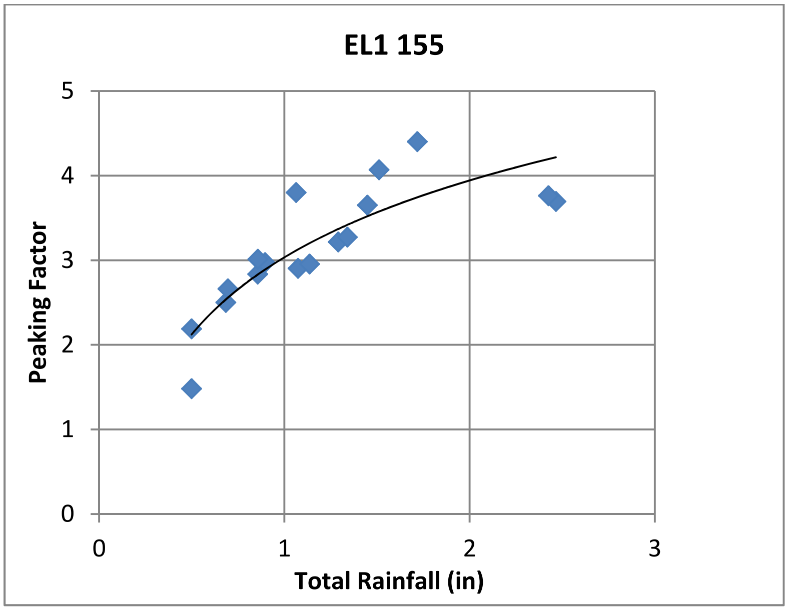


Figure 2.10. RDII Peaking Factors versus Rainfall

RDII peaking factors for all analyzed flow meters are provided in Appendix 2-B.

The line of regression should be expected to be a curve that points to an upper limit of peak flows as the local system nears its capacity, and this line should cross the vertical axis near a peaking factor of 1 as the flow approaches the DWF rate at a zero storm depth. This example flow meter is best characterized with a logarithmic curve, but some flow meters had a better fit with a linear regression line. This should be expected because the storms analyzed would not necessarily indicate an upper limit to the peak flow at every point in the system. Using this methodology and flow meter EL1 155 as an example, a peaking factor (PF) of 3.9 can be inferred for the 2-inch storm, and a PF of 3.0 can be inferred for the 1-inch storm. The average 2-inch storm PF was not observed to be twice the 1-inch PF, which was expected. The PFs for the 2-inch and 1-inch rain events are listed in Table 2.9.

Table 2.9
Peaking Factors

Flow Meter	PF 2-inch Storm	PF 1-inch Storm	2in/1in PF Ratio
WWTP	3.5	2.3	1.5
BC1 003	7.6	4.6	1.7
EL1 155	3.9	3.0	1.3
KR2 003/214	6.5	4.2	1.4
KR3 166	4.9	2.7	1.8
KR4 017	11.3	5.3	2.1
KR6 001	7.7	4.0	1.9
KR6 015	10.5	5.5	1.9
NL1 056	8.8	5.8	1.5
WR2 059	4.4	2.6	1.7
WR2 090	4.4	3.4	1.3
WR2 151	5.7	3.4	1.7
WR4 18A	5.1	2.8	1.8
WR4 302	6.7	4.0	1.7
YTC1 016	6.2	4.0	1.6
YTC2 016	5.4	3.3	1.6
YTC2 127	6.5	5.0	1.3
AVERAGE	6.4	3.9	1.7

The PF multiplied by the average DWF gives the peak flow in the sanitary system for the given rain event. Hence, the DWF* (PF -1) gives the portion of the peak flow rate that is attributable to the RDII for a given rain even.

E. Metershed Ranking by RDII

The interior metershed ranking was then found by comparing the peak RDII flow rate per inch diameter mile (IDM) sewer for the interior metersheds. The interior metersheds that included the University of Kansas campus have large poorly documented sanitary systems that would skew the IDM values because of the missing network information, so the metershed IDMs affected by the campus were increased proportionally according to the percentage of the campus area to the total interior metershed area. The interior metershed rankings according to RDII per IDM, or relative "leakiness," are listed in Table 2.10.

Table 2.10
Interior Metershed Rankings

(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)
Flow Meter	Peaking Factor for the 2-inch Event	DWF (MGD)	Flow in the 2-in Event (MGD)	RDII in the 2-in Event (MGD)	IDM	RDII / IDM (gpd/IDM)	Rank	Sewered Area (ac)	RDII / Sewered Area (gpd/ac)	RDII / Sewered Area Rank
	From Table 2.6	From Table 2.4	PF * DWF (interior)	2-in flow – DWF	Calculated from GIS data					
BC1 003	7.6	0.083	0.631	0.548	64.7	8,467	7	151	4,255	5
EL1 155	3.9	0.316	1.232	0.916	111.0	8,256	8	506	2,161	10
KR2 003/214	6.5	0.265	1.7225	1.458	418.8	2,717	15	1,732	2,942	7
KR3 166	4.9	0.254	1.245	0.991	63.9	15,502	5	221	4,701	4
KR4 017	11.3	0.099	1.119	1.020	15.8	64,538	1	59	17,283	1
KR6 001	7.7	0.048	0.370	0.322	72.2	4,455	12	228	2,084	11
KR6 015	10.5	0.217	2.279	2.062	46.3	44,488	3	183	10,272	3
NL1 056	8.8	0.340	2.992	2.652	139.7	18,984	4	655	2,426	9
WR2 059	4.4	0.183	0.8052	0.622	183.7	3,387	17	640	617	17
WR2 090	4.4	0.439	1.932	1.493	124.8	11,962	6	560	4,108	6
WR2 151	5.7	0.145	0.8265	0.682	145.5	4,684	11	332	2,607	8
WR4 18A	5.1	0.592	3.0192	2.427	303.4	8,000	9	995	1,620	13
WR4 302	6.7	0.252	1.688	1.436	31.3	45,939	2	132	11,600	2
YTC1 016	6.2	0.053	0.329	0.276	68.9	4,002	13	245	1,153	15
YTC2 016	5.4	0.176	0.950	0.774	201.6	3,841	14	544	1,432	14
YTC2 127	6.5	0.053	0.3445	0.292	84.9	3,433	16	220	1,100	16

(1) Highest RDII/IDM metershed is ranked 1, lowest is ranked 17.

(2) IDM is adjusted for KU campus from KU Main estimates.

A graphical interpretation of these results is shown in Figure 2.11, where the RDII rankings are used to color the metersheds (worst to best is symbolized by coloring the metershed from reds to yellow to greens), and the age of the infrastructure is shown using similar symbolization.

Because various methodologies were employed to arrive at these ratings and rankings, they may be compared to check the reasonableness and consistency of results as presented in Table 2.10 as measured by RDII production per sewered acre. The RDII production per sewered acre produced rankings that are very similar to those based on RDII/IDM.

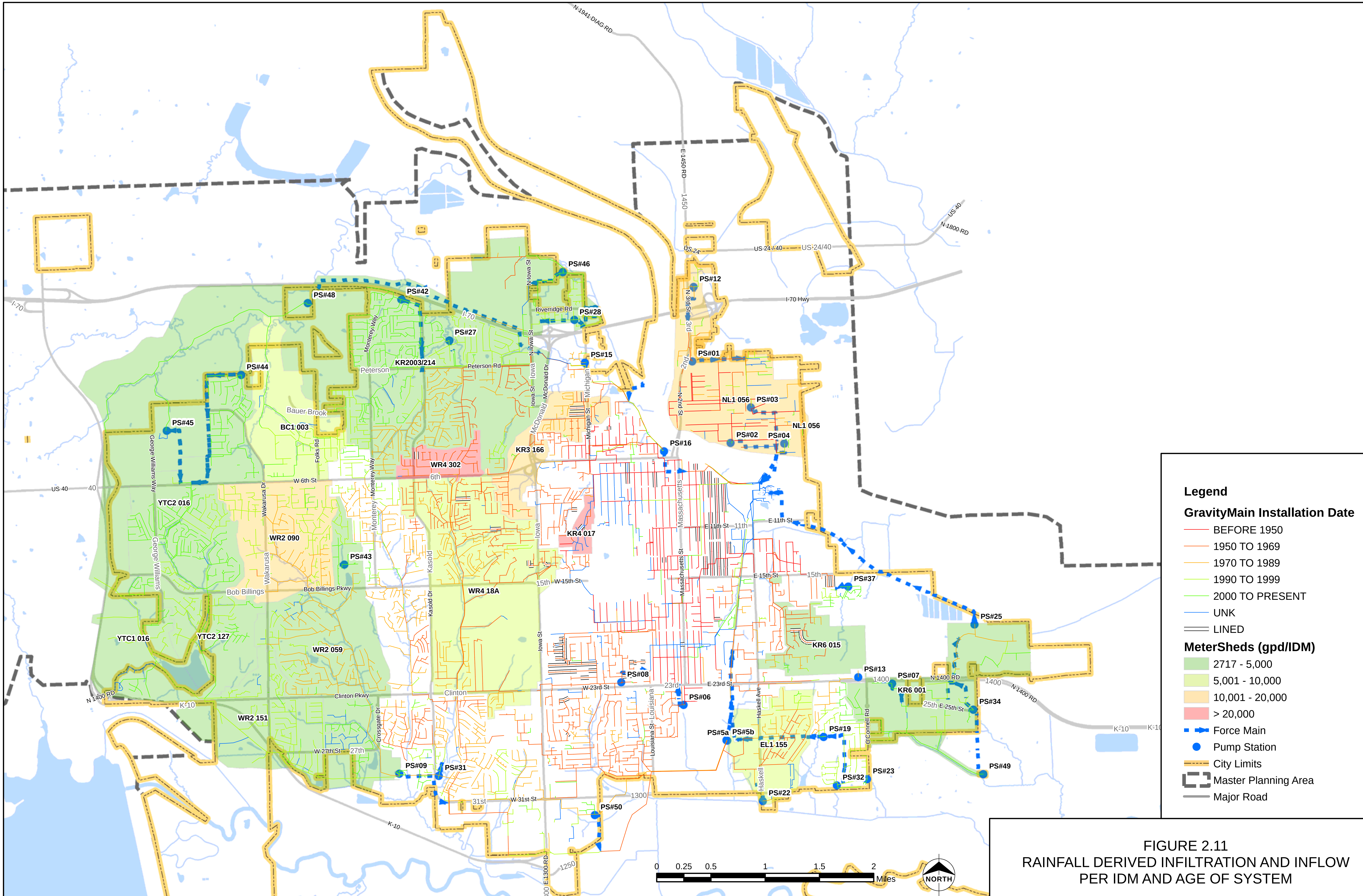


FIGURE 2.11
RAINFALL DERIVED INFILTRATION AND INFLOW
PER IDM AND AGE OF SYSTEM

F. Existing System Flows and Capacity Analysis

1. Hydraulic Model Development

a. Model Construction

A hydraulic model of the wastewater collection system was prepared including all gravity sewers, manholes, pumping stations and force mains, and storage facilities. The model was prepared from the City's wastewater geodatabase. The existing system model database includes approximately 10,250 sewer pipe elements, matching the quantities in the geodatabase. Further details of the model development are presented in Appendix 2-C.

b. Existing Flow Development

Model existing wastewater flows were developed to match the dry weather flow and rainfall derived infiltration and inflow components previously described. Flows are developed by the model by component for an analysis of the collection system as set forth below.

Dry Weather Flow (DWF)

DWF includes Wastewater Flow (WWF) discharged by the utility's customers and is measured by winter quarter metered water usage. Metered water usage was input to the model using a geodatabase of each utility customer's metered water usage. Areas closest to each manhole were determined by the software, and the winter quarter metered water usage within each area was used to allocate the DWF (WWF plus DWI), and input to the model at each manhole. WWF rates typically vary throughout the day. These diurnal flow patterns at each flow meter were developed previously and used by the model to input DWF at each manhole to simulate the daily variation of flow.

Rainfall Derived Infiltration/Inflow (RDII)

Rainfall derived infiltration and inflow (RDII) was input to the model within metersheds in accordance with how the collection system responds to rainfall events within each metershed as previously analyzed. For metersheds where it was not possible to analyze RDII levels, RDII levels in other analyzed metersheds were used that have comparable system age and pipe materials. RDII is input to the model at various loading manholes or nodes located throughout the system.

RDII is estimated by developing a storm hydrograph specific to each flow monitoring location for each rainfall event. The curve-fitting procedure uses RTK variables, which consists of 9 parameters that were manually determined for each meter at each rainfall event. During calibration, peak flow rate and total volume predicted by the model are compared to observed flow monitoring data and RTK parameters are adjusted until a reasonable fit has been established. R is the percent of rainfall that enters the system and can vary from storm to storm. R is dependent on antecedent moisture conditions, where wet conditions with soil already saturated usually have higher R values than dry conditions. T and K parameters, which define the shape of the wet weather response curve, represent time to peak and recession of the curve. A normal limitation of modeling is the insertion of RDII into the system model at selected inflow points, rather than inserting RDII at every manhole in the system.

c. Model Calibration and Validation

The model was calibrated to duplicate flows occurring during an actual wet weather event using rainfall and flow meter data. Various model flow input parameters were adjusted until a good correlation was developed between modeled flows and actual metered flows, including flows measured at the wastewater treatment plant. Once calibrated, a second specific wet weather event using actual rainfall data was modeled and the model output checked for correlation with the flow meter data to verify or validate the accuracy of the model. The model calibration and validation wet weather events were more frequently occurring rainfall events of 1 to 3 inches. A further validation was performed using a wet weather event that occurred in May 2009 with total rainfall measuring 2.9 inches over a 6-hour period, which corresponds well with the 10 year design storm. This event resulted in a peak flow rate of 68 MGD at the wastewater treatment plant. The model was then refined to produce a reasonable correlation with actual flows at both the smaller rainfall validation storms and the May 2009 storm. Table 2.11 summarizes a comparison of metered and modeled peak wet weather flows for the calibration and validation storms.

Table 2.11
Wet Weather Calibration Results

Flow Meter	Event	Depth - in	Duration - hr	Predicted / Observed	
				Volume (%)	Peak (%)
BC1 003	8/31/2010	1.21	15.00	100%	100%
	7/11/2010	0.86	1.00	78%	44%
EL1 155	3/2/2008	0.82	10.00	97%	99%
	3/17/2008	1.52	16.00	96%	103%
KR2 003/214	8/31/2010	1.21	15.00	119%	*
	7/11/2010	0.86	1.00	66%	*
KR3 166	8/31/2010	1.21	15.00	114%	99%
	7/11/2010	0.86	1.00	105%	97%
KR4 017	3/2/2008	0.82	10.00	99%	102%
	3/17/2008	1.52	16.00	101%	98%
KR6 001	3/2/2008	0.82	10.00	89%	*
	3/17/2008	1.52	16.00	85%	*
KR6 015	3/2/2008	0.82	10.00	100%	111%
	3/17/2008	1.52	16.00	102%	70%
PS 04	3/2/2008	0.82	10.00	95%	**
	3/17/2008	1.52	16.00	88%	**
WR2 059	3/2/2008	0.82	10.00	101%	100%
	3/17/2008	1.52	16.00	96%	96%
WR2 090	3/2/2008	0.82	10.00	106%	96%
	3/17/2008	1.52	16.00	93%	85%
WR2 151	3/2/2008	0.82	10.00	103%	101%
	3/17/2008	1.52	16.00	109%	98%
WR4 18A	8/31/2010	1.21	15.00	29%	408%
	7/11/2010	0.86	1.00	150%	113%
WR4 302	3/2/2008	0.82	10.00	100%	102%
	3/17/2008	1.52	16.00	106%	90%
YTC1 016	3/2/2008	0.82	10.00	101%	104%
	3/17/2008	1.52	16.00	104%	124%
YTC2 016	3/2/2008	0.82	10.00	98%	97%
	3/17/2008	1.52	16.00	100%	98%
YTC2 127	8/31/2010	1.21	15.00	140%	112%
	7/11/2010	0.86	1.00	118%	94%
WWTP	3/2/2008	0.82	10.00	91%	112%
	3/17/2008	1.52	16.00	95%	104%

* Model produces very large spikes with every pump cycle, hence peaks are not comparable.
** Only the daily running time for PS 04 was available for calibration of North Lawrence, so no peak flow is available.

The comparison of model predicted and observed volumes and peak flow rates shown in Table 2.11 is shown to provide an indication of the calibrated model's ability to simulate actual wet weather flow conditions in the collection system. Based on these comparisons, the model's ability to simulate actual wet weather flow conditions is considered to be reasonable.

2. Design Storm

The design storm selected for system evaluation is the 10-year storm, i.e., a storm that has a 10% chance of occurring in any given year. There is not at this time any State or federal regulatory standard directing the use of a 10-year storm or any other design storm when evaluating the performance of municipal wastewater collection systems. Federal rules concerning wet weather performance of separate sanitary sewer systems have been under development for many years and it is not clear at this time if a level of performance based on a design storm will be adopted. This design storm was selected with input from City staff because it is considered to provide a level of system performance and reliability that is consistent with what is now commonly practiced for evaluating the performance of municipal wastewater collection systems in the United States. The model was used to estimate the time of concentration of flows arriving at the wastewater treatment plant by simultaneously inputting a continuous flow rate into each manhole and modeling the time required for the flow at the wastewater treatment plant to reach a maximum. Based on this analysis, a 6-hour storm duration was used for the design storm. A 10-year, 6-hour storm at Lawrence, Kansas has a maximum intensity of 2.60 inches per hour and a total rainfall depth of 3.95 inches. Figure 2.12 is a cumulative hyetograph of the storm used for this analysis.

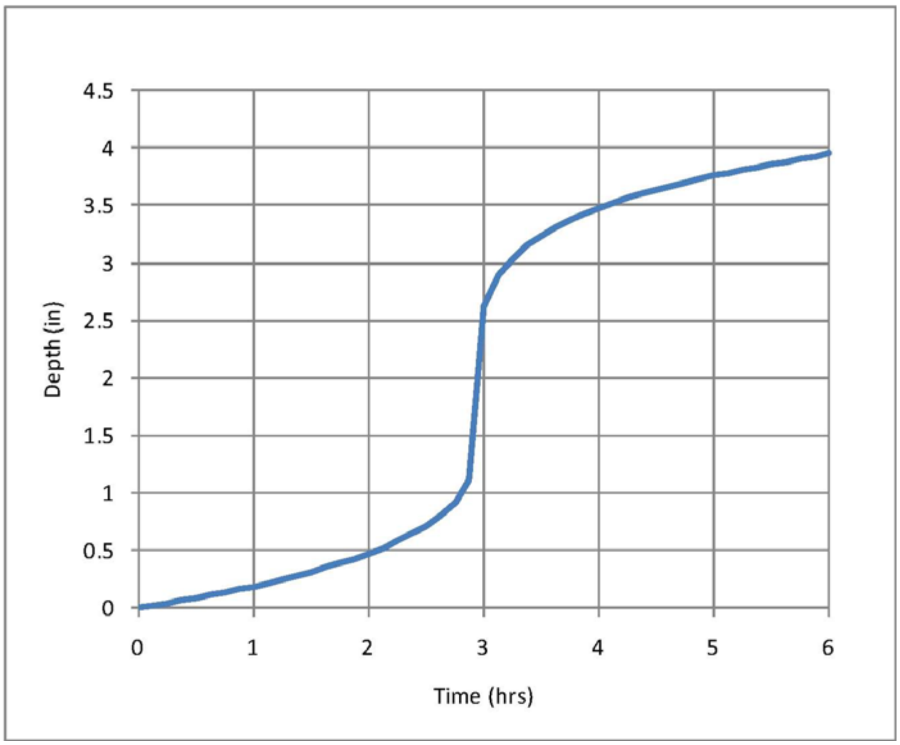


Figure 2.12. Cumulative Rainfall for 10-year 6-hour Storm

When the design storm is inserted into the model it is set to begin at 6:00 a.m. so that the peak intensity occurs at 9:00 a.m. This makes the peak flow rate from the RDII arrive at the WWTP near the same time of the day the maximum expected dry weather flow rate occurs.

3. Results of Analysis

a. Existing System

A summary of the existing system model at the design storm is depicted on Figure 2.13. Surcharged sewers are highlighted in yellow. A flow hydrograph predicted by the model at the KRWWTP is shown on Figure 2.14. The actual flow hydrograph that occurred at the KRWWTP during the May 2009 storm event is also shown to illustrate the correlation between the modeled design storm event and an actual similar but lesser storm event.

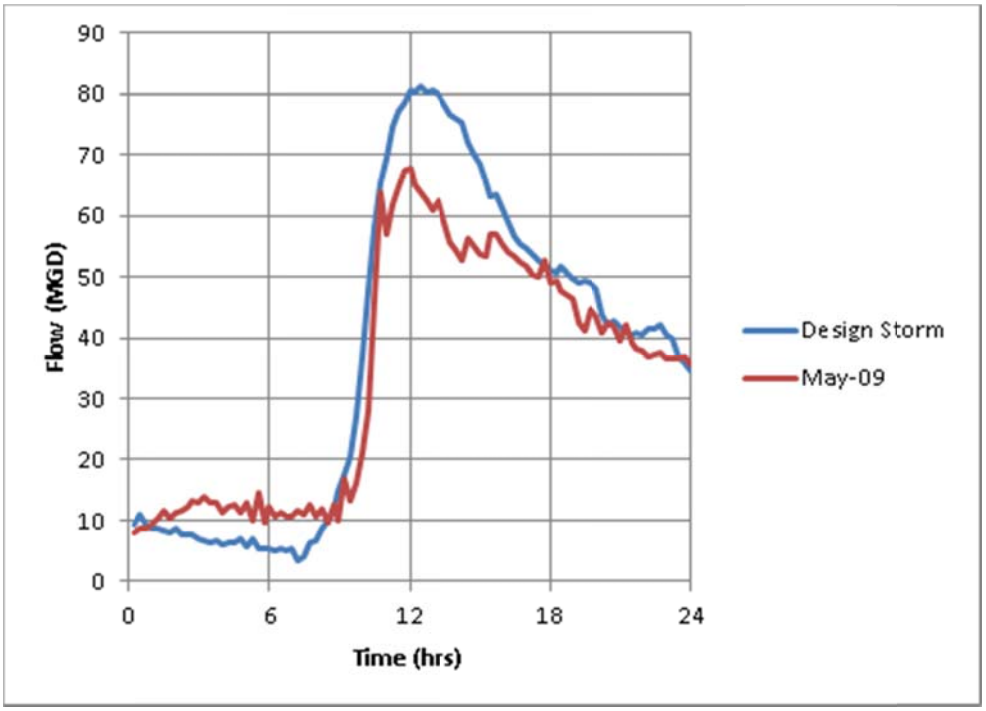


Figure 2.14. Existing System Design Storm and May 2009 Storm Flow Hydrographs at Kansas River WWTP

The model predicts an instantaneous peak flow rate of 81 MGD. This result is considered to be reasonable when compared to the instantaneous peak flow rate of 68 MGD measured during the May 2009 storm event. Based on an analysis of the model results, the following conclusions concerning the adequacy of the existing collection system can be made:

- The instantaneous peak flow rate of 81 MGD exceeds the KRWWTP peak flow firm capacity of 65 MGD.

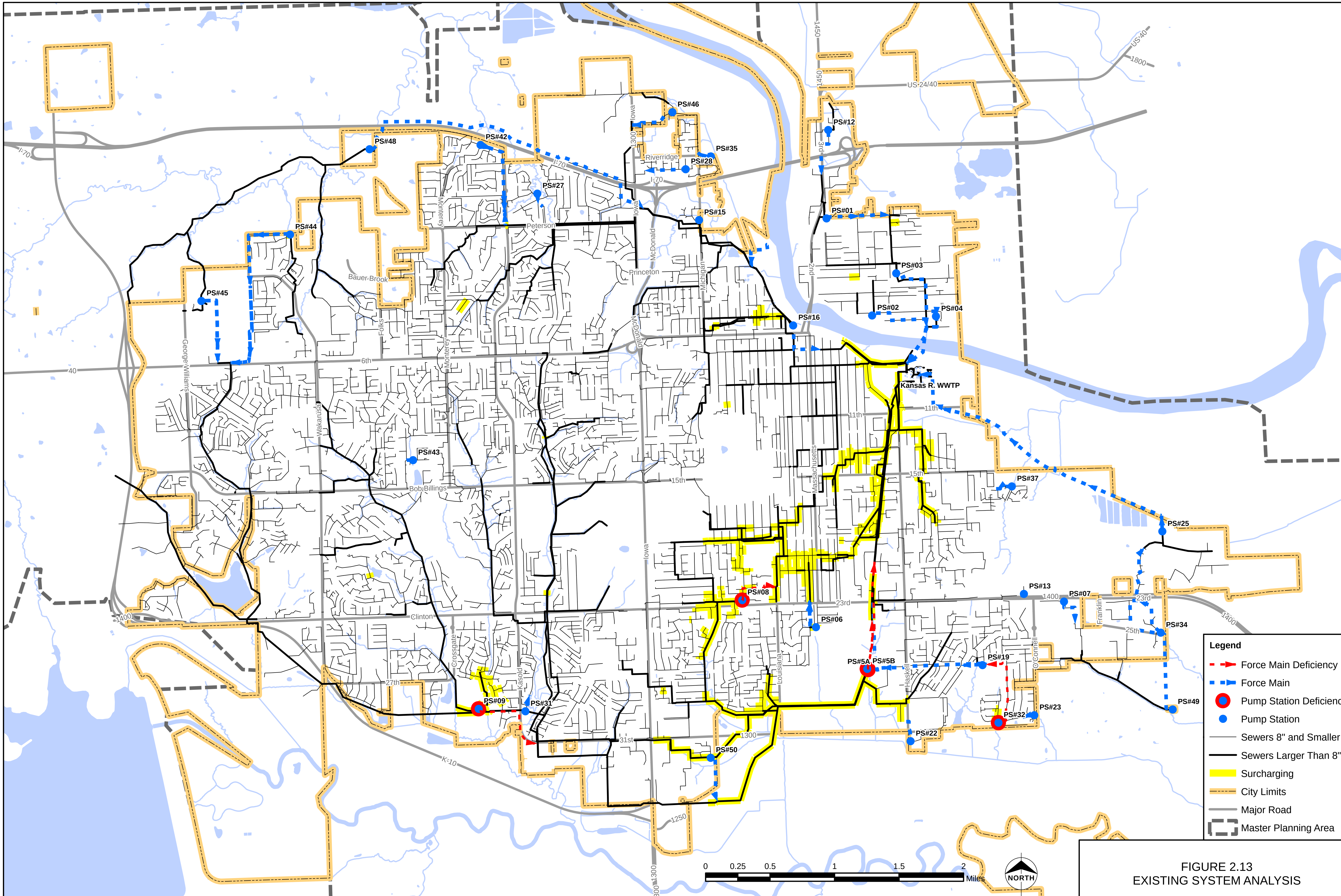


FIGURE 2.13
EXISTING SYSTEM ANALYSIS

- Many of the sewers the model predicts to be overloaded or surcharged are upstream of pumping stations which the model indicates do not have sufficient firm pumping capacities. In these cases, the pumping station wet well levels rise above the crowns of the upstream sewers causing them to be surcharged. The surcharged conditions in these cases are not due to inadequate sewer capacities but to inadequate pumping station capacities. This is the case for surcharged sewers upstream of Pumping Station Nos. 5A/5B, 8, 9 and 32.
- Some surcharging of sewers downstream of Pumping Station Nos. 5A/5B and 8 is due to gravity sewer capacities less than peak flow rates predicted by the model.
- The KRWWTP flow hydrograph shown on Figure 2.14 shows a very quick flow response following the beginning of the storm event up to the instantaneous wet weather peak flow rate, followed by a relatively rapid decline in flow rate which is then followed by a period of steady sustained flow higher than DWF before flows return to normal. These characteristics are typical of actual significant storm events such as the May 2007 validation storm. From an existing system capacity analysis perspective, the ability of the system to convey and treat at the peak instantaneous flow rate is more critical to system performance rather than the total volume of wet weather flows. As such, the most immediate concern is limiting peak flow rates. The quick response to the storm event up to the peak flow rate means there are appreciable infiltration/inflow (I/I) sources that rapidly contribute I/I to the system that are relatively close to the KRWWTP. This is as expected based on the RDII rankings of metersheds presented earlier which identified higher RDII levels for the older parts of the collection system that are relatively close to the KRWWTP. This suggests the rehabilitation/replacement plan set forth in Technical Memorandum No. 4 should make removal of rapid I/I sources in close proximity to the KRWWTP a priority.

Table 2-12 presents a summary of existing system deficiencies at the design storm event.

Table 2.12
Summary of Existing System Deficiencies

Drainage Sub-Basin	Description	Existing Peak Capacity-MGD(1)	Design Storm Peak Flow-MGD
C-2	PS 08	2.9	12.7
C-2	PS 08 Force Main	3.3	12.7
EL-1B	PS 23	0.05	0.05
EL-1B	PS 32	0.7	1.6
EL-1B	PS 32 Force Main	0.8	1.6
KR-5C	12-Inch Sewer	1.0	2.9
KR-6B	Kansas River WWTP	65	81
KR-6B	21-Inch Sewer	4.0	9.4
WR-1	PS 09	8.6	11
WR-6	PS 5A/5B	15.5	24
WR-6	PS 5A/5B Force Mains	15.5	24

(1) Pumping station capacities shown are based on firm pumping capacities.

b. Existing System Analysis Assumptions Moving Forward

(1) Future Infiltration/Inflow Reduction

The City will be continuing efforts to rehabilitate and replace aging and defective components of the collection system in accordance with a plan set forth in Technical Memorandum No. 4. As stated earlier, priority will be given to removal of rapid I/I sources relatively close to the KRWWTP. This will reduce levels of RDII in the future, thereby reducing the required capacities of parts of the existing collection system and the KRWWTP. As such, this plan assumes it is not necessary to address existing system problems that will not occur after completing a program of further I/I reduction. A reduction of 35% of I/I as measured by peak flow rates within the targeted area is considered to be achievable over a reasonable period of time based on a survey of I/I reduction programs implemented by other cities. From this point forward, the existing system analysis is modified to reflect a 35% reduction of I/I within the targeted area.

(2) Future Wakarusa Wastewater Treatment Plant

As stated earlier, average dry weather flows and their associated pollutant loadings plus peak wet weather flow rates will exceed the capacities of the KRWWTP at some time during the master planning period. This will require the construction of a new wastewater treatment plant which this plan will name as the Wakarusa Wastewater Treatment Plant (Wakarusa) located south of the Wakarusa River on a site owned by the City. Based on projections set forth in Technical Memorandum No. 3, this will need to occur sometime before 2020. As such, this plan assumes it is not necessary to address existing system problems that will not occur after a portion of flows are diverted to the future Wakarusa. Flow diversion to the future Wakarusa will involve at a minimum the pumping of some or all flows that exceed the capacities of Pumping Station Nos. 5A/5B to the south and east to the Wakarusa. The plan should maximize utilization of the KRWWTP, thereby minimizing the initial constructed capacity of the future Wakarusa. As such, the existing system analysis from this point forward is based on modeled conditions that occur with this flow diversion.

c. Flow Diversion to Future Wakarusa Wastewater Treatment Plant Analysis and Plan

(1) Background

Previous planning for the future Wakarusa included diverting flows from Pumping Station No. 9 south of the Wakarusa River and then east to the future treatment plant site. Since the time the previous planning was completed, some important planning factors have changed, making it appropriate to re-evaluate the flow diversion plan. First, the previous planning was based on a year 2025 service area population forecast of 153,000. The majority of the population growth was expected to occur within the Baldwin Creek, Yankee Tank Creek and Wakarusa South drainage basins. This growth would result in significantly higher flow rates to Pumping Station No. 9. This plan forecasts service area population to increase to only 119,529 in year 2030. The lower population forecast will result in significantly smaller increases in flow rates to Pumping Station No. 9. Second, peak flow rates seen at the KRWWTP have increased significantly from those recorded at the time previous planning was done. This will require potentially significant diversions of wet weather peak flows to the Wakarusa from areas that are in close proximity to the KRWWTP in order to limit peak flow rates to the KRWWTP to 65 MGD and to address surcharged sewer conditions downstream of Pumping Station Nos. 5A/5B and 8.

(2) Flow Diversion from Pumping Station No. 9

The first flow diversion plan investigated was diversion of flows from Pumping Station No. 9 as previously planned. The existing system model was run incorporating the 35% system infiltration/inflow reduction plan previously described and the diversion of all flows from Pumping Station No. 9 to the future Wakarusa. The model determined these steps alone are not nearly sufficient to reduce the peak flow rates in the system tributary to Pumping Station Nos. 5A/5B to its firm pumping capacity. This is because the wet weather peak flow storage at Pumping Station No. 9 effectively minimizes the impact of

Pumping Station No. 9 peak flow discharges to the downstream collection system. Since diversion of flows from Pumping Station No. 9 has limited impact on peak flow rates that occur within the downstream collection system, an alternate diversion plan is needed.

(3) Flow Diversion from Pumping Station Nos. 5A and 5B

The next plan investigated was diversion of flows from Pumping Station Nos. 5A /5B. This pumping station has a large capacity and is relatively close to the KRWWTP, making it more likely to have an impact on the peak flow rate seen at the KRWWTP. It is also necessary in any event to reduce peak flow rates to the firm capacities of Pumping Station 5A and 5B. This is also needed to address surcharged sewer conditions upstream of Pumping Station Nos. 5A/5B. It is not desirable to expand this station at its current location due to site constraints. For that reason, a separate and new pumping station upstream of the current site and intercepting a portion of the flows from the gravity sewer running parallel to 27th Street would be constructed. A force main would also be constructed routed west and then south across the Wakarusa River to the Wakarusa site, coordinated with plans for the construction of the South Lawrence Trafficway and related wetland mitigation areas. The most appropriate site appears to be near the northwest intersection of 31st and Louisiana Streets. When this flow diversion is modeled, there is a reduction of the peak flow rate seen at the KRWWTP. The capacities of the existing Pumping Station Nos. 5A/5B, plus this new station need to be sufficient for handling the design storm peak flow rates and achieve a peak flow diversion to the Wakarusa necessary to limit the KRWWTP peak flow rate to 65 MGD.

(4) Conclusions

Of the two flow diversion plans investigated, the diversion of flows upstream of Pumping Station Nos. 5A/5B will achieve necessary reductions of peak flow rates to the KRWWTP while at the same time providing additional firm pumping capacity needed to supplement existing 5A/5B firm pumping capacity. Technical Memorandum No. 3 addresses conditions forecast to occur in 2020 and 2030 including future capacity requirements for Pumping Station No. 9. With that information, it will be possible to determine if this flow diversion plan which is based on existing conditions will need to be modified.

G. Existing System Improvements

Existing system improvements are needed to correct current system limitations that remain after incorporating assumptions about the future Wakarusa and I/I reduction. In summary, they include the following:

- A new pumping station upstream of 5A/5B to provide the necessary firm pumping capacities at the design storm peak flow rate and accomplish the diversion of some dry and wet weather flows to the future Wakarusa. This new pumping station is identified as Pumping Station No. 5C and would need to have an initial firm peak flow pumping capacity of 8.5 MGD.
- Expansion of capacity of Pumping Station 8 including its force main, or alternatively the elimination of Pumping Station No. 8 and its force main by a new gravity sewer to convey flows south into gravity sewers that drain to Pumping Station Nos. 5A/5B and further expansion of Pumping Station Nos. 5A/5B and its force mains. The gravity sewer plan is preferred since it will eliminate Pumping Station No. 8.
- Expansion of capacity of Pumping Station Nos. 9, 23 and 32.
- Relief sewers as needed to correct sewer surcharging due to inadequate gravity sewer capacities.

H. Existing System Analysis with Improvements and Future System Assumptions

A summary of the existing system model at the design storm incorporating the assumptions just described concerning the future Wakarusa and 35% I/I reduction, plus the required pumping station expansions and relief sewers, is depicted in Figure 2.15 and the resulting flow hydrograph at the KRWWTP is shown in Figure 2.16.

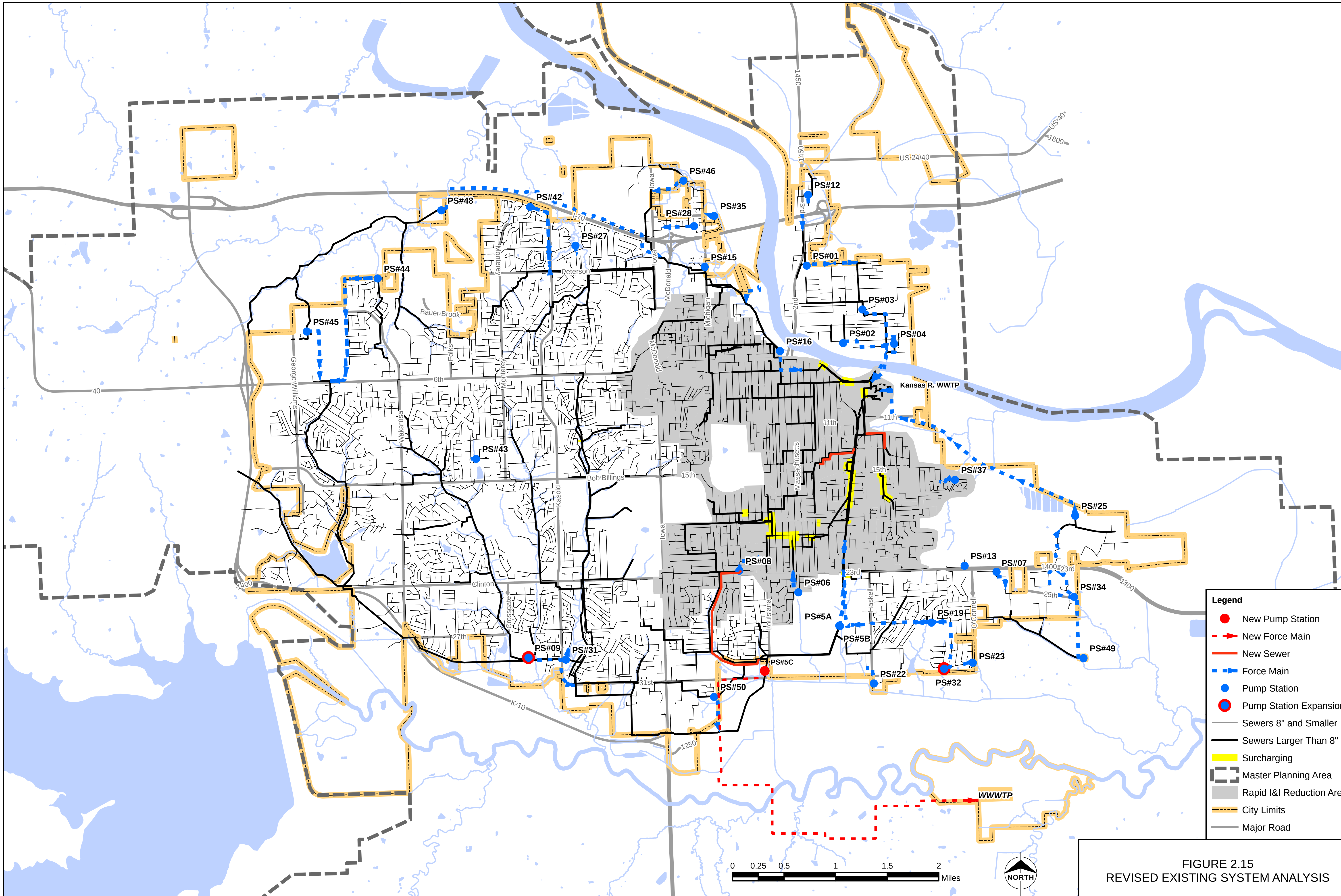


FIGURE 2.15
REVISED EXISTING SYSTEM ANALYSIS

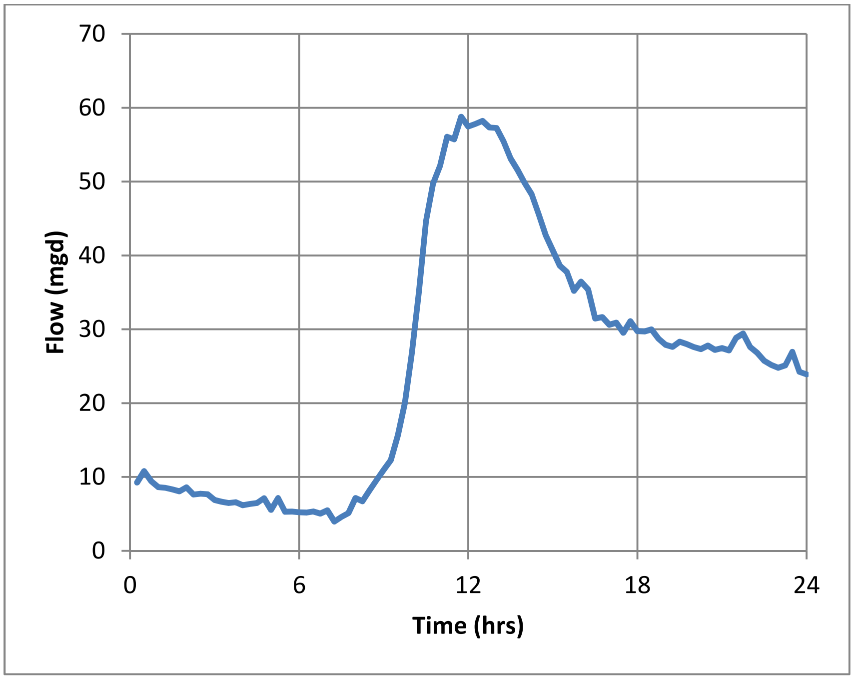


Figure 2.16. Improved System Design Storm Hydrograph at Kansas River WWTP

The model predicts the instantaneous peak flow rate at the KRWWTP to be reduced from 81 MGD to 59 MGD. Given the accuracy of the model and the likelihood there will be some further increase in peak flow rate due to projected growth in the service area, this result is considered to be acceptable in terms of KRWWTP peak flow capacity.

Technical Memorandum No. 3 addresses the forecast of future flows for planning years 2020 and 2030. The required capacities of these and other parts of the existing collection system may need to be increased further to address the forecast of future flows. As such, improvements to address existing conditions are included in Technical Memorandum No. 3 in order to provide for the additional capacity that may be needed for the forecast of future flows. These improvements are scheduled early in the capital improvements program set forth in Technical Memorandum No. 5.

* * * * *

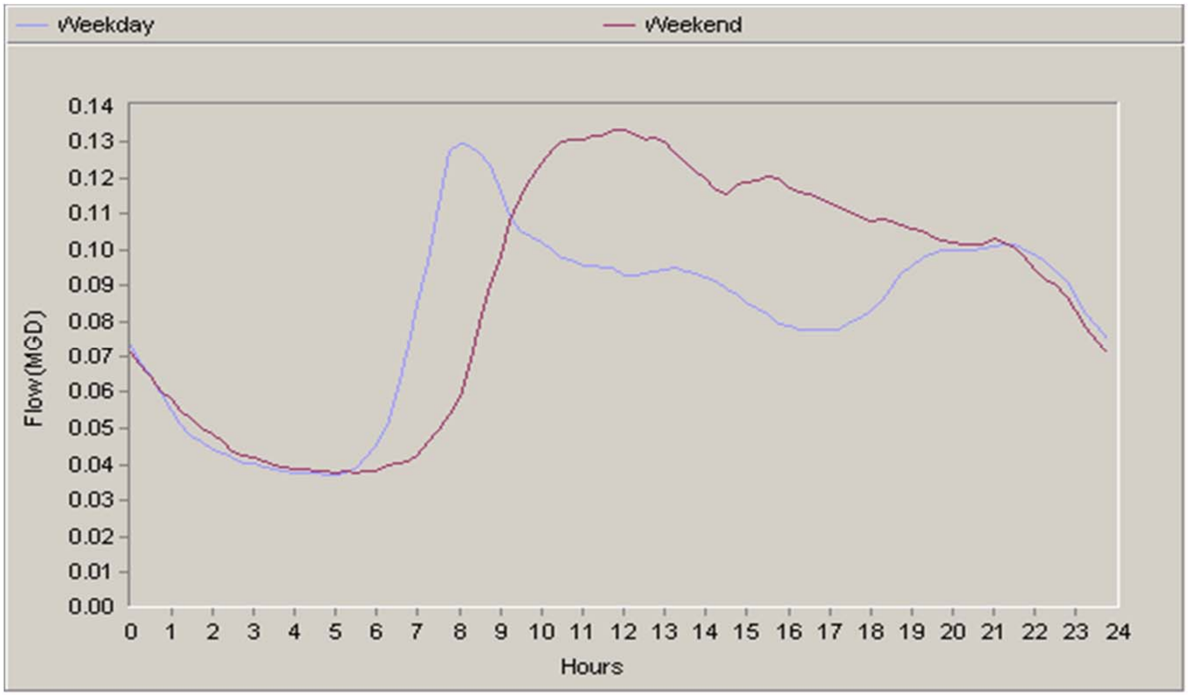
Appendix 2-A
Dry Weather Flows and Diurnal Curves

APPENDIX 2A

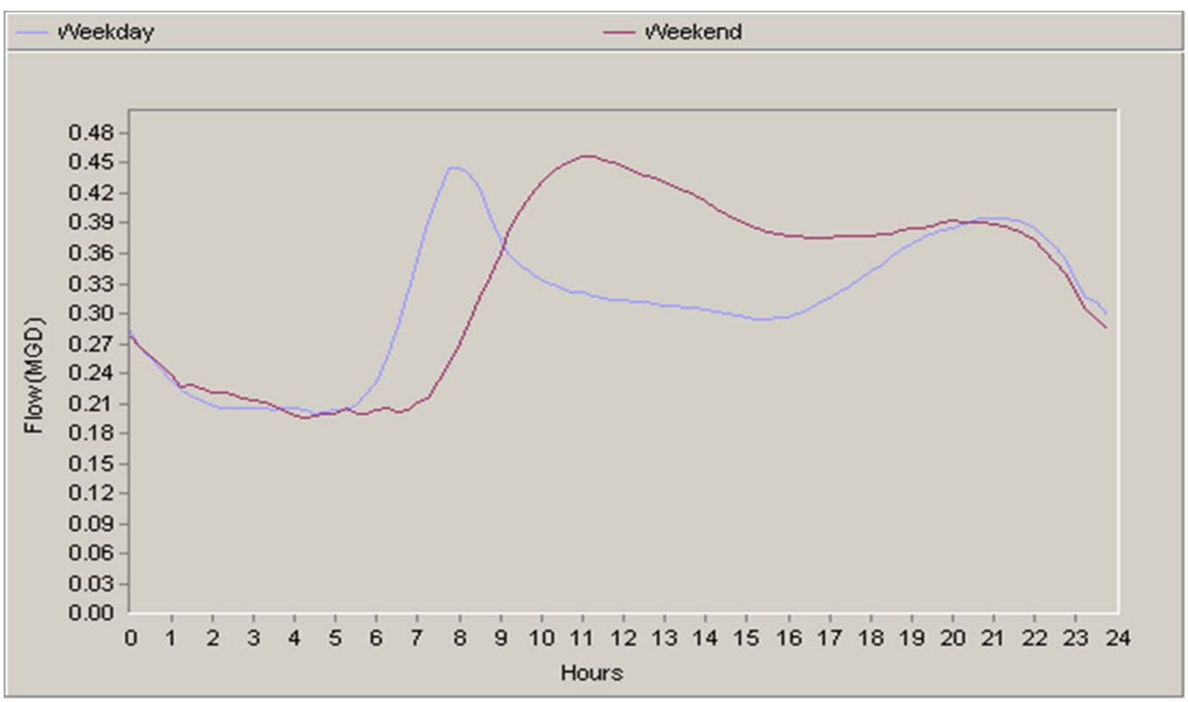
SSOAP-calculated Dry Weather Flows

This appendix contains a flow table and the hydrographs for each individual meter.
The 'Week' columns combine the weekday and weekend flow numbers with a weighted sum (2/7 * weekend + 5/7 * weekday) in order to have a single base flow statistic.

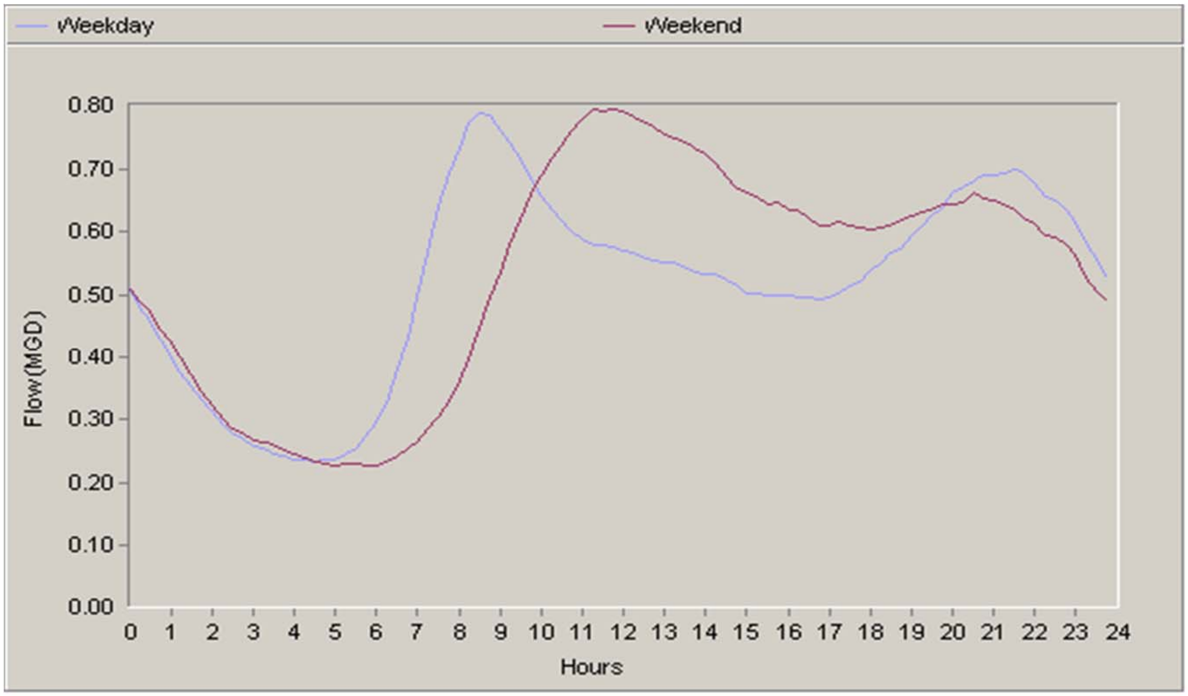
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
BC1_003	0.130	0.081	0.037	0.133	0.088	0.038	0.131	0.083	0.037
EL1_155	0.444	0.310	0.201	0.456	0.331	0.196	0.448	0.316	0.199
KR2 003/214	1.226	0.808	0.414	1.232	0.844	0.410	1.228	0.818	0.413
KR3_166	0.302	0.230	0.109	0.317	0.233	0.122	0.306	0.231	0.113
KR4_017	0.138	0.097	0.046	0.146	0.102	0.054	0.140	0.099	0.048
KR6_001	0.082	0.054	0.030	0.039	0.033	0.026	0.070	0.048	0.029
KR6_015	0.286	0.212	0.098	0.330	0.229	0.098	0.298	0.217	0.098
NL1_056	0.120	0.090	0.053	0.127	0.090	0.045	0.122	0.090	0.051
WR2_059	0.904	0.613	0.330	0.907	0.643	0.341	0.905	0.622	0.333
WR2_090	0.597	0.428	0.239	0.635	0.464	0.269	0.608	0.439	0.247
WR2_151	0.662	0.420	0.179	0.687	0.444	0.179	0.669	0.427	0.179
WR4_18A	0.969	0.767	0.408	1.151	0.858	0.462	1.021	0.793	0.423
WR3_302	0.376	0.245	0.111	0.414	0.271	0.128	0.387	0.252	0.116
YTC1_016	0.086	0.051	0.021	0.083	0.056	0.023	0.085	0.053	0.022
YTC2_016	0.345	0.170	0.053	0.347	0.193	0.058	0.346	0.176	0.054
YTC1_127	0.397	0.222	0.092	0.383	0.247	0.095	0.393	0.229	0.093
WWTP	10.810	8.675	4.817	11.810	8.671	4.863	11.096	8.674	4.830



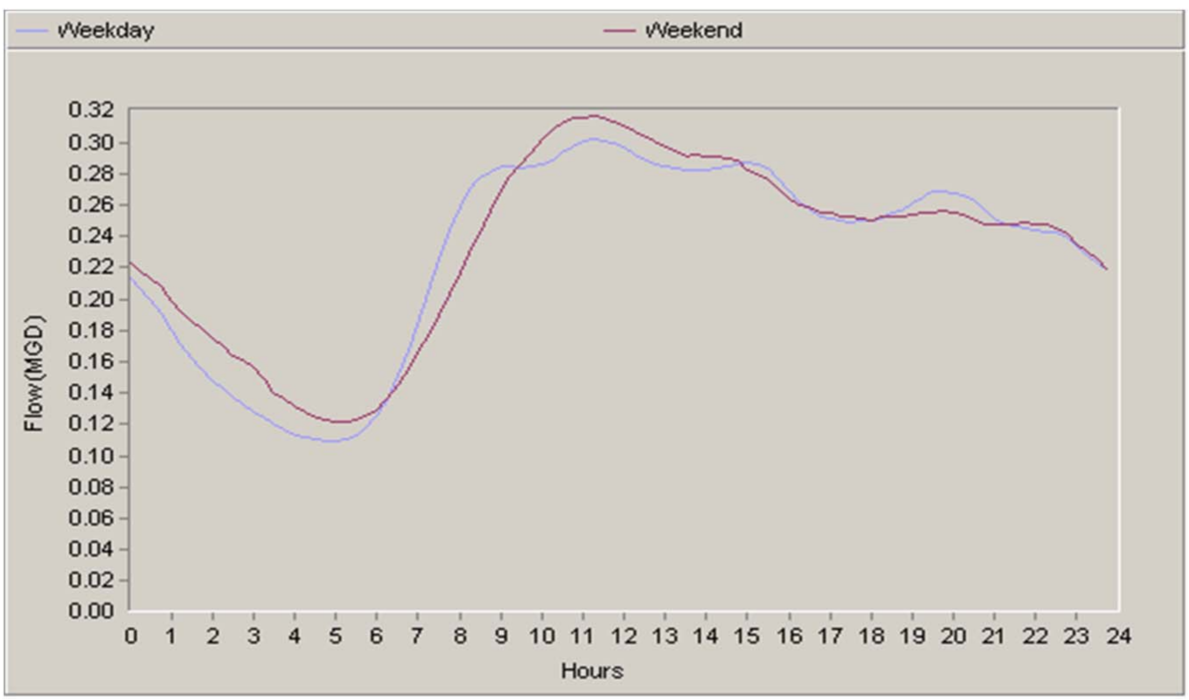
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
BC1_003	0.1297	0.0810	0.0372	0.1334	0.0881	0.0376	0.1308	0.0830	0.0373



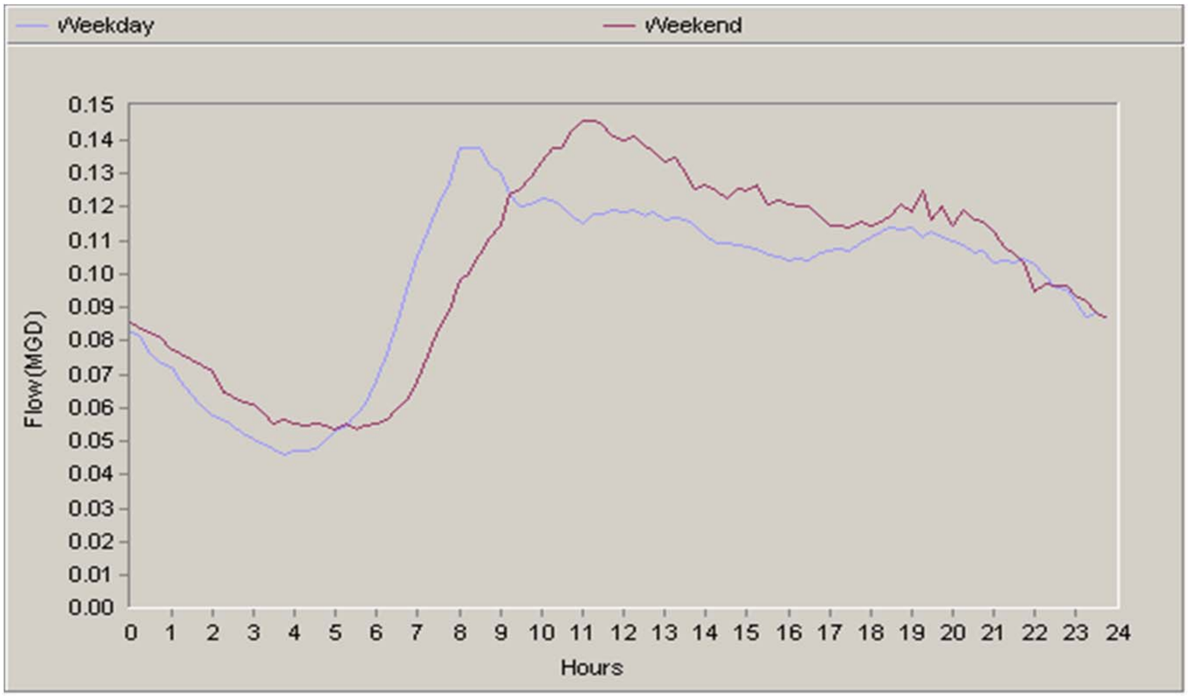
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
EL1_155	0.4442	0.3104	0.2005	0.4562	0.3311	0.1956	0.4476	0.3163	0.1991



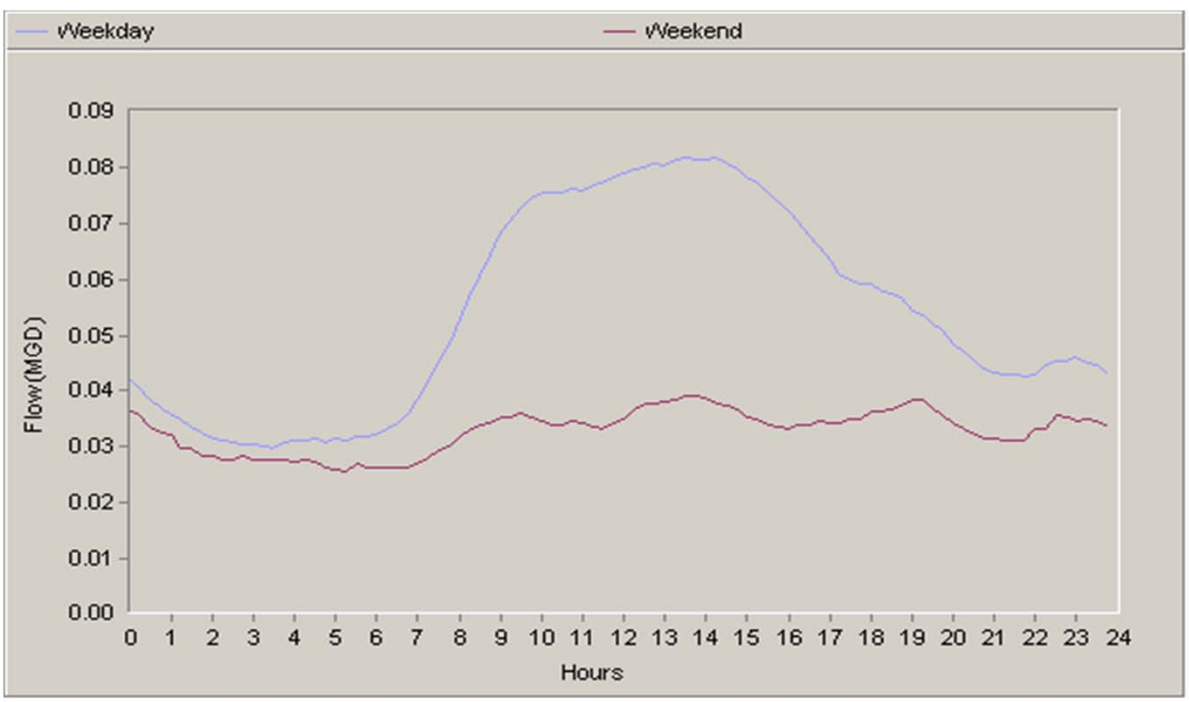
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
KR2_214	0.7894	0.5181	0.2324	0.7965	0.5302	0.2281	0.7914	0.5216	0.2312
KR2 003/214	0.7894	0.5181	0.2324	0.7965	0.5302	0.2281	0.7914	0.5216	0.2312



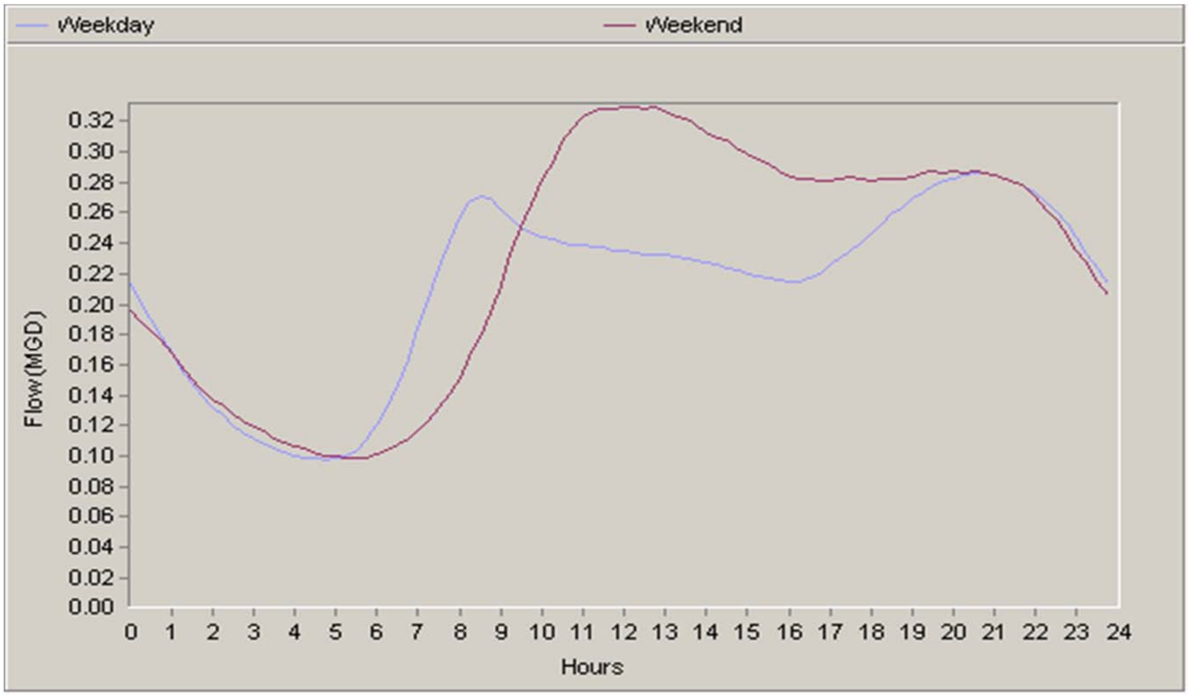
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
KR3_166	0.3022	0.2298	0.1090	0.3172	0.2330	0.1223	0.3065	0.2307	0.1128



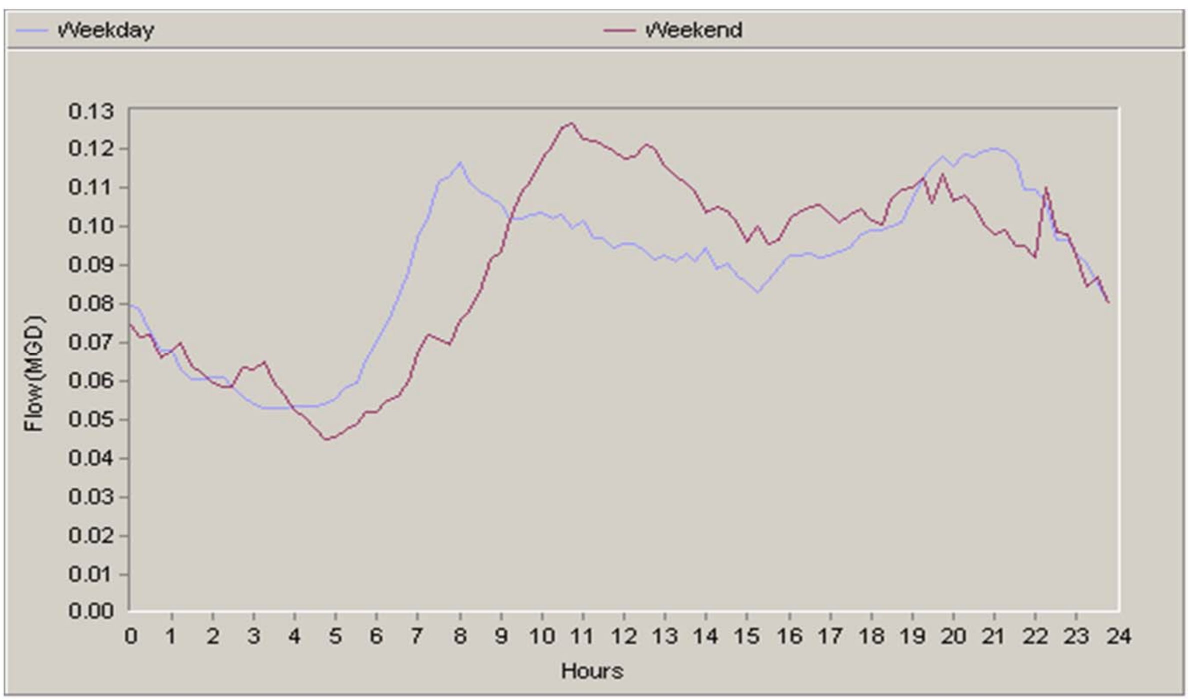
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
KR4_017	0.1378	0.0973	0.0463	0.1455	0.1015	0.0536	0.1400	0.0985	0.0484



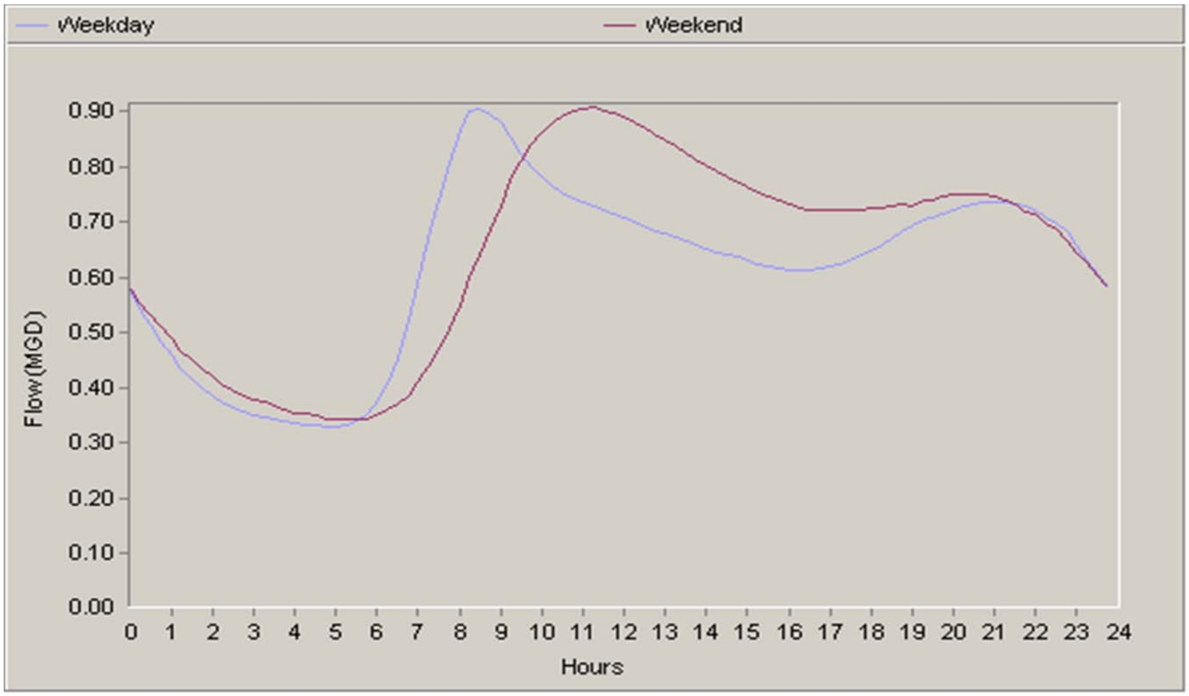
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
KR6_001	0.0820	0.0540	0.0298	0.0391	0.0328	0.0257	0.0697	0.0479	0.0286



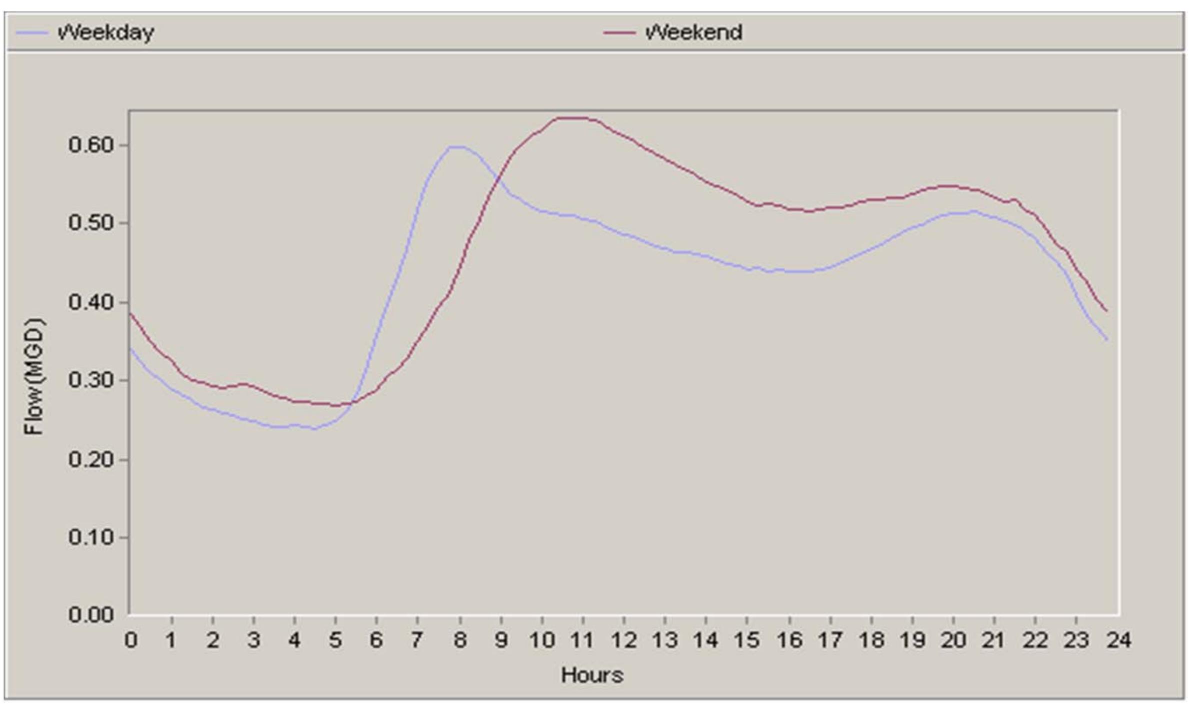
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
KR6_015	0.2858	0.2116	0.0977	0.3300	0.2288	0.0983	0.2984	0.2165	0.0979



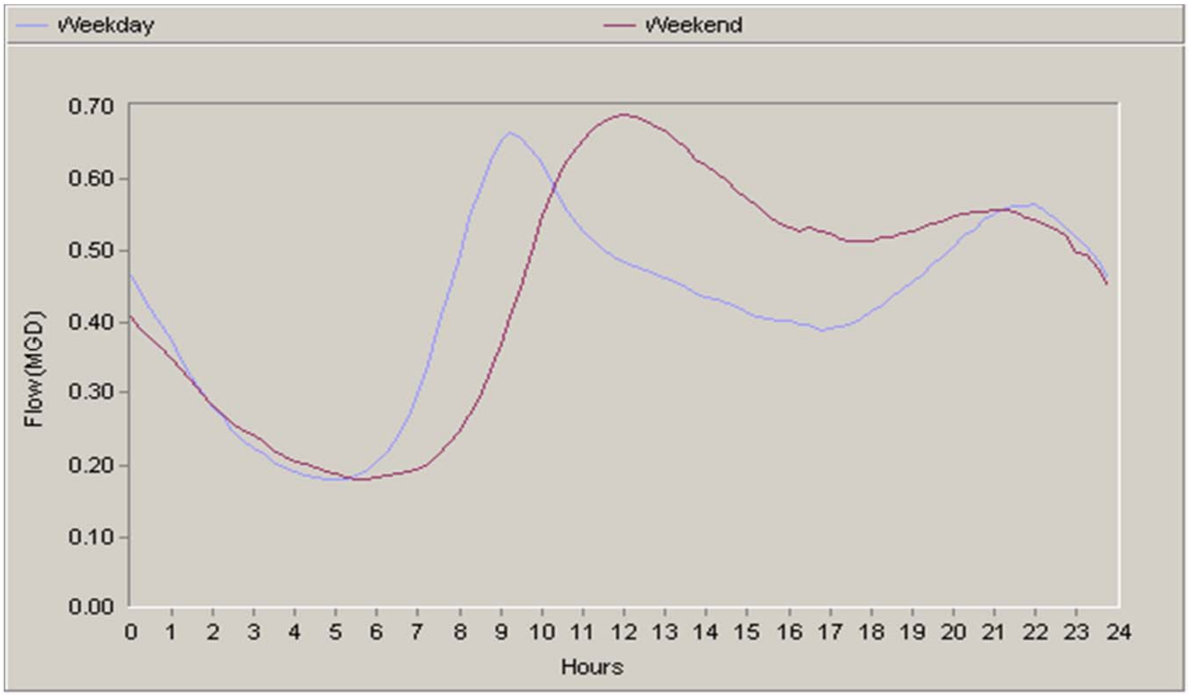
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
NL1_056	0.1200	0.0896	0.0529	0.1270	0.0898	0.0451	0.1220	0.0897	0.0507



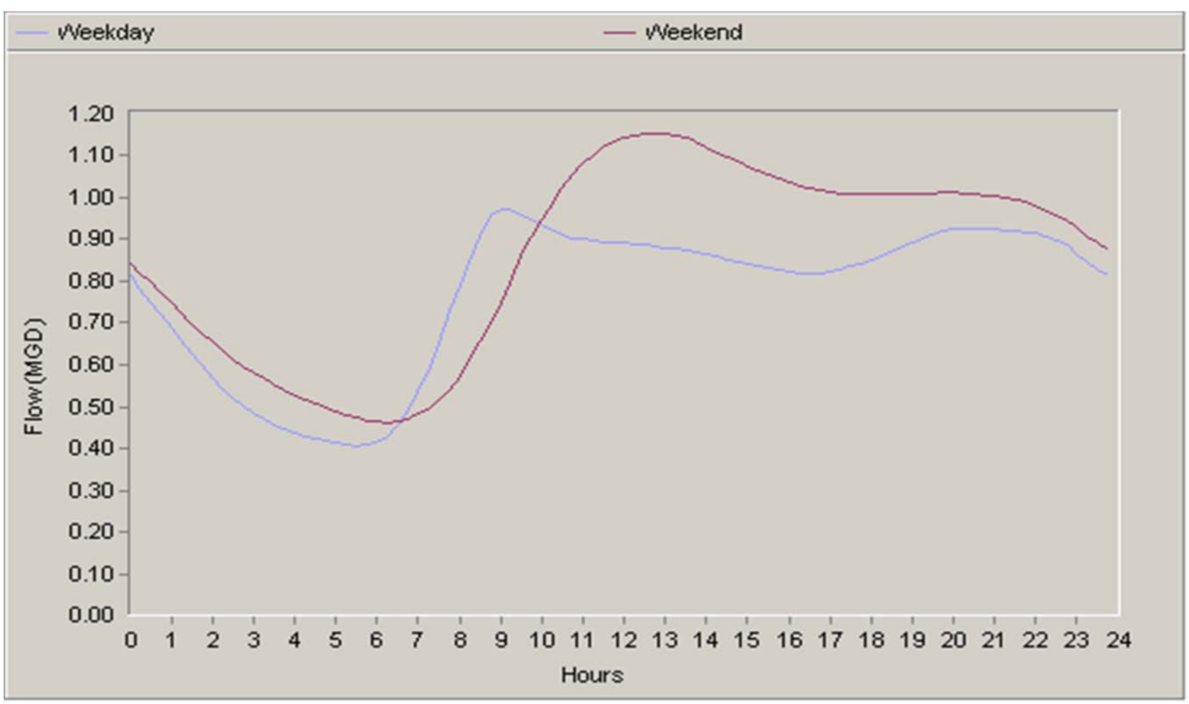
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
WR2_059	0.9038	0.6132	0.3302	0.9070	0.6430	0.3410	0.9047	0.6217	0.3333



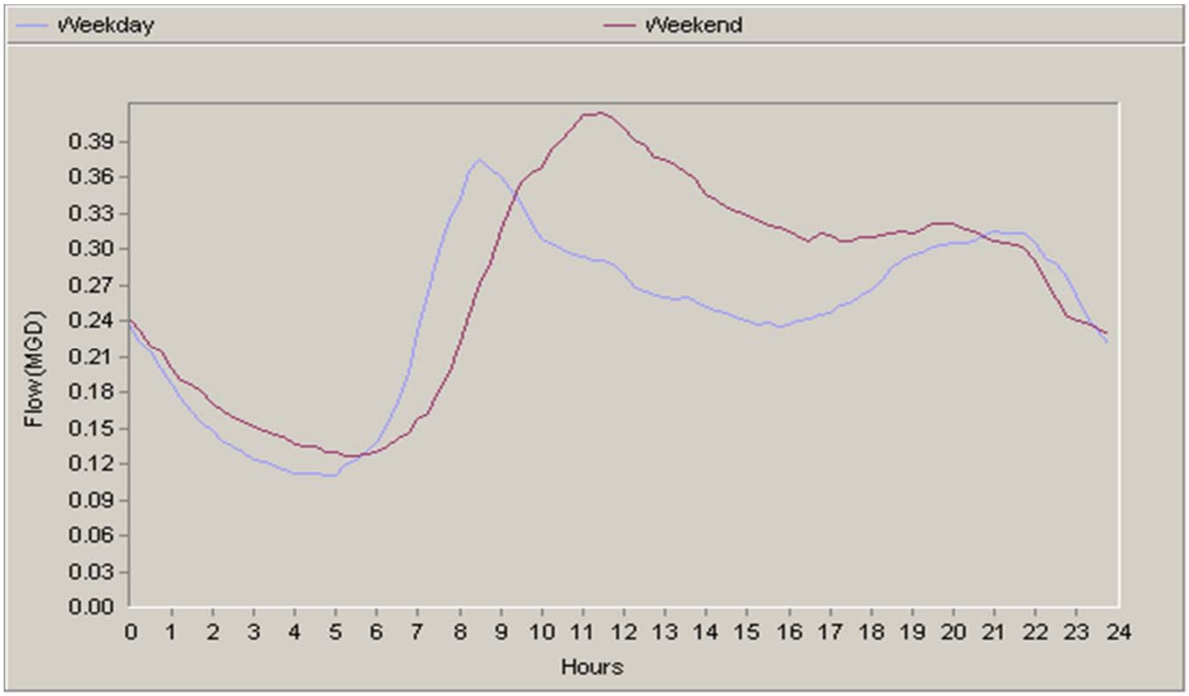
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
WR2_090	0.5973	0.4283	0.2385	0.6347	0.4643	0.2691	0.6080	0.4386	0.2472



Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
WR2_151	0.6624	0.4199	0.1794	0.6871	0.4435	0.1793	0.6695	0.4266	0.1794

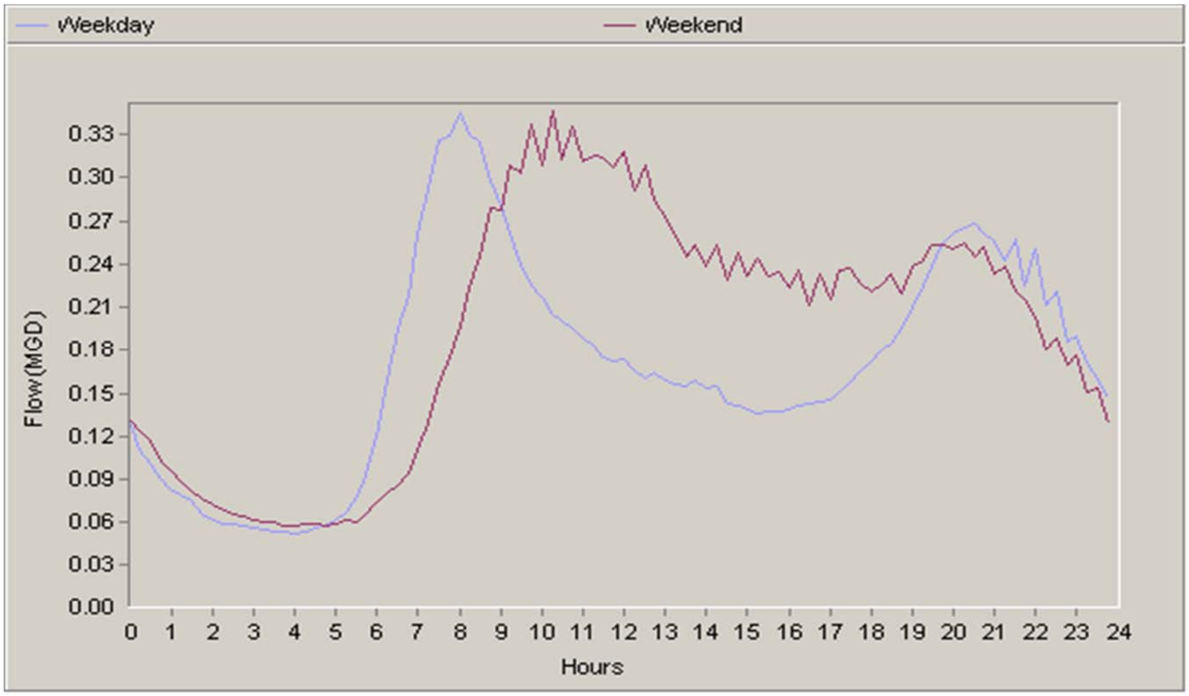


Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
WR4_18A	0.9692	0.7674	0.4077	1.1511	0.8583	0.4621	1.0212	0.7934	0.4232

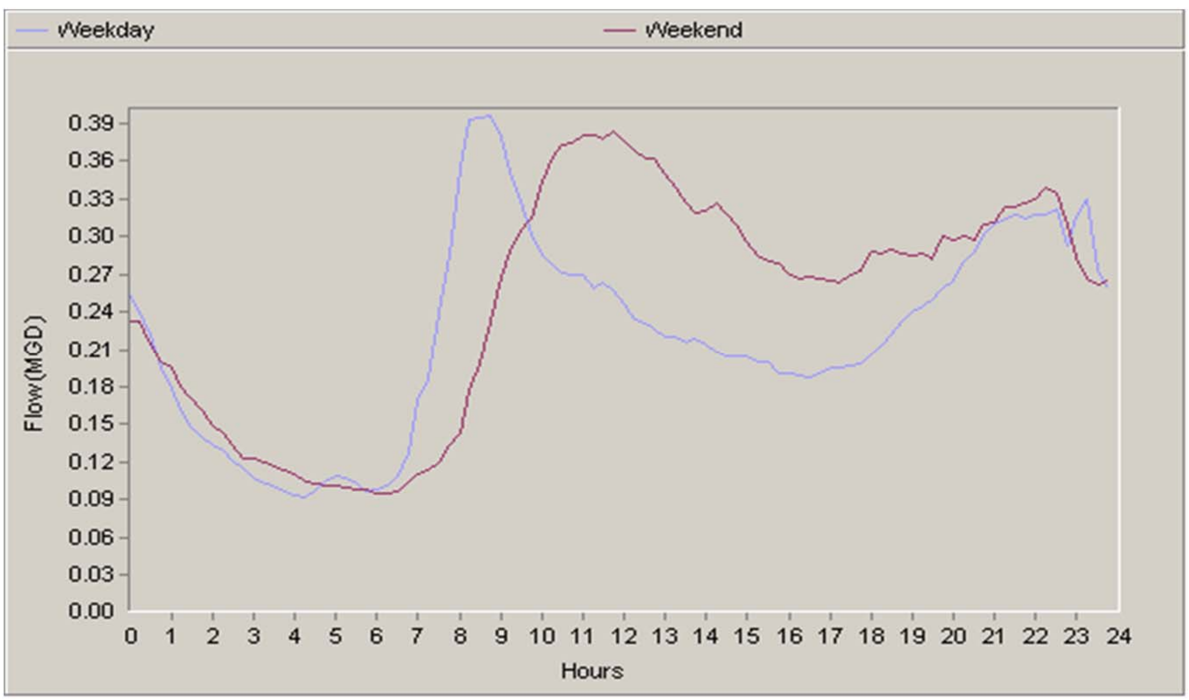


Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
WR3_302	0.3761	0.2445	0.1111	0.4144	0.2708	0.1278	0.3870	0.2520	0.1159

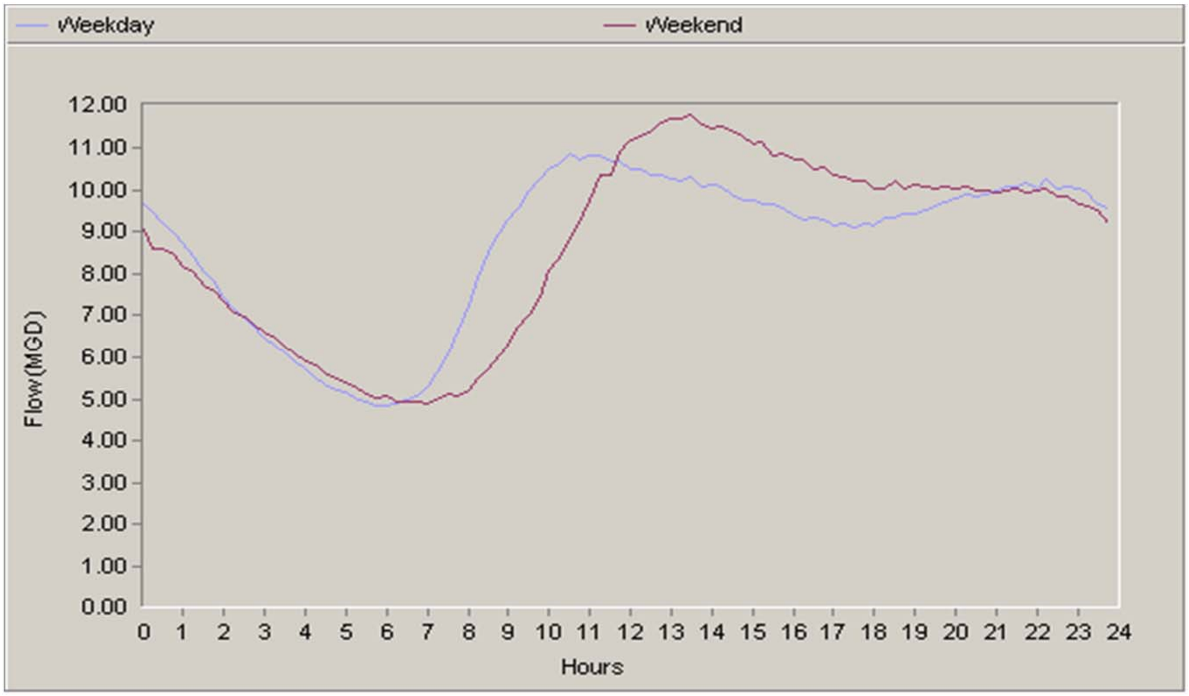
Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
YTC1_016	0.0861	0.0512	0.0210	0.0832	0.0563	0.0230	0.0853	0.0527	0.0216



Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
YTC2_016	0.3453	0.1698	0.0525	0.3468	0.1932	0.0579	0.3457	0.1765	0.0540



Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
YTC1_127	0.3967	0.2216	0.0920	0.3833	0.2472	0.0950	0.3929	0.2289	0.0929



Flow Meter	Weekday			Weekend			Week		
	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)	Max (mgd)	Ave (mgd)	Min (mgd)
WWTP_all	10.8101	8.6751	4.8168	11.8098	8.6714	4.8627	11.0957	8.6740	4.8299

Appendix 2-B
Peaking Factors

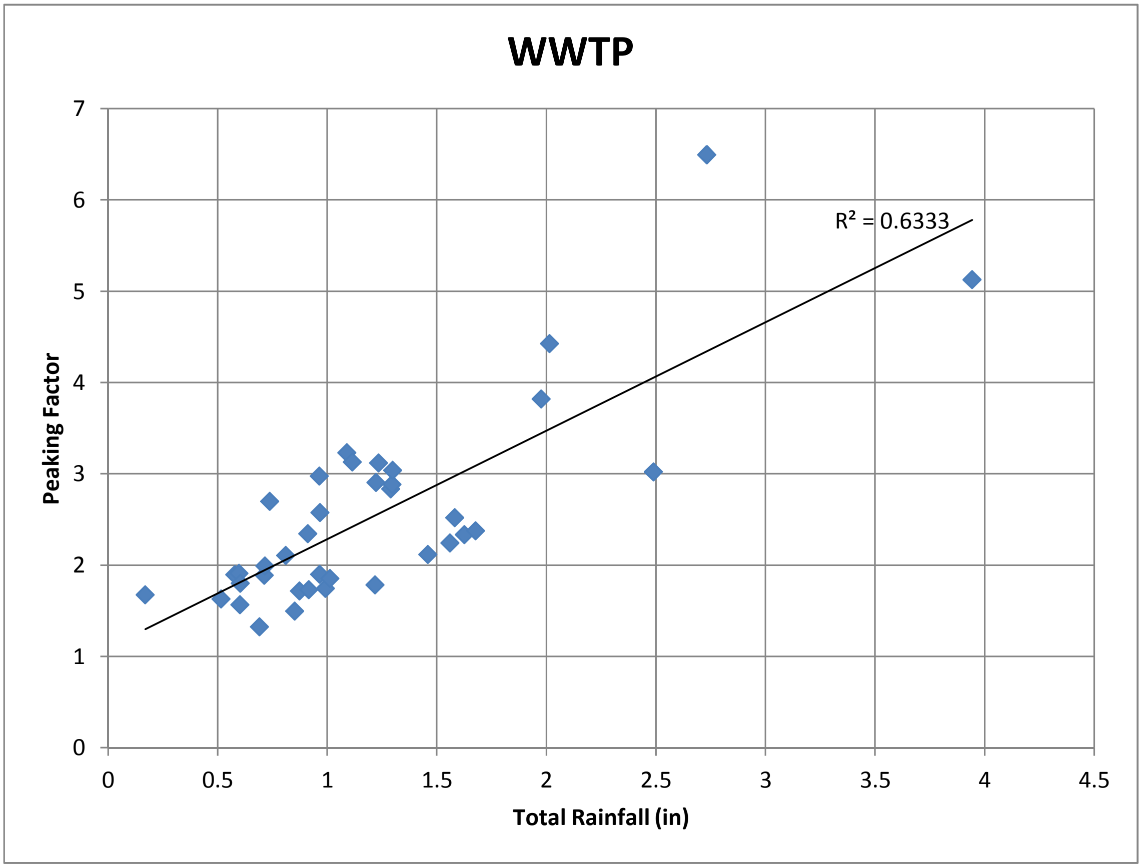
APPENDIX 2-B

Peaking Factors (PF)

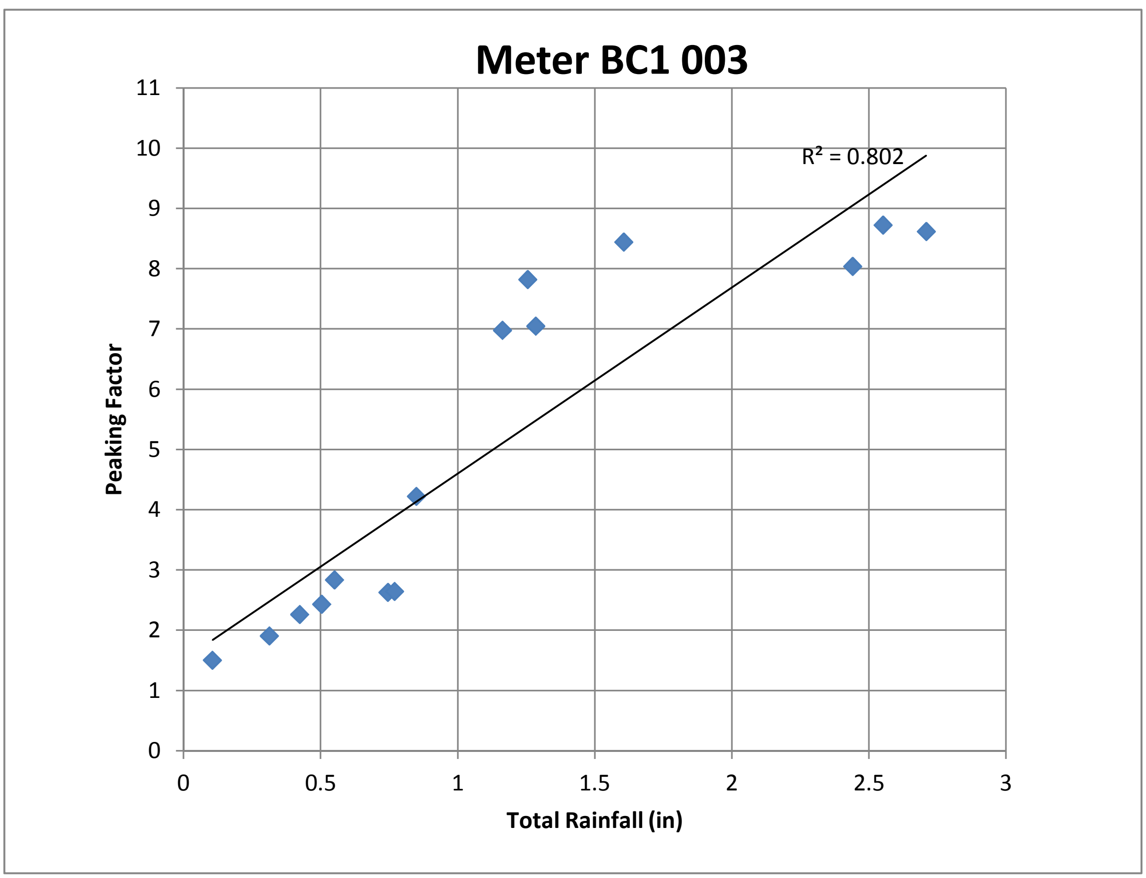
This Appendix contains all of the scatterplots and lines of regression for the flow meters analyzed for this memo. A summary from these plots is shown in the table below.

Flow Meter	PF 2-inch Storm	PF 1-inch Storm	2in/1in Ratio	Average Storm Duration (hrs)	Coefficient of Determination (R^2)
WWTP	3.5	2.3	1.5	8.82	0.633
BC1 003	7.6	4.6	1.7	6.20	0.802
EL1 155	3.9	3.0	1.3	8.00	0.732
KR2 003/214	6.5	4	1.4	7.71	0.935
KR3 166	4.9	2.7	1.8	7.58	0.930
KR4 017	11.3	5.3	2.1	5.27	0.839
KR6 001	7.7	4.0	1.9	7.78	0.877
KR6 015	10.5	5.5	1.9	7.95	0.952
NL1 056	8.8	5.8	1.5	6.74	0.808
WR2 059	4.4	2.6	1.7	6.97	0.946
WR2 090	4.4	3.4	1.3	6.00	0.883
WR2 151	5.7	3.4	1.7	9.04	0.826
WR4 18A	5.1	2.8	1.8	7.64	0.936
WR4 302	6.7	4.0	1.7	8.85	0.888
YTC1 016	6.2	4.0	1.6	6.14	0.808
YTC2 016	5.4	3.3	1.6	6.97	0.770
YTC2 127	6.5	5.0	1.3	5.67	0.646

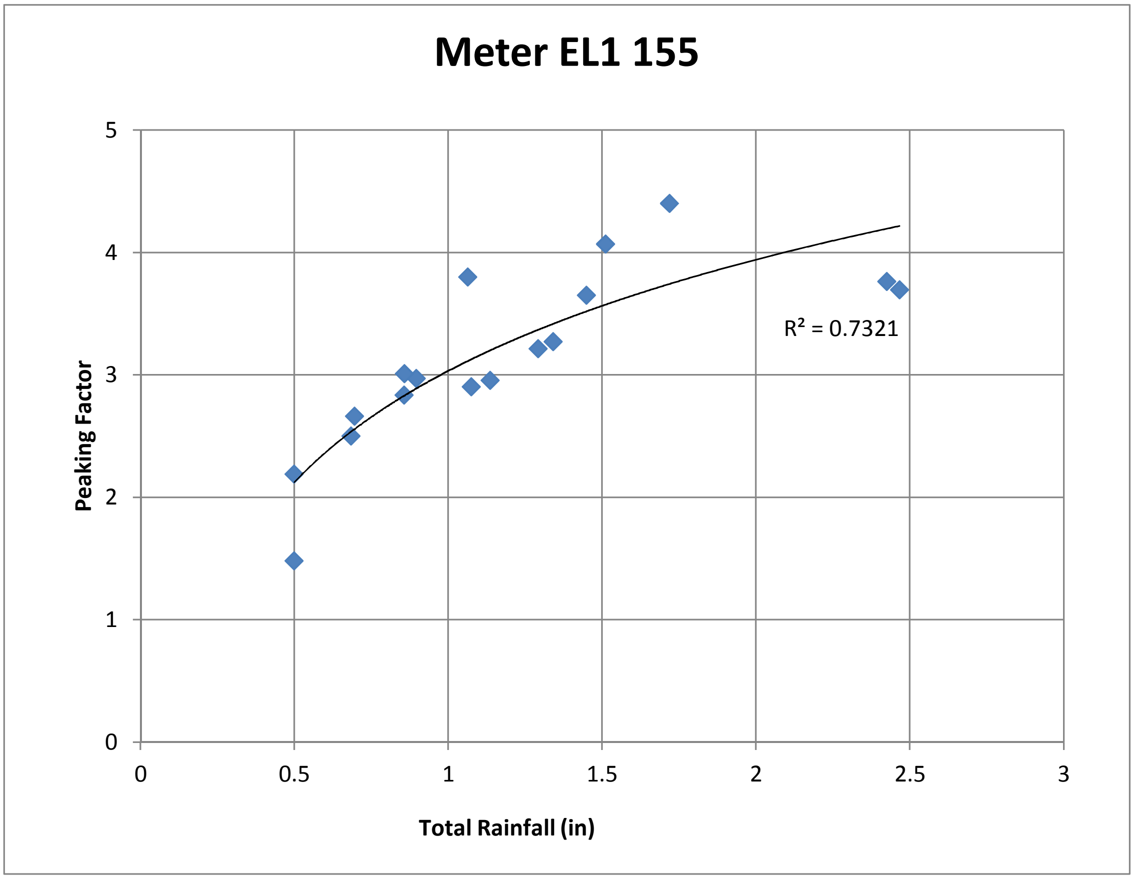
AVERAGE	6.42	3.87	1.64	7.25	0.836
MAX	11.30	5.75	2.13	9.04	0.952
MIN	3.50	2.30	1.29	5.27	0.633

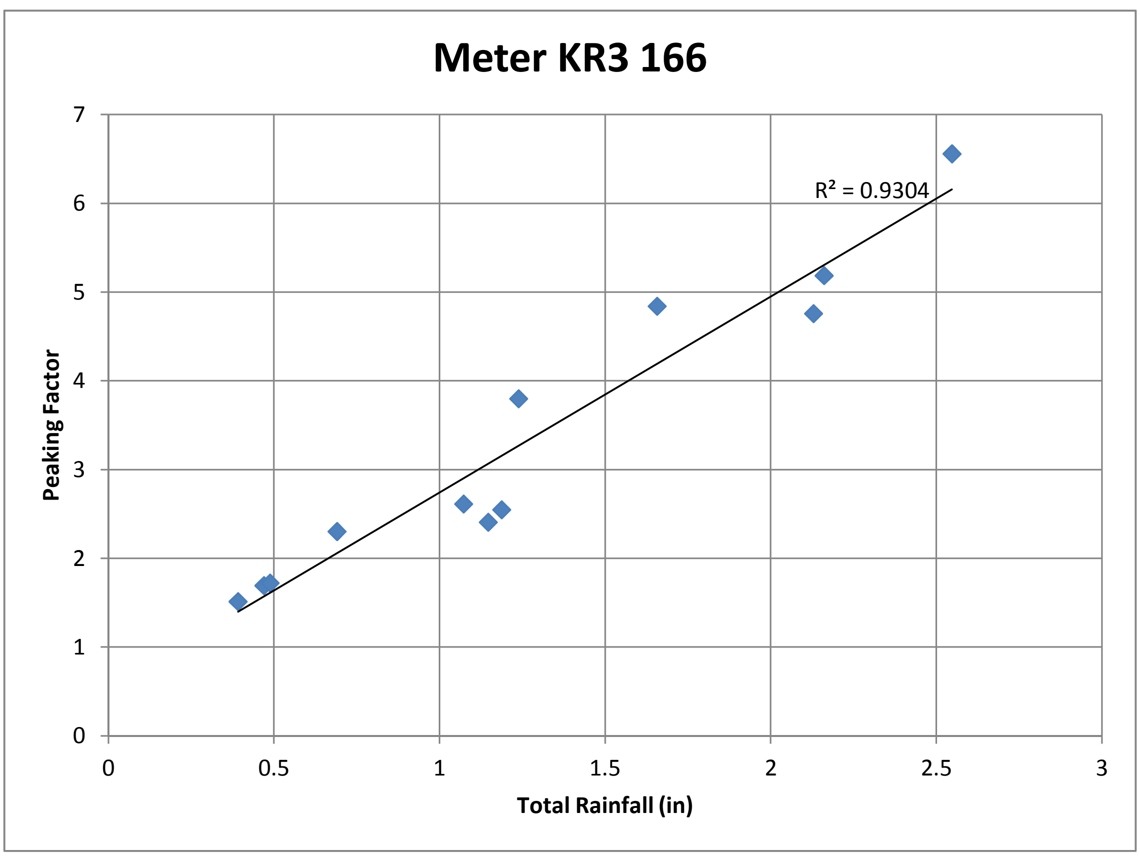


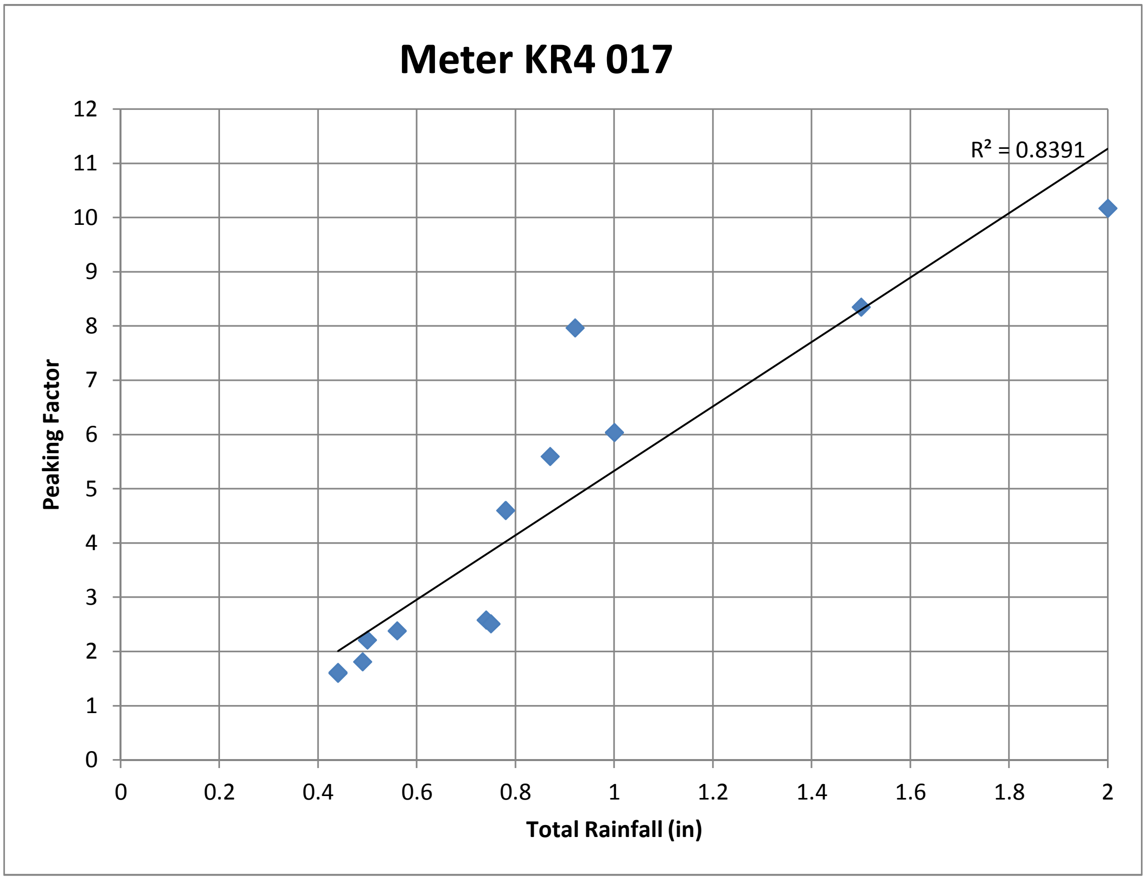
WWTP
R² = 0.6333
Peaking Factor
Total Rainfall (in)
0
1
2
3
4
5
6
7
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5



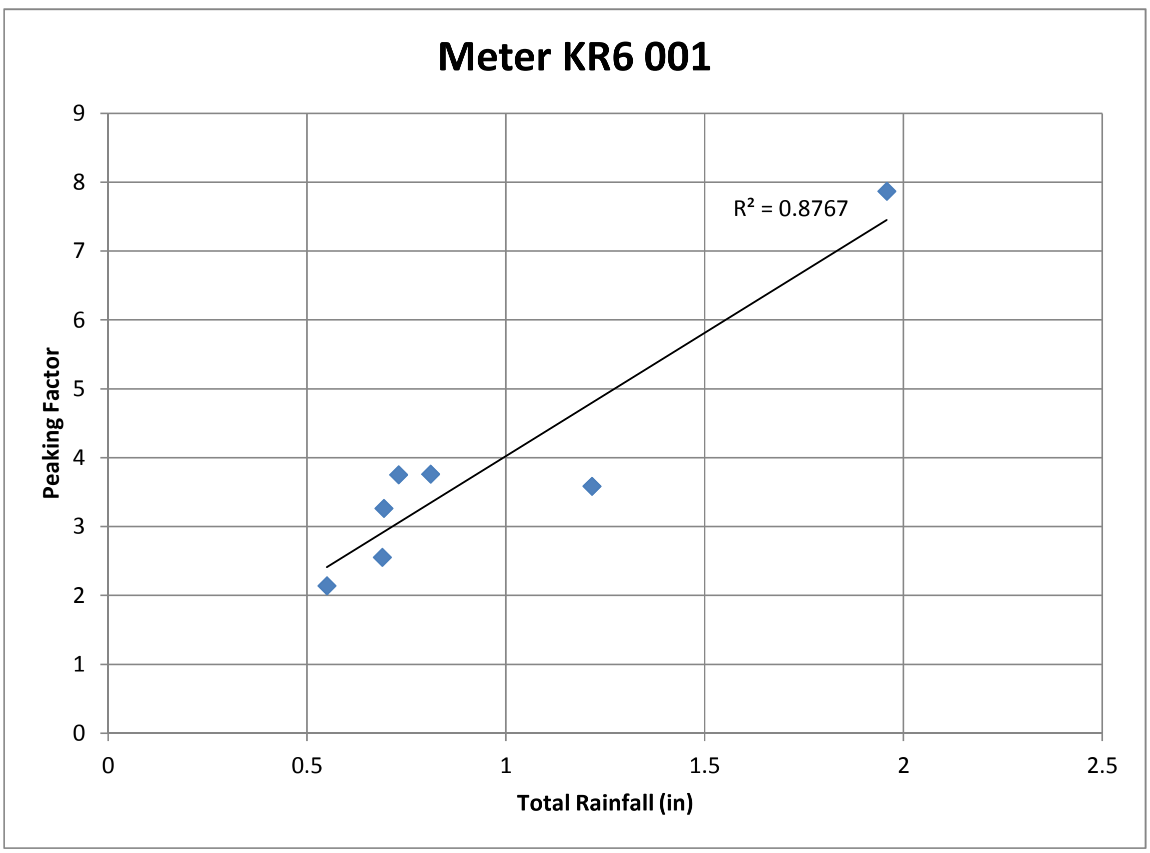
Meter BC1 003
R² = 0.802
Peaking Factor
Total Rainfall (in)
0
1
2
3
4
5
6
7
8
9
10
11
0
0.5
1
1.5
2
2.5
3


Meter EL1 155
Peaking Factor
Total Rainfall (in)
R² = 0.7321
0
1
2
3
4
5
0
0.5
1
1.5
2
2.5
3

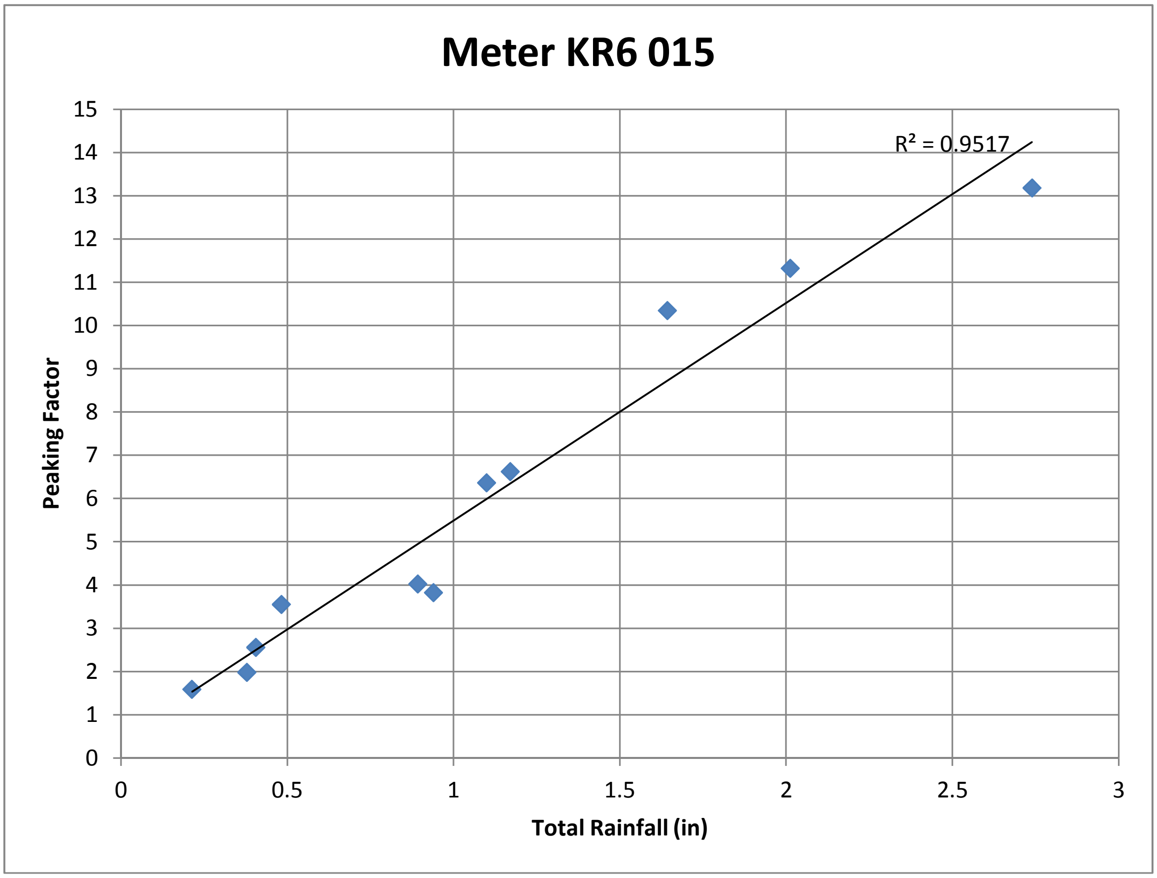

Meter KR3 166
Peaking Factor
Total Rainfall (in)
R² = 0.9304
0
1
2
3
4
5
6
7
0
0.5
1
1.5
2
2.5
3



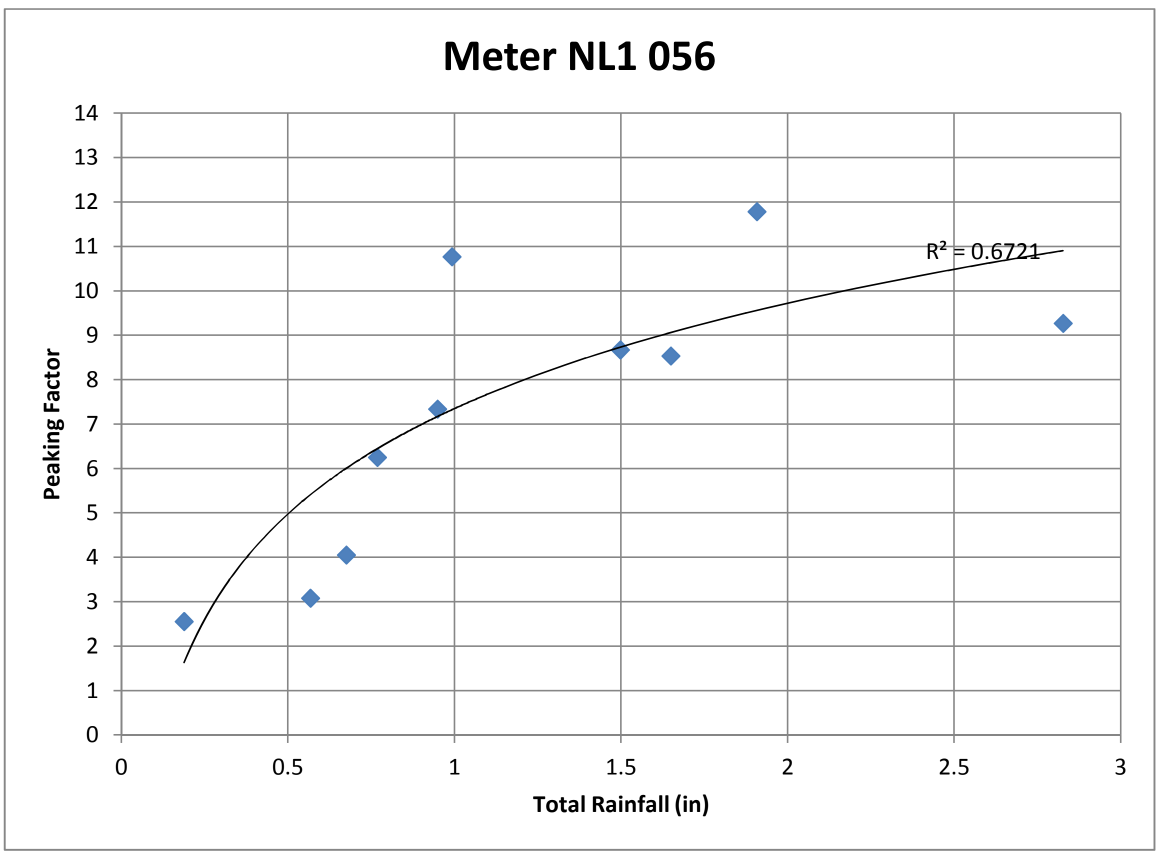
Meter KR4 017
Peaking Factor
Total Rainfall (in)
R² = 0.8391
0
1
2
3
4
5
6
7
8
9
10
11
12
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2



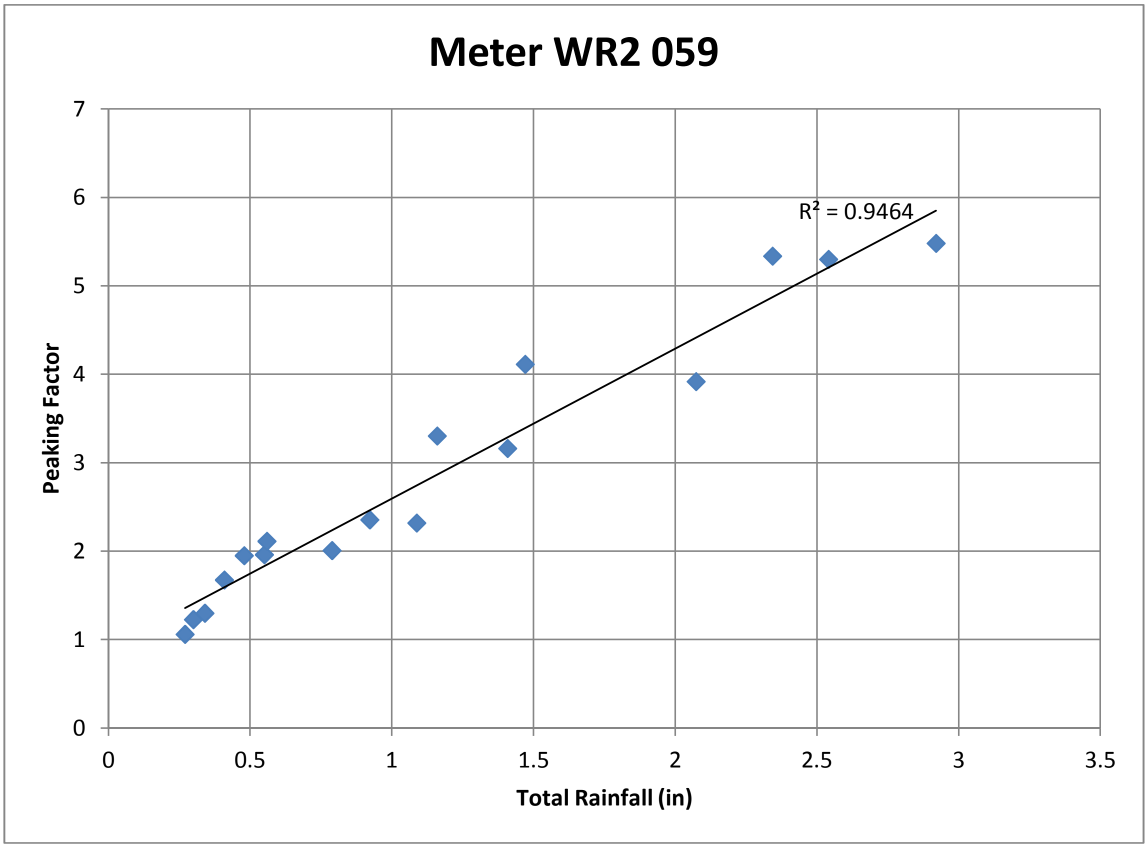
Meter KR6 001
Peaking Factor
Total Rainfall (in)
R² = 0.8767
0
1
2
3
4
5
6
7
8
9
0
0.5
1
1.5
2
2.5

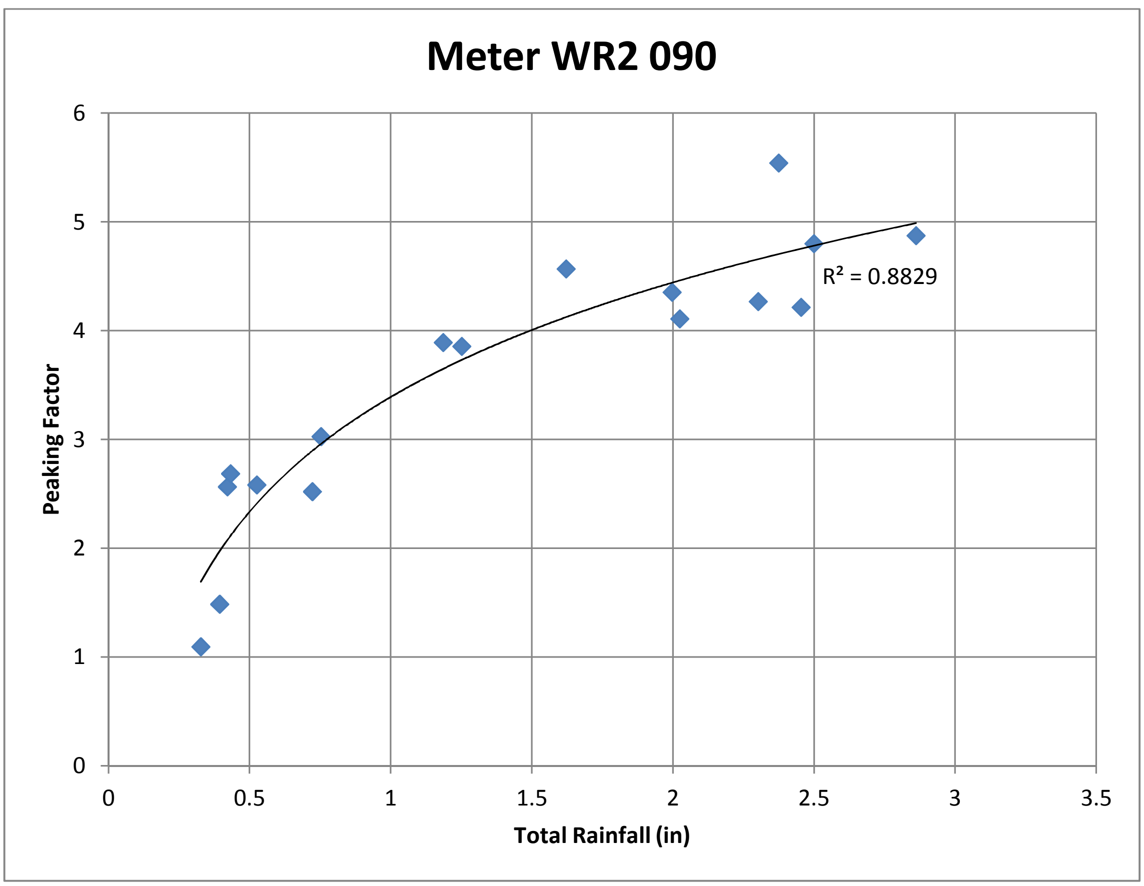


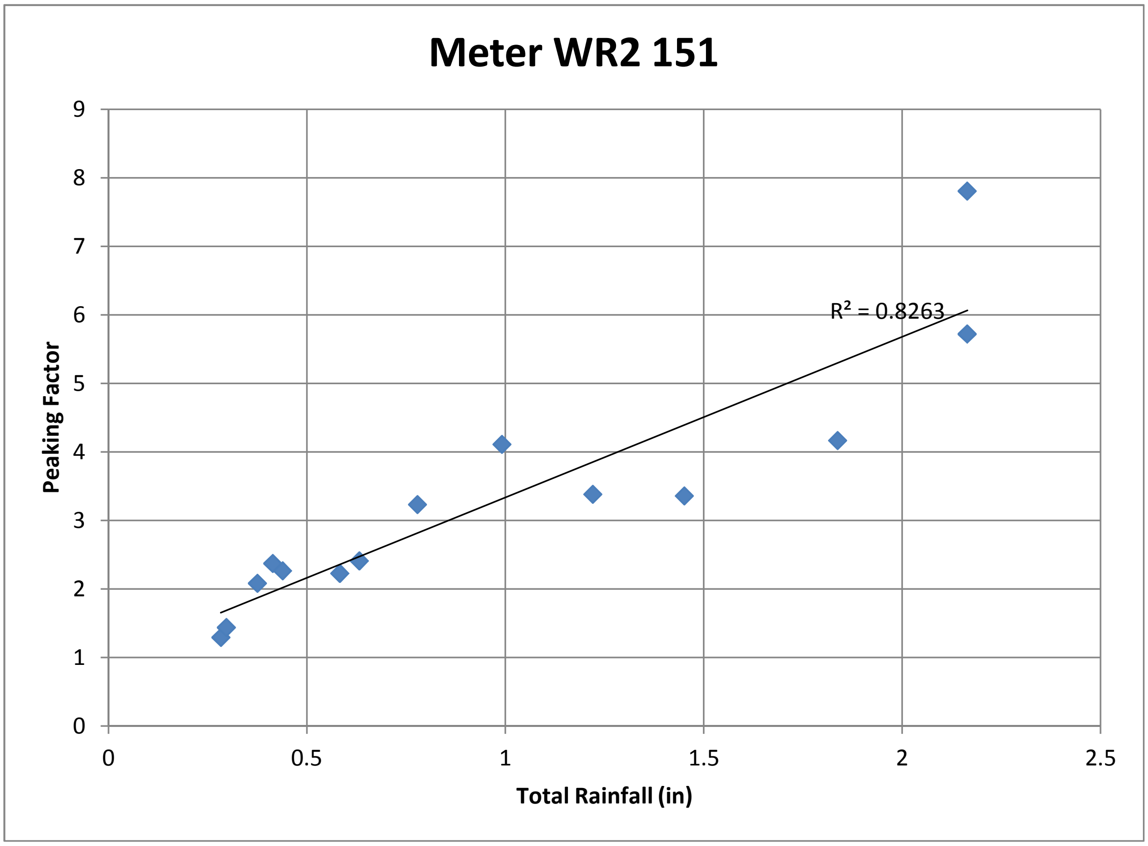
Meter KR6 015
R² = 0.9517
Peaking Factor
Total Rainfall (in)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
0
0.5
1
1.5
2
2.5
3

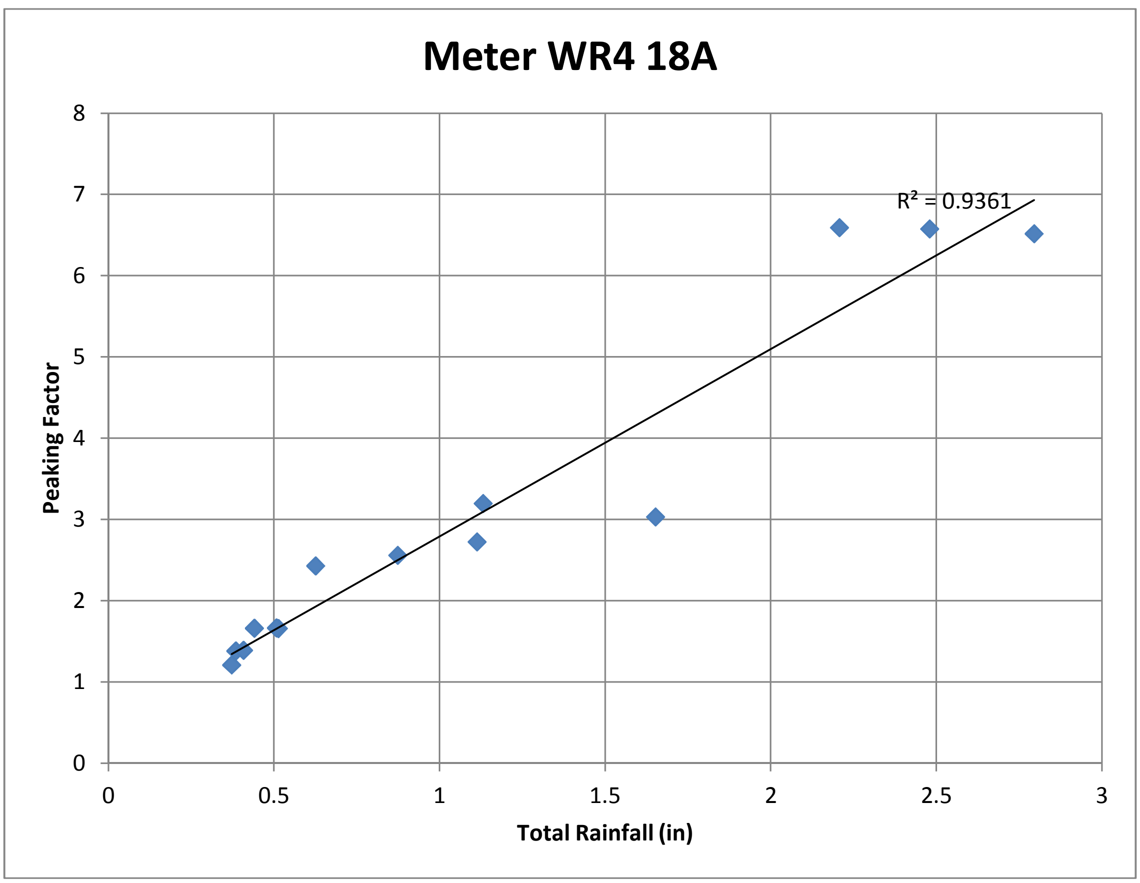


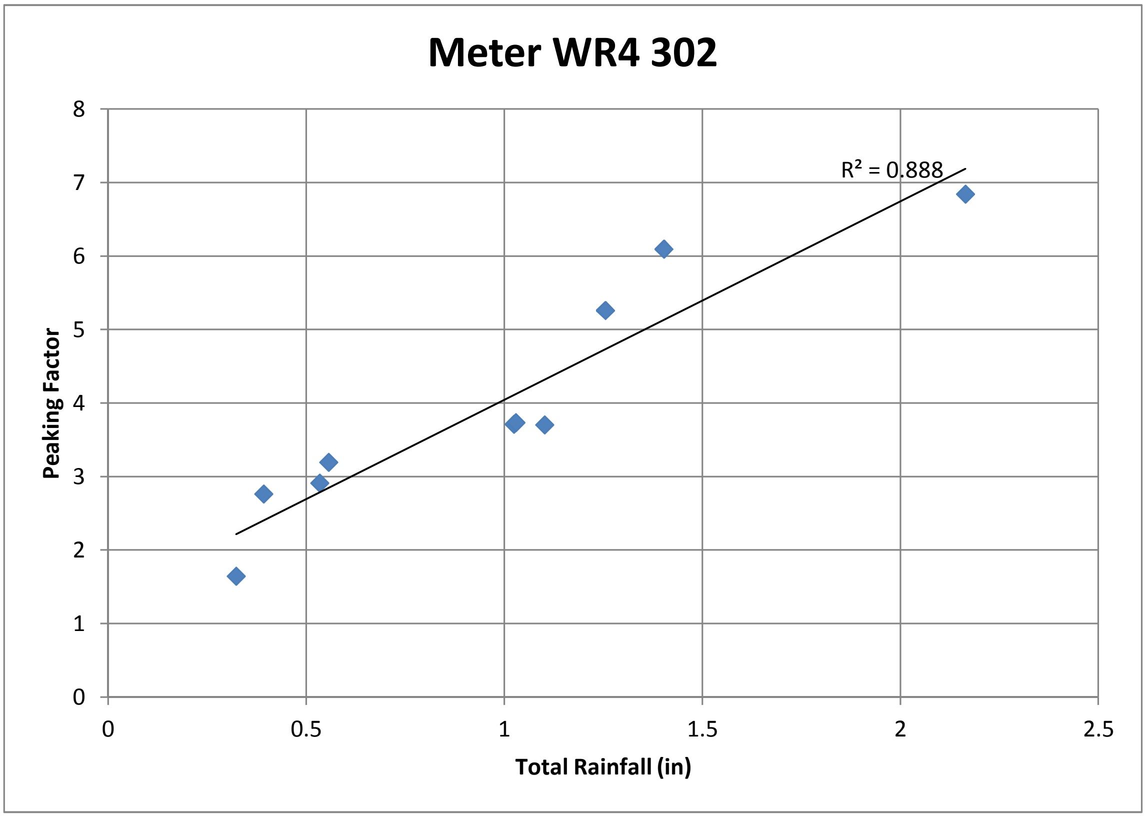
Meter NL1 056
R² = 0.6721
Peaking Factor
Total Rainfall (in)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
0
0.5
1
1.5
2
2.5
3

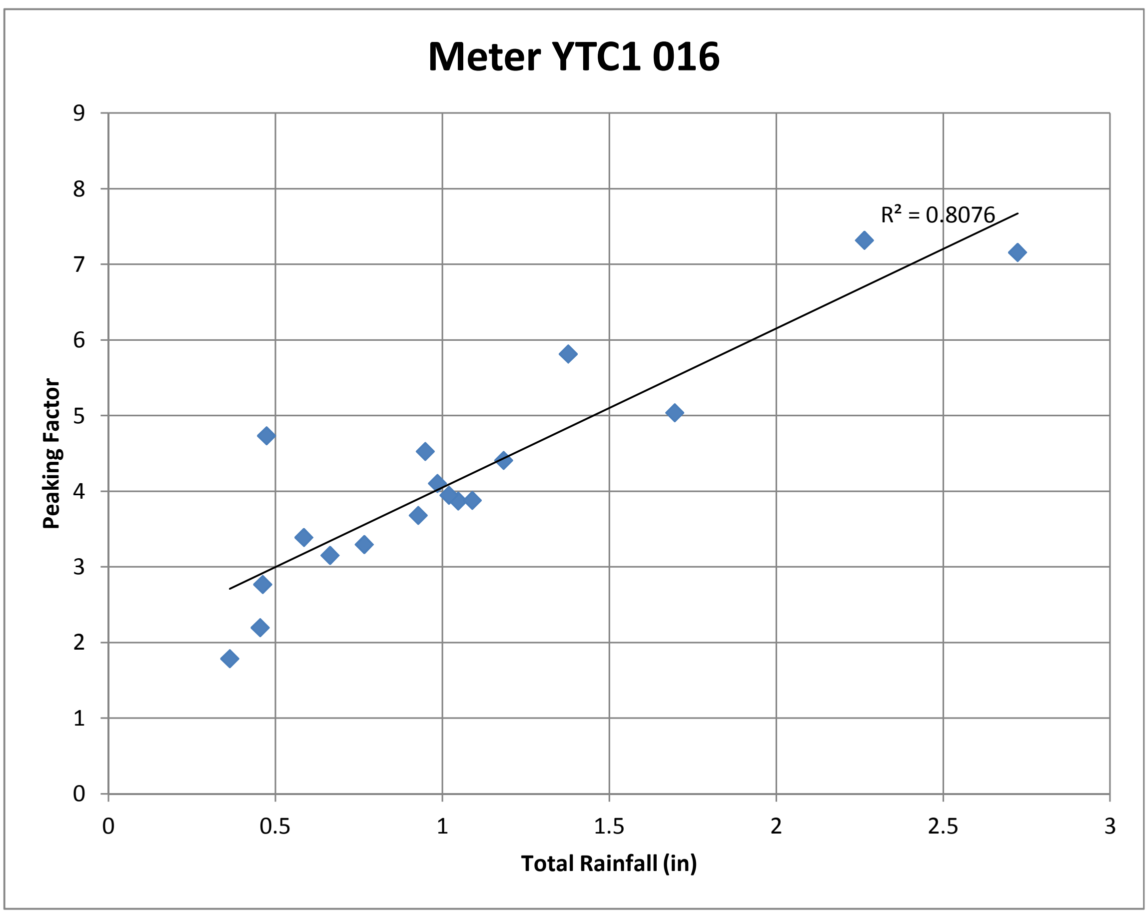

Meter WR2 059
R² = 0.9464
Peaking Factor
Total Rainfall (in)
0
1
2
3
4
5
6
7
0
0.5
1
1.5
2
2.5
3
3.5

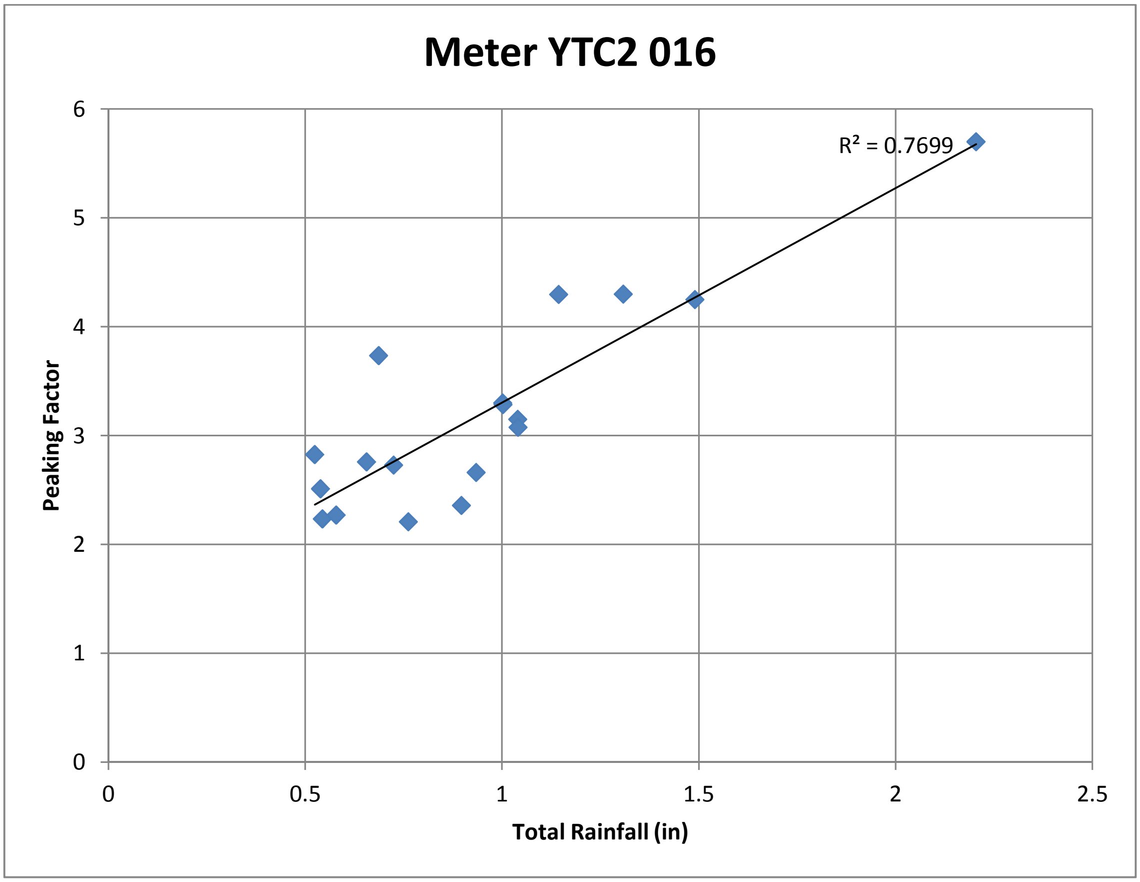

Meter WR2 090
R² = 0.8829
Peaking Factor
Total Rainfall (in)
0
1
2
3
4
5
6
0
0.5
1
1.5
2
2.5
3
3.5

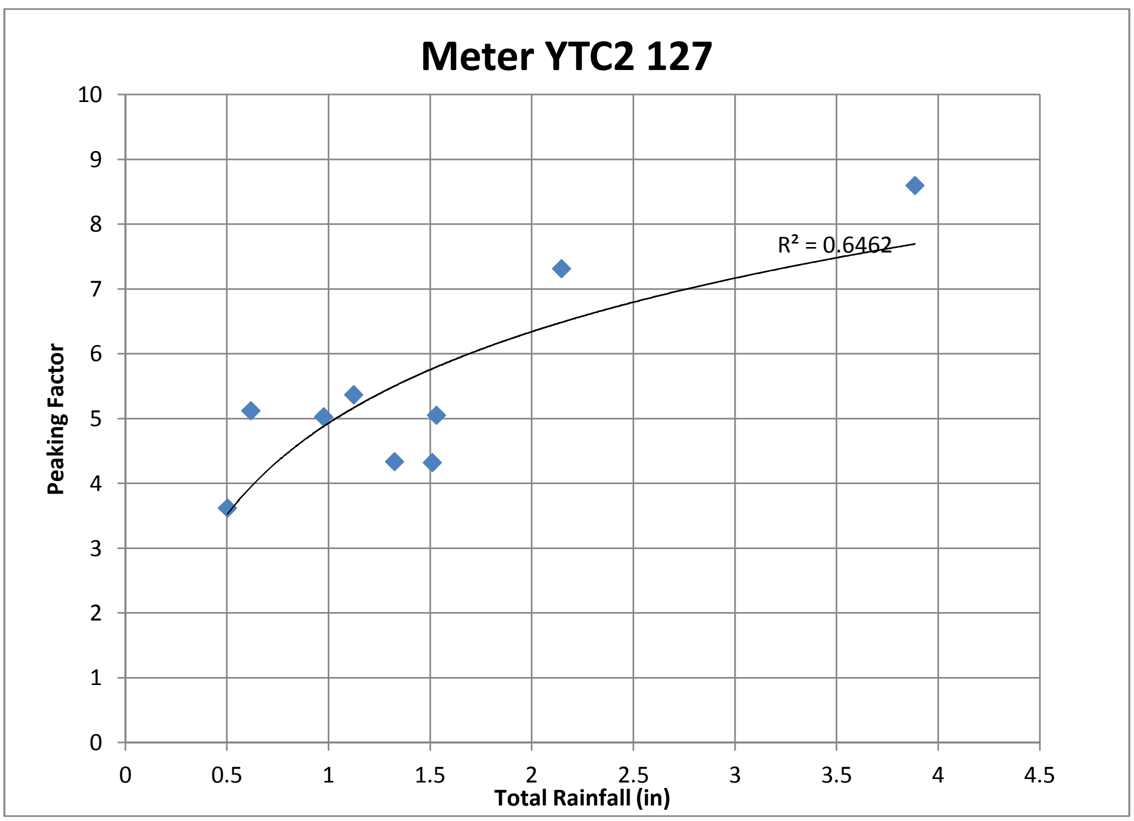

Meter WR2 151
Peaking Factor
Total Rainfall (in)
R² = 0.8263
0
1
2
3
4
5
6
7
8
9
0
0.5
1
1.5
2
2.5


Meter WR4 18A
Peaking Factor
Total Rainfall (in)
R² = 0.9361
0
1
2
3
4
5
6
7
8
0
0.5
1
1.5
2
2.5
3


Meter WR4 302
Peaking Factor
Total Rainfall (in)
R² = 0.888
0
1
2
3
4
5
6
7
8
0
0.5
1
1.5
2
2.5


Meter YTC1 016
Peaking Factor
Total Rainfall (in)
R² = 0.8076
0
1
2
3
4
5
6
7
8
9
0
0.5
1
1.5
2
2.5
3


Meter YTC2 016
R² = 0.7699
Peaking Factor
0
1
2
3
4
5
6
0
0.5
1
1.5
2
2.5
Total Rainfall (in)


Meter YTC2 127
R² = 0.6462
Peaking Factor
0
1
2
3
4
5
6
7
8
9
10
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
Total Rainfall (in)

Appendix 2-C
Model Plot and Data Summary

Model Plot and Data Summary

Wastewater Facilities Master Plan

for

Lawrence, Kansas

City of Lawrence, Kansas

BMcD Project No. 54793

City P.O. 07629

Burns & McDonnell Engineering Company, Inc.
9400 Ward Parkway
Kansas City, MO 64114

A. Submittal Summary

This model plot and data summary is submitted in partial fulfillment of the Lawrence, Kansas Wastewater Facilities Master Plan. Included in this summary are the following:

- Discussion of the SewerGEMS software option selected for this model.
- Electronic GIS map-document file and modifications to GIS files.
- Electronic data summary spreadsheet file.
- Model validation procedure and outcome.

B. Selected SewerGEMS Software Option

The software used for modeling the wastewater collection system is Bentley Systems SewerGEMS version 8i (SELECT Series 20, version 12/13/2010 [08.11.02.49]. This version includes two programs called SewerGEMS and SewerGEMS Sanitary. The two programs have various advantages and disadvantages. They were evaluated based on the conditions that exist in the City's wastewater collection system, and considering which of the two programs is most suitable for modeling those conditions and for meeting the objectives of the master plan. The evaluation included consultations with Bentley Systems technical support. Based on this evaluation, SewerGEMS was selected for this model. The significant factors leading to this decision are summarized below.

- SewerGEMS is able to model the hydrology of rainfall events in a more realistic manner than SewerGEMS Sanitary which is considered to be an important attribute of the model.
- SewerGEMS is able to model situations where flow is split among two or more sewers exiting from a single manhole. There are approximately 100 locations where this occurs in the City's wastewater collection systems, so this becomes another important attribute of the model. SewerGEMS Sanitary requires the user to provide a "rating curve" to essentially tell the model how flows are to be split among multiple exiting sewers rather than the model software analyzing the split of flows.

C. Electronic GIS Map-Document File and GIS Modifications

An electronic copy in two versions (.mxd and PDF) of the GIS map-document file is included with this summary. They were created directly from the model and reflect various and necessary modifications made to the City's database files when the model was created. The current model is based on the GIS update from the City dated 10/22/2010. This GIS update has been edited and all edits have been documented. The documentation of edits is provided in Appendix A to this summary. The editing is intended to correct various types of conditions found in the City's files such as:

- Duplicate facility ID's.
- Locations where connectivity was absent.
- Locations where elevations were clearly in error due to their magnitude (such as 100 feet).
- Locations where a large pipe flows into a very small pipe (such as a 21" sewer into a 6" sewer).
- Locations where there are significant differences in upstream and downstream pipe inverts.
- In GIS, pump stations are represented by a single point. For modeling purposes, it is necessary to show individual pumps in a pump station as well as the pump station wet well as separate nodes, with links or "virtual pipes" connecting them to the system. An example of this situation is shown below in Figures 1 and 2.

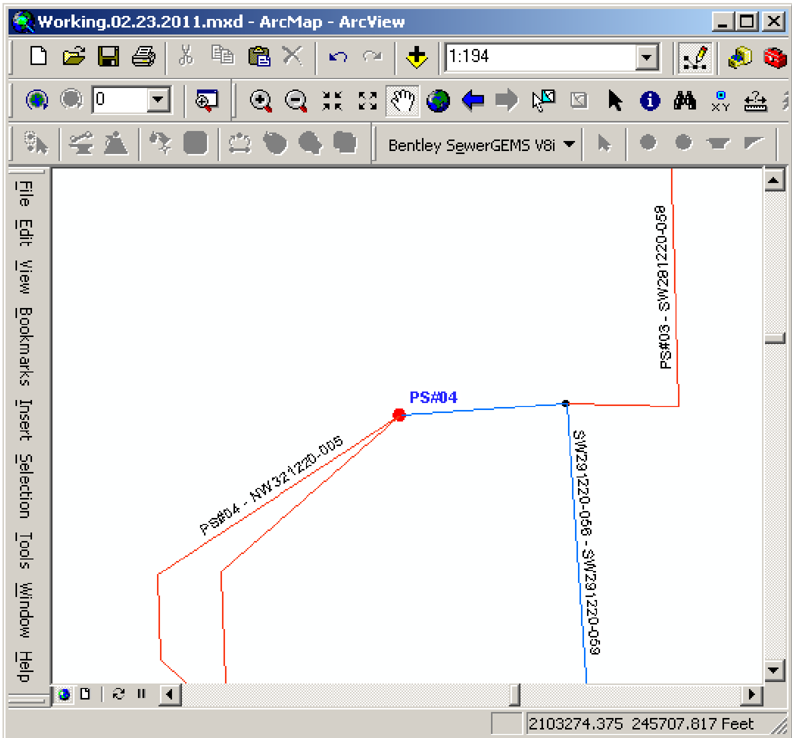


Figure 1. GIS Representation of Pump Station 04

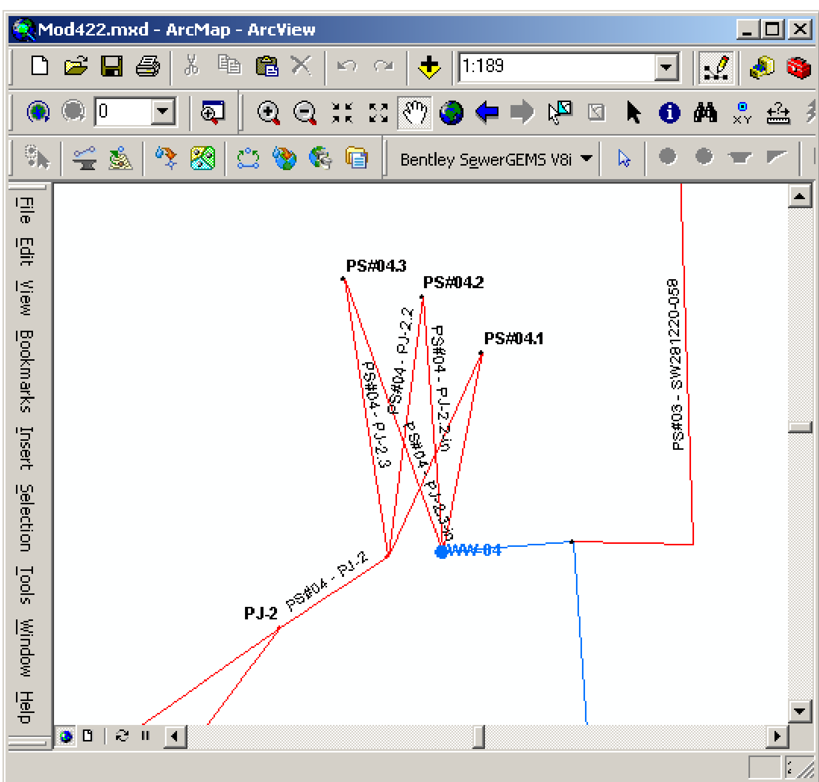


Figure 2. Model Representation of Pump Station 04

There are other GIS modifications made necessary by technical shortcomings that exist in all computational hydraulic models. Changes to the "real world" are necessary for conditions such as short (meaning sewers shorter than 30 feet) sewers where they are lengthened and given a lower roughness coefficient to result in a sewer equivalent to the actual sewer. This and other similar adjustments are accomplished automatically by the software.

D. Electronic Data Summary Spreadsheet File

An electronic data Excel spreadsheet file is included with this summary. It provides a record of all nodes (manholes, junctions, pressure junctions, and pump station wet wells), links (gravity sewers and force mains), and pump data from the model. The data are copied directly from the model.

E. Model Validation Procedure and Outcome

A validation procedure has been performed to demonstrate the model has been properly constructed. The procedure involves inputting a small amount of flow, in this case 1 gallon per minute per manhole throughout the system, and verifying the input flow arrives at various points in the system such as pump stations and the wastewater treatment plants at the volumes expected at those points in the system.

Upon running the model using this validation procedure, it was determined that just over 99% of the total flow input to the model was seen at the output end of the model which is the wastewater treatment plant. This degree of accuracy is considered to be very good and constitutes an acceptable validation of the model. In addition, expected influent flows to pump stations were compared to flows leaving pump stations. This validation at each pump station is summarized in Table 1 based on a 12 hour simulation run. The weighted average result of this validation procedure is an influent flow volume of slightly more than 100% of the total flows leaving the pump stations which is again considered to be a very good and acceptable level of accuracy.

Table 1: Validation Calculations at Pump Stations

Facility ID	Direct MHs	Sum of u/s MHs	Expected 6-hour Volume (cf)	Inflow to PS (cf)	Pumped Volume out of PS (cf)	Outflow Volume / Inflow Volume	Expected Volume / Inflow Volume
PS#01	40	61	21,960	22,506	23,079	103%	97.6%
PS#02	44	44	15,840	15,840	18,432	116%	100.0%
PS#03	171	232	83,520	83,553	95,280	114%	100.0%
PS#04	77	349	125,640	136,616	131,849	97%	92.0%
PS#5a		331	119,160	123,817	126,894	102%	96.2%
PS#5b		4,575	1,647,000	1,650,656	1,652,794	100%	99.8%
PS#06	100	100	36,000	35,999	48,426	135%	100.0%
PS#07	8	8	2,880	2,880	735	26%	100.0%
PS#08	360	360	129,600	134,825	160,835	119%	96.1%
PS#09		2,480	892,800	902,706	894,964	99%	98.9%
PS#12	20	20	7,200	7,200	5,954	83%	100.0%
PS#13	2	2	720	720	617	86%	100.0%
PS#15	16	16	5,760	5,760	8,823	153%	100.0%
PS#16		2,304	829,440	835,105	822,545	98%	99.3%
PS#19	103	230	82,800	84,227	102,656	122%	98.3%
PS#22	19	19	6,840	6,840	7,025	103%	100.0%
PS#23	16	16	5,760	2,160	4,512	209%	266.7%
PS#25	57	165	59,400	58,056	52,369	90%	102.3%
PS#27	99	99	35,640	35,639	26,665	75%	100.0%
PS#28	40	40	14,400	14,400	15,715	109%	100.0%
PS#31	5	5	1,800	1,800	1,884	105%	100.0%
PS#32	109	125	45,000	44,705	38,877	87%	100.7%
PS#34	13	13	4,680	4,680	4,571	98%	100.0%
PS#35	16	16	5,760	5,760	6,033	105%	100.0%
PS#37	60	60	21,600	21,599	22,493	104%	100.0%
PS#42	213	213	76,680	76,677	77,037	100%	100.0%
PS#43	13	13	4,680	4,680	4,927	105%	100.0%
PS#44	175	175	63,000	62,998	47,459	75%	100.0%
PS#45	11	11	3,960	3,960	3,957	100%	100.0%
PS#46	86	102	36,720	36,766	31,021	84%	99.9%
PS#48	210	**210**	75,600	75,597	65,544	87%	100.0%
PS#49	87	95	34,200	33,327	34,822	104%	102.6%
PS#50	11	11	3,960	3,960	3,850	97%	100.0%

Technical Memorandum No. 3
Future System Evaluation and Improvement Plan

Wastewater Facilities Master Plan

for

Lawrence, Kansas

City of Lawrence, Kansas

BMcD Project No. 54793

City P.O. 07629

Burns & McDonnell Engineering Company, Inc.
9400 Ward Parkway
Kansas City, MO 64114

City of Lawrence, Kansas

Wastewater Facilities Master Plan Technical Memorandum No. 3

Table of Contents

List of Tables

List of Figures

* = follows page number.

Appendix

A. Introduction

Technical Memorandum No. 3 is a summary of the forecast and distribution of future wastewater flows for the planning years 2020 and 2030; the analysis of a flow-development "trigger" that will be used to guide the scheduling for planning, design and construction for the future Wakarusa Wastewater Treatment Plant (Wakarusa); and the analysis of wastewater collection facilities improvements needed to serve growth and development forecast for planning years 2020 and 2030 plus conveyance of flows to the future Wakarusa; all in partial fulfillment of the Lawrence, Kansas Wastewater Facilities Master Plan. The goals of this technical memorandum were to:

- Determine a flow-development trigger for the start of further planning and then design and construction of the future Wakarusa which will put the new Wakarusa in service before Kansas River Wastewater Treatment Plant (KRWWTP) flows and pollutant loadings reach its capacity.
- Modify the existing collection system computer hydraulic model to reflect the forecast of 2020 and 2030 growth and associated increases in wastewater flows within the existing service area, plus extensions of the service area.
- Develop a plan of improvements to the existing collection system to address current capacity deficiencies during the design storm wet weather event and accommodate projected 2020 and 2030 service area growth.
- Develop a plan to convey flows in excess of KRWWTP capacities to the future Wakarusa.

B. Flow-Development Trigger for Future Wakarusa Wastewater Treatment Plant

1. General Considerations

The flow/development trigger must anticipate how much time will be needed to complete the design and construction of the Wakarusa. Based on experience with projects of this type, a minimum of 4 years should be scheduled for design and construction of a new wastewater treatment plant of the scale of the Wakarusa. A 5-year schedule is recommended to provide for scheduling uncertainties such as permitting and regulatory reviews of the project.

The flow/development trigger should also be based on bringing the Wakarusa on line before the full capacity of the KRWWTP is reached. This will provide an appropriate contingency for various factors such as accelerated growth during the 5-year project design and construction schedule, to insure the

KRWWTP will perform within its permitted effluent limits up to the time the Wakarusa is on line and to provide time for the start-up phase of the Wakarusa.

2. Population Based Trigger

The first flow/development trigger evaluated is based on wastewater utility service area population. Population is seen to be a more meaningful trigger than wastewater flow for the following reasons:

- Wastewater flow can vary significantly from year to year due to reasons that do not relate to service area development, such as unusually wet or dry conditions.
- Population accounts for other parameters which affect wastewater treatment plant capacity in addition to flow rate, such as pollutant loading rates that tend to relate closely to service area population.

The 1999 KRWWTP design memorandum establishes plant capacities based on a design population of 100,000, or nearly 7,300 more than 2010 population. There is some possibility that parts of the plant are designed for somewhat more capacity, such as an extra 10% aeration basin capacity for nitrification (ammonia reduction). This analysis, however, is based on an overall plant capacity for a population of 100,000. As suggested earlier, a contingency should be incorporated into the setting of the trigger. A population contingency of 2,000 is recommended to provide a buffer of more than one year. This requires the Wakarusa to be scheduled to be on line when population reaches 98,000. With utility service area population projected to grow annually by 1,394 from 2010 to 2020, it will be necessary for design and construction of the proposed Wakarusa to start at a population trigger of 91,000. This would provide a schedule of about 5 years to complete the project and have the Wakarusa on line before development exceeds the design capacity of the KRWWTP.

3. Pollutant Loading Based Trigger

The second flow/development trigger is based on pollutant loading rates. A brief review of KRWWTP operating data indicates influent wastewater characteristics that are typical for municipal wastewater. There are several pollutant loading rate parameters used for establishing wastewater treatment plant capacities. The single most significant pollutant loading parameter for establishing wastewater treatment plant capacities is BOD. Unlike flow rates which can vary from year to year for reasons unrelated to service area development, BOD loading rates for typical municipal wastewater normally track population and commercial development in a fairly predictable manner.

The 1999 KRWWTP design memorandum establishes plant capacities based on an average BOD loading rate of 15,800 pounds per day (ppd) and a maximum month BOD loading rate of 20,370 ppd. The resulting BOD loading rates per person based on the design population of 100,000 are 0.158 ppd/person and 0.204 ppd /person, respectively, at the average and maximum month BOD loading rates. Based on plant operating data and population estimates from 2003 to present, daily BOD loadings per person have averaged 0.165 ppd and maximum month BOD loadings have averaged 0.194 ppd. The historical average BOD loading rate per person is somewhat higher than the plant design average loading rate, but the historical maximum month BOD loading rate per person is lower than the plant design maximum month BOD loading rate. Plant facilities are sized for the maximum month BOD loading rate. As such, the KRWWTP has sufficient capacity for a population of 105,000 based on its design maximum month BOD loading rate of 20,370 and the recent historical maximum month BOD loading rate per person of 0.194 ppd/person.

Once again, a population contingency of 2,000 is recommended to provide a buffer of more than one year. This requires the Wakarusa to be scheduled to be on line when population reaches 103,000. This would result in a population trigger of 96,000.

4. Recommended Wakarusa Wastewater Treatment Plant Trigger

Of the two triggers evaluated, the recommended Wakarusa trigger is the pollutant loading based trigger. It is recommended since it more accurately relates the existing KRWWTP capacity to population by reflecting actual per person pollutant loading rates rather than estimated per person pollutant loading rates established at the time the KRWWTP was designed in 1999. The recommended population trigger is 96,000. According to the service area population forecast shown earlier, a service area population of 96,000 is expected to be reached by 2012-13. Construction of the new Wakarusa would be completed by 2017-18 under this growth scenario.

Some judgment will be needed to decide when conditions have actually reached the trigger point to start design as follows:

- The recommended outcomes are based on population increasing at a rate of 1,394 per year. If actual growth proves to be slower - say 1,000 per year - the pollutant loading based trigger can be revised to 98,000 with design starting in 2015 and construction completed in 2020.
- There has been some scatter in KRWWTP BOD loading rates measured from month to month and year to year due to various factors, including sampling frequency and technique,

and analytical methods used. This is why the BOD loading analysis is converted to an equivalent population, which is expected to track actual BOD loading rates to the KRWWTP closely. It is probably unreasonable, for example, to trigger the start of the Wakarusa design based on a single high month BOD result.

- The analysis of triggers is based on KRWWTP design capacities established at the time the current plant facilities were designed in 1999. It is possible that actual plant capacities could be greater than design capacities, which may be proven out by historical operating data and plant performance. This would involve a formal process with KDHE to re-rate the plant capacity and modify the NPDES discharge permit to reflect the revised capacity. This topic was discussed with the Kansas Department of Health and Environment (KDHE) during a meeting on future regulation changes and effluent limits. KDHE indicated that re-rating the Kansas River WWTP would require an antidegradation review, likely resulting in nutrient limits for the re-rated plant and is, therefore, not a practical option. Minutes of the meeting with KDHE are included in Appendix 3-A.
- Not addressed by this analysis is the likelihood of future nutrient limits at the KRWWTP and their timing. Some de-rating of KRWWTP capacity might be necessary to meet future nutrient limits depending on what limits may be required and the type and size of new facilities that are needed to meet the limits. Based on available information, however, having to de-rate the plant capacity to meet future nutrient limits appears unlikely.

C. Year 2020 System Analysis

1. Hydraulic Model Development

The existing system computer hydraulic model developed in TM-2 was extended, as appropriate, to serve projected growth and development to year 2020 as set forth in TM-1. The model includes the same assumptions concerning the rapid I/I reduction program and diversion of flows to the future Wakarusa as incorporated in the improved existing system model. The model also includes additional firm pumping capacity as needed at Pumping Station Nos. 9 and 32, and elimination of Pumping Station No. 8 by a new gravity sewer. Some new gravity sewers, pumping stations and force mains are needed to extend service to the projected year 2020 growth areas. Sizing of gravity sewers is based on ultimate or build-out development within the tributary area, while pumping stations and force mains are sized for year 2030 peak flows forecast within the tributary area. Flows from new development areas south of the Wakarusa River will be conveyed directly to the future Wakarusa. A summary of the year 2020 system model at the design storm is depicted on Figure 3.1. Overloaded or surcharged sewers are highlighted in yellow. A flow hydrograph predicted by the model at the KRWWTP is shown on Figure 3.2.

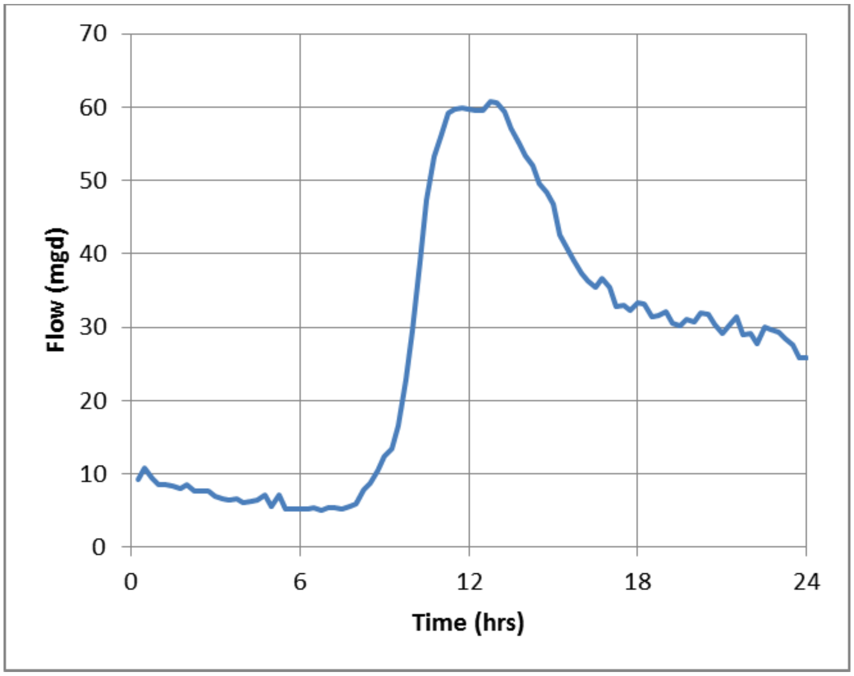


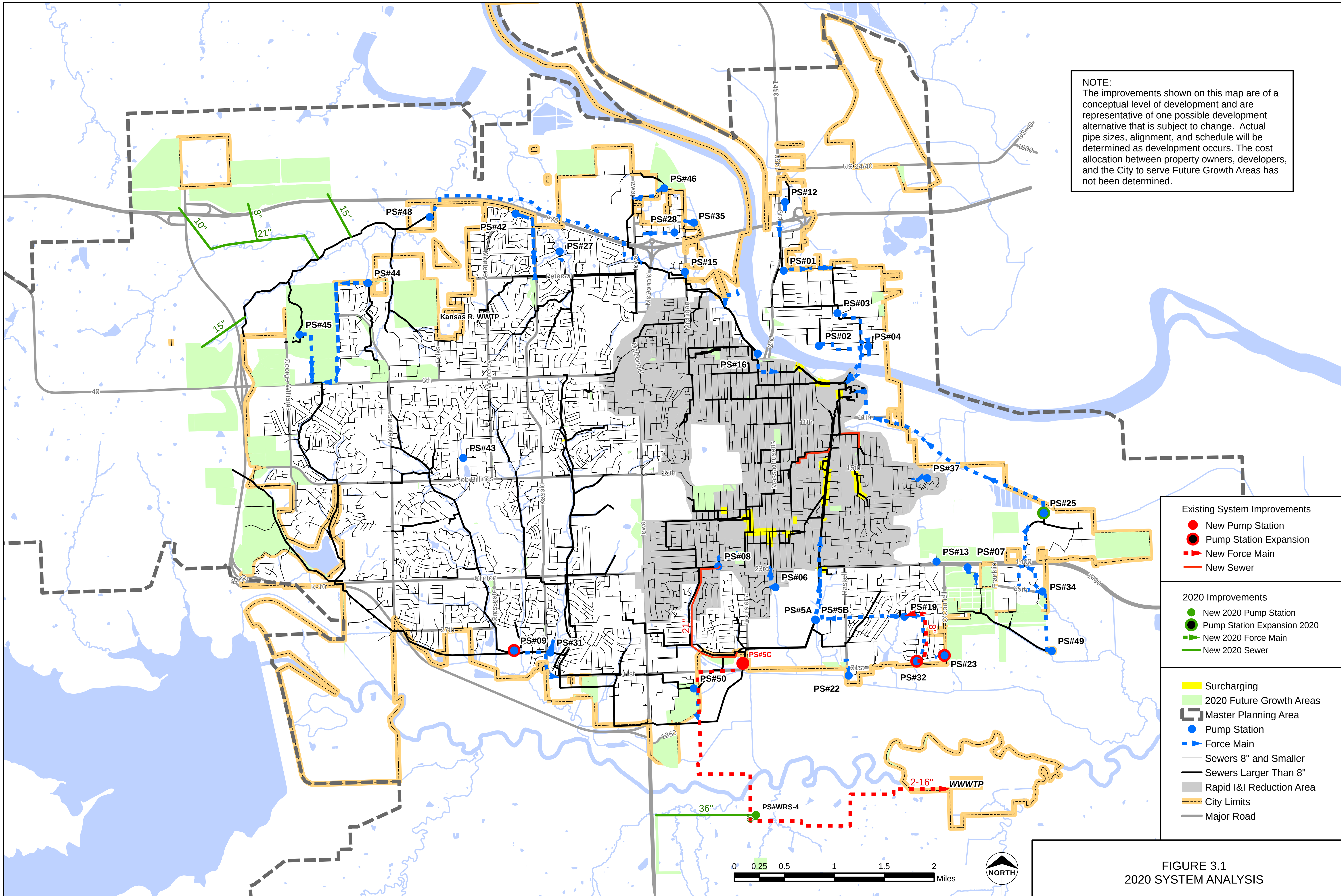
Figure 3.2. 2020 Design Storm Flow Hydrograph at the Kansas River WWTP

The model predicts an instantaneous peak flow rate of 61 MGD.

2. Year 2020 System Analysis Conclusions

Based on an analysis of the year 2020 model results, the following conclusions can be made:

- There is some limited additional surcharging of gravity sewers compared to the improved existing system model.
- The peak flow at the KRWWTP remains below its existing peak flow firm capacity.
- The peak flow at Pumping Station No. 23 exceeds its firm capacity causing some limited surcharging of upstream sewers.
- The peak flow at Pumping Station No. 25 exceeds its firm pumping capacity causing some limited surcharging of upstream sewers. Conditions at Pumping Station No. 25 at projected 2030 development and flows are examined later to determine what is the best approach to addressing this deficiency.



NOTE:
The improvements shown on this map are of a conceptual level of development and are representative of one possible development alternative that is subject to change. Actual pipe sizes, alignment, and schedule will be determined as development occurs. The cost allocation between property owners, developers, and the City to serve Future Growth Areas has not been determined.

FIGURE 3.1
2020 SYSTEM ANALYSIS

D. Year 2030 System Analysis

1. Hydraulic Model Development

The existing system computer hydraulic model developed in TM-2 was extended, as appropriate, to serve projected growth and development to year 2030 as set forth in TM-1. The model includes the same assumptions concerning the rapid I/I reduction program and diversion of flows to the future Wakarusa as incorporated in the improved existing system model. The model also includes additional firm pumping capacity, as needed, at Pumping Station Nos. 9, 23, 25 and 32, and elimination of Pumping Station No. 8 by a new gravity sewer. Some new gravity sewers, pumping stations and force mains are needed to extend service to the projected year 2030 growth areas. Sizing of gravity sewers is based on ultimate or build-out development within the tributary area, while pumping stations and force mains are sized for year 2030 peak flows forecast within the tributary area. A summary of the year 2030 system model at the design storm is depicted on Figure 3.3. Overloaded or surcharged sewers are highlighted in yellow. A flow hydrograph predicted by the model at the KRWWTP is shown on Figure 3.4.

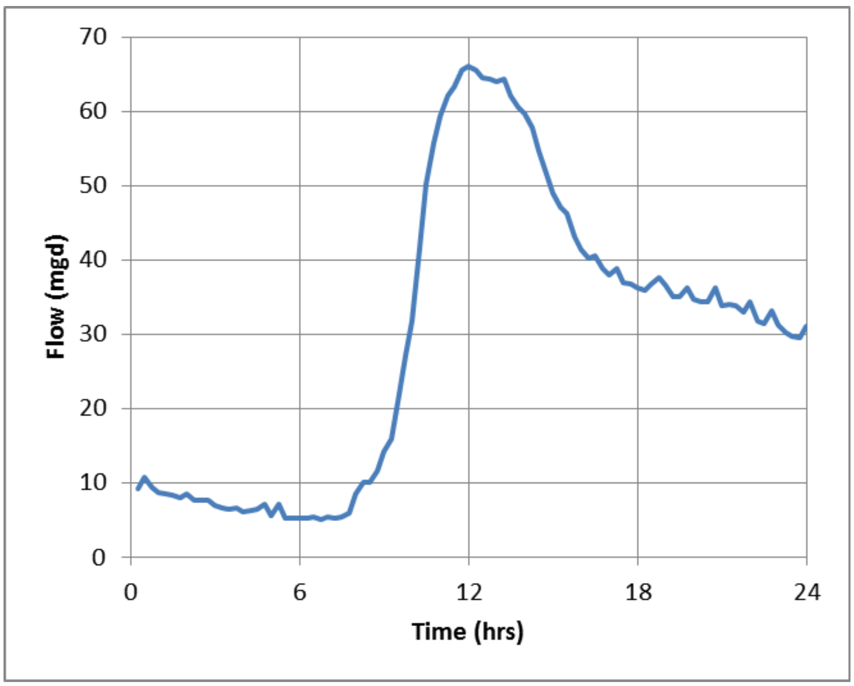
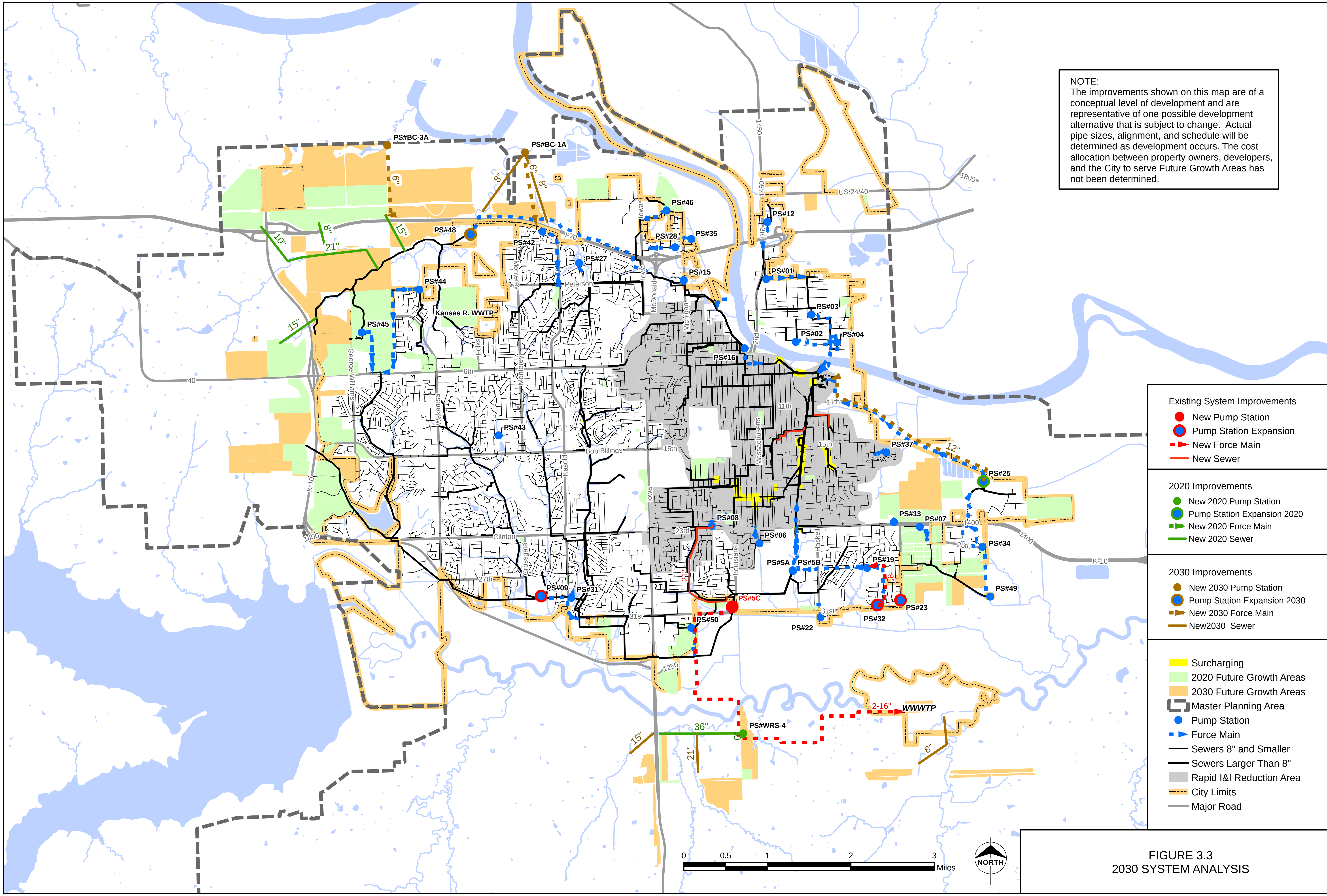


Figure 3.4. 2030 Design Storm Flow Hydrograph at the Kansas River WWTP

The model predicts an instantaneous peak flow rate of 65 MGD.


NOTE:
The improvements shown on this map are of a conceptual level of development and are representative of one possible development alternative that is subject to change. Actual pipe sizes, alignment, and schedule will be determined as development occurs. The cost allocation between property owners, developers, and the City to serve Future Growth Areas has not been determined.
Existing System Improvements
New Pump Station
Pump Station Expansion
New Force Main
New Sewer
2020 Improvements
New 2020 Pump Station
Pump Station Expansion 2020
New 2020 Force Main
New 2020 Sewer
2030 Improvements
New 2030 Pump Station
Pump Station Expansion 2030
New 2030 Force Main
New2030 Sewer
Surcharging
2020 Future Growth Areas
2030 Future Growth Areas
Master Planning Area
Pump Station
Force Main
Sewers 8" and Smaller
Sewers Larger Than 8"
Rapid I&I Reduction Area
City Limits
Major Road
0 0.5 1 2 3
Miles
NORTH
PS#BC-3A
PS#BC-1A
PS#48
PS#42
PS#46
PS#28
PS#35
PS#27
PS#15
PS#12
PS#01
PS#03
PS#02
PS#04
PS#44
PS#45
Kansas R. WWTP
PS#16
PS#43
PS#37
PS#25
PS#13
PS#07
PS#08
PS#06
PS#34
PS#5A
PS#5B
PS#19
PS#49
PS#09
PS#31
PS#5C
PS#50
PS#32
PS#23
PS#22
PS#WRS-4
WWWTP
2-16"
36"
US-24/40
I-70
K-10
40
1400
1450
1800
1250
6th
11th
15th
23rd
25th
27th
31st
Bob Billings
Peterson
Clinton
Iowa
Kasold
Wakarusa
Monterey
Folks
George Williams
McDonald
Michigan
Massachusetts
Louisiana
Haskell
Crossgate
2nd
3rd
FIGURE 3.3
2030 SYSTEM ANALYSIS

2. Year 2030 System Analysis Conclusions

Based on an analysis of the year 2030 model results, the following conclusions can be made:

- There is some limited additional surcharging of gravity sewers compared to the 2020 system model.
- The peak flow at the KRWWTP reaches its existing peak flow firm capacity.
- The peak flows at Pumping Station No. 48 exceed its firm capacity.

E. System Improvements Plan

1. Summary of Deficiencies

A summary of system deficiencies is presented below in Table 3.1, indicating the affected system component, existing capacity, and the amount and timing of additional capacity needed.

Table 3.1
Summary of Deficiencies

Drainage Sub-Basin	Description	Existing Peak Capacity - MGD(1)	Design Storm Peak Flow - MGD			Year Needed
			2010	2020	2030	
BC-1	PS 48	6.0	0.3	3.2	6.4	2030(2)
C-2	PS 08	2.9	12.7	6.4	6.6	(3)
C-2	PS 08 Force Main	3.3	12.7	6.4	6.6	(3)
EL-1	PS 23	0.05	0.05	0.07	0.1	(3)
EL-1	PS 32	0.7	1.6	1.7	1.7	(3)
EL-1	PS 32 Force Main	0.8	1.6	1.7	1.7	(3)
KR-5C	12-inch Sewer	1.0	2.9	2.6(4)	2.6(4)	(3)
KR-6A	PS 25	3.6	1.7	3.4	5.1	2020(5)
KR-6A	PS 25 Force Main	3.6	1.7	3.4	5.1	2020(5)
KR-6B	21-inch Sewer	4.0	9.4	6.6(4)	8.3(4)	(3)
WR-1	PS 09	8.6	11	13	15	(3)
WR-6	PS 5A/5B	15.5	24	26	26	(3)
WR-6	PS 5A/5B Force Mains	15.5	24	26	26	(3)

(1) Pumping station capacities shown are based on firm pumping capacities.
(2) Verify based on actual growth and development.
(3) As soon as funding will allow to provide capacity for design storm peak flow rate.
(4) Following Rapid I/I Reduction Program
(5) Verify expanding PS 25 instead of directing PS 49 flow to future Wakarusa WWTP is preferred plan based on actual growth and development.

2. Analysis of Required Improvements

a. Pumping Station Nos. 5A/5B

Pumping Station Nos. 5A/5B requires immediate expansion to provide capacity for the design storm peak flow rate, as well as some further additional capacity for future growth and development through year 2030. As explained in TM-2, significant expansion of this pumping station is not feasible due to site constraints. Furthermore, TM-2 determined the only plan for diverting flows to the future Wakarusa, which also addresses the wet weather peak flow issues concerning Pumping Station Nos. 5A/5B and the KRWWTP is the construction of a new pumping station upstream of 5A/5B. This new pumping station is identified as Pumping Station 5C and will have a firm capacity of 11 MGD.

b. Pumping Station No. 8

Plans have been in place to eliminate Pumping Station No. 8 due to its age and condition and need for additional capacity. As such, a 21-inch diameter gravity sewer intercepting flows into Pumping Station No. 8 and conveying them south to the interceptor sewer tributary to Pumping Station Nos. 5A/5B is recommended.

c. Pumping Station No. 9

Pumping Station No. 9 requires expansion from 8.6 MGD to 15 MGD to accommodate existing wet weather peak flows and projected upstream growth and development. The expansion could be done in stages, but it is considered to be more cost effective to complete the full expansion at one time. The existing structure and piping is designed to accommodate two more pumps. It may also be necessary to replace existing pumps to provide the needed capacity. The additional pumps and other station improvements should be configured to provide flexibility for pumping all flows east to the downstream collection system during dry weather periods, and pumping varying portions of wet weather peak flows east to the downstream collection system and to the existing wet weather peak flow storage basins. The division of capacities needs to be approximately 5 MGD east to the downstream collection system, and 10 MGD to the peak flow storage basins. Use of the wet weather peak flow storage basins will continue to limit peak flows received by the collection system downstream of Pumping Station No. 9, thereby reducing the required peak flow capacities of the downstream system. The 2030 model predicts a maximum of 5.9 MG of storage is needed or somewhat less than the current 6.25 MG capacity of the existing storage basins.

d. Pumping Station No. 23

The existing system model predicts the design storm wet weather peak flow to Pumping Station No. 23 equals its firm pumping capacity. Pumping capacity will need to be increased to accommodate growth through 2030. Future development and flows tributary to this pumping station should be monitored and necessary expansions be done as dictated by actual development. Additional firm capacity for this station beyond what is forecast for 2030 should be considered when it is expanded.

e. Pumping Station No. 25

The 2020 system model predicts 2020 wet weather peak flows to this pumping station will nearly reach its firm capacity. Past planning for the future Wakarusa anticipated it will be necessary to divert Pumping Station No. 49 flows which are now conveyed to Pumping Station No. 25 to the Wakarusa due to growth and development within the East Lawrence Drainage Basin. While the 2020 model indicates expansion of Pumping Station No. 25 could be delayed if this were done by 2020, the 2030 model shows it will be necessary to expand Pumping Station No. 25 even if Pumping Station No. 49 flows were diverted to the future Wakarusa. As such, an initial expansion of Pumping Station No. 25 by 2020 to 4.4 MGD firm capacity by addition of a third pump is recommended. A further expansion of Pumping Station No. 25 to 6 MGD by the addition of a second, parallel 12-inch diameter force main is recommended by 2025. The diversion of Pumping Station No. 49 flows to the future Wakarusa can be deferred until sometime after 2030. Actual development should be examined at the time it becomes necessary to expand Pumping Station No. 25 to confirm that is still appropriate to do so, or if actual growth and development in the East Lawrence Drainage Basin would instead dictate diverting Pumping Station No. 49 flows to the Wakarusa.

f. Pumping Station No. 32

The existing system model shows the Pumping Station No. 32 existing firm capacity of 0.7 MGD is exceeded by the design storm wet weather peak flow rate and requires expansion. The design storm peak flow rate in 2030 to this pumping station is forecast to be 1.7 MGD. Expansion of Pumping Station No. 32 firm capacity to 1.7 MGD is recommended, which will also require installation of a parallel 8-inch force main to provide the necessary peak flow capacity.

g. New Wakarusa Pumping Station 5C and Force Mains

As previously explained by the evaluation of Pumping Station Nos. 5A/5B, a new Pumping Station No. 5C with a firm capacity of 11 MGD is recommended to provide sufficient peak flow capacity through year 2030. This same pumping station will also serve to divert dry weather flows to the future Wakarusa.

A location near the northwest intersection of 31st and Louisiana Streets would be preferred location for this pumping station. The new pumping station and force mains should be constructed and placed into service at the same time the future Wakarusa is placed into service.

The force main from this pumping station will be routed west and then south and east to the future Wakarusa site. The range of dry and wet weather flows to be handled by this pumping station is wide, from as little as 1 to 3 MGD during dry weather periods up to the 11 MGD peak flow rate. As such, a dual force main is proposed, with one force main in service during dry weather periods, and both in service during peak wet weather flow conditions. Two 16-inch diameter force mains are recommended to provide sufficient flow velocity during dry weather flows when one force main will be in service.

h. Relief Sewers

Gravity sewer surcharging remains at two locations following the Rapid I/I Reduction Program within the program target area due to inadequate flow capacities for conveying the design storm peak flow rate. In these instances, parallel gravity relief sewers are recommended to provide the additional peak flow capacity needed to convey the design storm peak flow rate. The 12-inch gravity sewer in Drainage Sub-basin KR-5C requires a 12-inch parallel relief sewer. The 21-inch gravity sewer in Drainage Sub-basin KR-6B requires a 24-inch parallel relief sewer.

i. New Wakarusa Wastewater Treatment Plant

Earlier discussion concluded the future Wakarusa should be constructed and in service by the time the service area population reaches 103,000 which is forecast to occur in 2018. The KRWWTP pollutant loading capacity is estimated to be equivalent to a service area population of 105,000. The annual average daily flow at a service area population of 105,000 is estimated to be 12.2 MGD or somewhat less than the permitted flow capacity of the KRWWTP. At the projected 2030 service area population of 119,529, the annual average daily flow is estimated to be 13.9 MGD. The minimum required 2030 permitted flow capacity of the Wakarusa is, therefore, 1.7 MGD. A minimum initial treatment capacity of 2 MGD or more is recommended. A larger initial capacity may be appropriate given that the future Wakarusa is expected to be put into service in 2018, and with a 2 MGD capacity would nearly be operating at its capacity 12 years later in 2030 based on the population forecast used for this plan. The final selection of treatment capacity remains to be determined by further planning for the Wakarusa and will be based on costs and other factors concerning the most appropriate initial treatment capacity.

With a 2 MGD annual average daily flow capacity, the Wakarusa could readily be designed to fully treat wet weather peak flow rates up to approximately 6 MGD. This will not be sufficient peak flow rate capacity for the Pumping Station No. 5C required firm pumping capacity of 11 MGD. As such, flows received at the Wakarusa in excess of its peak flow capacity will need to be stored and then fully treated after flow rates return to less than its peak flow treatment capacity. The storage volume needed for the design storm event is estimated to be 4 MG.

j. Pumping Station No. 48

The 2030 system model predicts 2030 wet weather peak flows to this pumping station will marginally exceed its existing firm capacity. As such, future development and flows tributary to this pumping station should be monitored and necessary expansion be done as dictated by actual development. Additional firm capacity for this station beyond what is forecast for 2030 should be considered at the time it needs to be expanded.

k. Kansas River Wastewater Treatment Plant

At a meeting with KDHE arranged to discuss regulatory actions that may affect this master plan, KDHE informed the City that new effluent limits for nutrients (total nitrogen and phosphorus) should be anticipated at the time of the second 5 year renewal of the KRWWTP discharge permit. This would occur at approximately year 2020 with a compliance deadline likely to occur three years thereafter. Minutes of the meeting with KDHE are included in Appendix 3-A. This will require significant improvements to the KRWWTP as previously documented by others. Future wastewater utility capital improvements planning should include funding for the necessary improvements.

l. System Extensions to Future Growth Areas

Extensions of the existing collection system are needed to provide service to the future growth areas forecast to occur by years 2020 and 2030 as described below. The collection system extensions to future growth areas are based on a conceptual level of development and representative of one possible development alternative that is subject to change. Actual pipe sizes, alignment, and schedule will be determined as growth occurs. The cost allocation between property owners, developers, and the City to serve future development areas has not been determined.

- Baldwin Creek West of K-10 (BC-2): 15-inch gravity sewer west from existing Baldwin Creek Interceptor across K-10 to serve 2020 and 2030 growth areas west of K-10.

- Baldwin Creek North of I-70 (BC-3): 8-, 10-, 15-, and 21-inch gravity sewers extended from the existing Baldwin Creek Interceptor crossing to the north side of I-70 at three locations to serve 2020 and 2030 growth areas north of I-70.

- Wakarusa River South – US 59 & 1100 Road (WRS-5): 36-inch gravity sewer, 1 MGD firm capacity pumping station, and 8-inch force main connecting to one of the Pumping Station 5C force mains to the future Wakarusa to serve 2020 and 2030 growth areas east of US 59 and south of the Wakarusa River.

- Baldwin Creek North of I-70 (BC-1A): 8-inch gravity sewers, 0.5 MGD firm capacity pumping station, and 6-inch force main connecting to the 15-inch BC-3 gravity sewer to serve 2030 growth areas north of I-70.

- Baldwin Creek North of I-70 (BC-3A): 8-inch gravity sewers, 0.5 MGD firm capacity pumping station and 6-inch force main connecting to the Pumping Station No. 48 force main to serve 2030 growth areas north of I-70.

- Wakarusa River South – US 59 & 1100 Road (WRS-3, 5 & 6): Extension of 36-inch gravity sewer west across US 59 and 15-inch gravity sewer to serve 2030 growth areas west of US 59 and south of the Wakarusa River. New 21-inch gravity sewer from the 36-inch gravity sewer to serve 2030 growth areas east of US 59 and south of the Wakarusa River. New 8-inch gravity sewer from the future Wakarusa site south to serve 2030 growth areas south of the future Wakarusa.

3. Recommended System Improvements Plan

The recommended system improvements plan along with extensions to future growth areas are presented in Figure 3.5. Improvements are shown as required by 2020 to provide capacity for wet weather peak flow rates during the design storm event, to provide for projected growth and development through 2020, and for diversion of a portion of both dry and wet weather flows as necessary to the future Wakarusa. A limited amount of further improvements are needed by 2030 to accommodate further growth and development expected to occur between 2020 and 2030. Table 3.2 provides a summary of recommended improvements to the existing system and their timing.

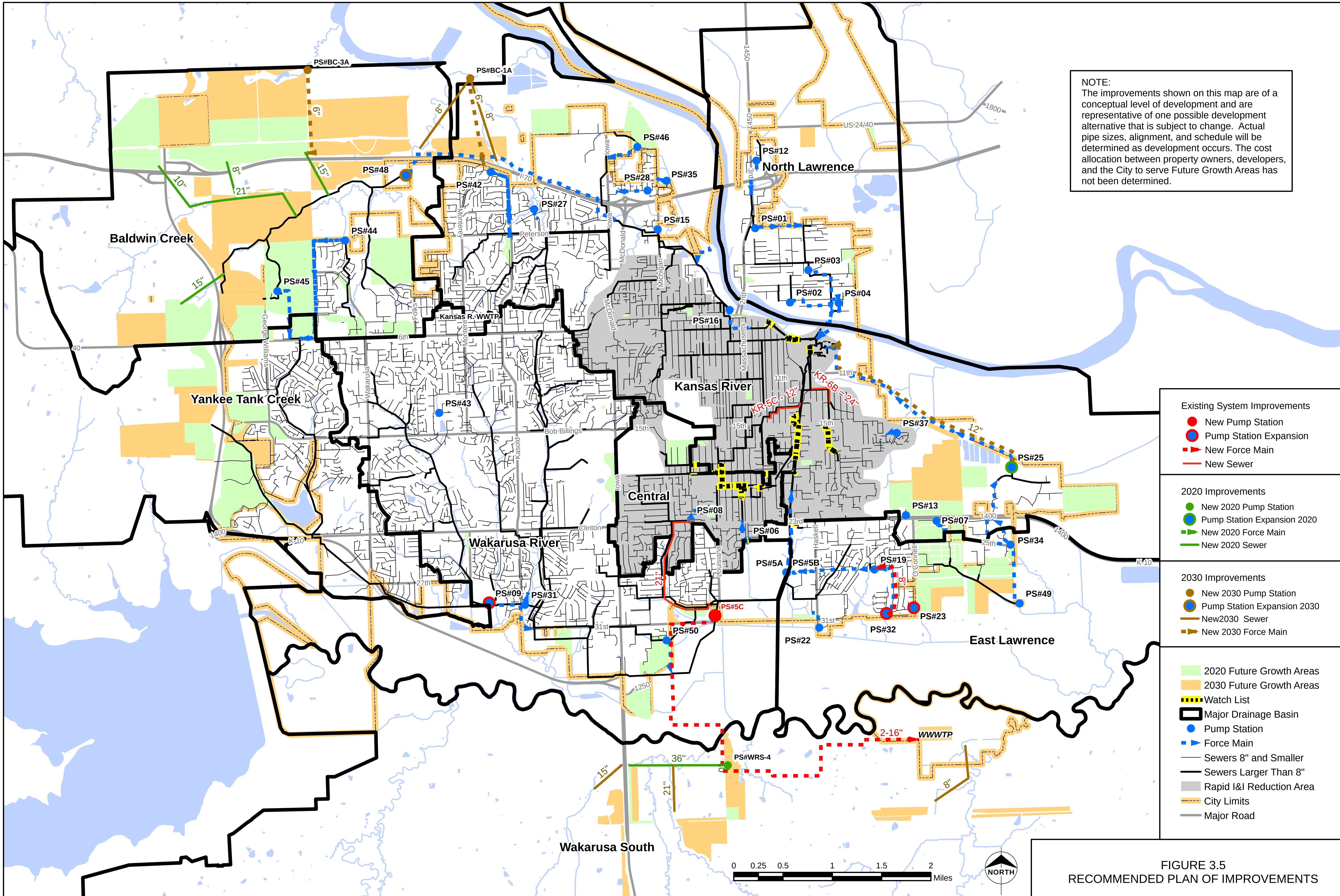


FIGURE 3.5
RECOMMENDED PLAN OF IMPROVEMENTS

Table 3.2
Summary of Recommended Existing System Improvements

Drainage Sub-Basin	Description	Existing Peak Capacity - MGD(1)	Design Storm Peak Flow - MGD			Year Needed
			2010	2020	2030	
(1)	Rapid I/I Reduction Program (1)	(2)	(2)	(2)	(2)	2016(2)
BC-1	Expand PS 48 to 6.4 MGD	6.0	0.3	3.2	6.4	2030
C-2	Replace PS 08 with Gravity Sewer	2.9	6.1(3)	6.4(3)	6.6(3)	(4)
C-2	Replace PS 08 Force Main with Gravity Sewer	3.3	6.1(3)	6.4(3)	6.6(3)	(4)
EL-1	Expand PS 23 to 0.1 MGD	0.05	0.05	0.07	0.1	(4)
EL-1	Expand PS 32 to 1.7 MGD	0.7	1.1	1.7	1.7	(4)
EL-1	Parallel PS 32 Force Main	0.8	1.1	1.7	1.7	(4)
KR-5C	12-inch Relief Sewer	N/A	N/A	1.6(3)	1.6(3)	(4)
KR-6A	Expand PS 25 to 4.4 MGD	3.6	1.6	3.4	5.1	2020
KR-6A	Expand PS 25 to 6.0 MGD	3.6	1.6	3.4	5.1	2025(5)
KR-6B	24-inch Relief Sewer	N/A	N/A	4.2(3)	4.3(3)	(4)
WR-1	Expand PS 09 to 15 MGD	8.6	11	13	15	(4)
WR-6	New PS 5C to Wakarusa WWTP	N/A	8.5	10.5	10.5	2018
WR-6	New PS 5C Force Mains to Wakarusa WWTP	N/A	8.5	10.5	10.5	2018
WRS-3	New 2 MGD Wakarusa WWTP	N/A	N/A	2	2	2018
WRS-3	4 MG Storage at Wakarusa WWTP	N/A	N/A	N/A	N/A	2018

(1) Pumping station capacities shown are based on firm pumping capacities
(2) As explained in further detail in TM-4.
(3) After completion of Rapid I/I Reduction Program.
(4) As soon as funding will allow to provide capacity for design storm peak flow rate.
(5) Verify expanding PS 25 instead of directing PS 49 flow to future Wakarusa WWTP is preferred plan based on actual growth and development.

The model indicates some sewer surcharging remaining following implementation of the recommended improvements. The model indicates these to be marginally surcharged conditions which are considered to be acceptable at these locations and within the degree of accuracy of the system model. These locations are identified on Figure 3.5 as sewers recommended to be put on a “watch list” to be periodically monitored over time during significant wet weather events to verify conditions are acceptable, or if necessary, corrective action taken.

* * * * *

Appendix 3-A
Minutes of September 22, 2010 Meeting with the Kansas Department of Health and Environment

WASTEWATER MASTER PLAN

City of Lawrence, Kansas

Meeting with Kansas Department of Health and Environment
September 22, 2010, 2:00 PM
KDHE – Curtis Building

Kansas River WWTP and Proposed Wakarusa River WWTP
Flow Capacities, Effluent Limits

MEETING MINUTES

1. See attached attendance sheet for list of attendees.

2. Summary of Current Master Planning
   a. Population
      -Population forecast prepared by City Planning and Development Services for both water and wastewater master plans through 2030
         2010: 92,000 (estimate)
         2020: 108,500
         2030: 125,000
      -This population forecast is less than that used by the 2003 master plan which was a population of 150,000 in year 2025.
   b. Master Plan Service Area: See attached service area map. The service area for the current master plan has been modified from the 2003 master plan service area to reflect an expanded service area in the northwest consistent with the K-10 and Farmers Turnpike Sector Plan and a scaled back service area in the northeast consistent with the draft Northeast Sector Plan. The south service area border is based on the adopted Horizon 2020 and Transportation 2030 urban growth area.
   c. The focus of the current master plan is the collection system. The 2003 master plan only modeled sewers 12" and larger and was based on 6 flow meter locations and two months of flow meter data. The current master plan will model all sewers and will be based on 37 flow meter locations with up to four years of data.

3. Kansas River WWTP Capacity – See Attached Draft Wakarusa Trigger Memorandum
   a. Design Population Based Capacity: 100,000
   b. Pollutant Loading Based Design Capacity: 105,000. It was suggested we consider ammonia loadings in the assessment of the pollutant loading based design capacity. Ammonia loadings will generally affect aeration system capacity. The operational and performance data of the aeration basins suggests (even though plant ammonia and hydraulic loadings approach or exceed design criteria) that there is a significant amount of remaining treatment capacity for additional organic and or ammonia load. Specifically the basins routinely operate with excess oxygen as aeration rates are often driven by minimum mixing requirements and not by oxygen demand.

4. Forecast Wakarusa River Start-up Date
   a. Population Based:
      -Near Term Growth - 2015
      -Master Plan Projection - 2014
   b. Pollutant Loading Based:
      -Near Term Growth - 2019
      -Master Plan Projection - 2017

5. Discussion of Effluent Standards
   a. Kansas River WWTP – Based on 12.5 MGD Permitted Capacity
      (1) Near Term: The next permit is expected to be issued for 5 years and have a requirement to perform a study of nutrient removal costs for various nutrient reduction goals. There are no expected changes to the effluent limits contained in the draft permit to which the EPA objected. No new effluent limits are expected.
      (2) After Wakarusa WWTP Start-up: New permit limits on nutrients (both total nitrogen and total phosphorus) should be anticipated for the second 5 year permit cycle or 8 years from the issuance of the draft permit.
      (3) Longer Term: New ammonia criteria are under development by EPA that will likely result in lower ammonia limits – perhaps as low as one-third of current limits. The new criteria could be in place within the next 6 years (two triennial review cycles of the water quality standards) and thus come into play as soon as the second 5 year permit cycle. Disinfection standards based on *enterrococci* rather than *E-coli* may be put in place, perhaps by the second 5 year permit cycle. The 503 biosolids land application rules are undergoing review and may become more restrictive in terms of pollutant limits. KDHE will follow up on whether there are any new TMDL's under development for the Kansas or Wakarusa Rivers.
      (4) Kansas River has been listed on the 303d impaired streams for Phosphorus.
      (5) Pharmaceuticals will likely be an area of future regulatory requirements.
   b. Wakarusa WWTP – Based on 7 MGD Permitted Capacity
      (1) Start-up – Based on 7.0 MGD Design Flow or Less: The current permit expires in 2011. Expect no changes in permit limits over the next 5 year renewal of the permit. The City needs to plan on doing the Wakarusa River biota assessment to benchmark its condition prior to beginning construction of the Wakarusa WWTP. This will be used after Wakarusa start-up to measure water quality impacts of the plant effluent on the Wakarusa River.
      (2) Longer Term: Same as Kansas River WWTP.

6. Wet Weather Treatment Strategies
   a. Maximize use of Kansas River Actiflo:
      -Plant Peak Flow Capacity – 25 MGD
      -Actiflo Peak Flow Capacity – 40 MGD
      -Total Peak Flow Capacity – 65 MGD
   b. Distribution of Flows between Treatment Plants:
      -To be determined by master plan
      -Maintain capability to direct some flows to either plant (Four Seasons Pump Station)

c. Wakarusa WWTP Peak Flow Management Strategy: Storage and full treatment of stored flows. Utilize existing storage at Four Seasons Pump Station plus new storage at the Wakarusa WWTP.
d. During wet weather peak flows, peak flow capacities of both treatment plants will be used, followed by Actiflo capacity, followed by storage and subsequent full treatment.
e. KDHE advised the City to continue with a meaningful program of collection system infiltration/inflow correction in combination with future use of Actiflo. Lack of an infiltration/inflow correction program will, in KDHE's words, "clash" with use of Actiflo given EPA's current objections to the Kansas River permit renewal. The City advised KDHE that they have not funded collection system infiltration/inflow correction over the past 2 years but are seeking funding to resume it this year.
f. KDHE takes no position on a design storm to be used for collection system infiltration/inflow evaluation. It was noted that Kansas City Missouri's program is based on a 5 year storm. A 10 year storm was used for the 2003 master plan and will be used for the current master plan.

7. Other Considerations
    a. Kansas River WWTP capacity rating analysis necessary to re-rate plant to allow construction of the Wakarusa WWTP to be delayed: KDHE pointed out a re-rating of the Kansas River WWTP would require an antidegradation review, likely resulting in nutrient limits for the re-rated plant, consistent with KDHE's nutrient reduction plan.
    b. "Effluent Trading" possibilities between Kansas River and Wakarusa WWTP's: KDHE is open to the possibility of effluent trading between the two plants.
    c. KDHE noted the Kansas River WWTP flows have exceeded its permitted capacity during some wet weather months which is acceptable as long as permit limits are met.

* * * * *

KDHE Meeting
Lawrence, Kansas Wastewater Facilities Master Plan
09/22/10

Name	Organization	Phone	Email
Karl Mueldener	KDHE	785-296-5502	kmueldener@kdheks.gov
Rod Geisler	"	5527	rgeisler@kdheks.gov
AURORA SHIELDS	CITY OF LAWRENCE	785 832-7817	ashield@ci.lawrence.ks.us
Mark Hegeman	"	923-3380	mhegeman@ci.lawrence.ks.us
DAVE WAGNER	City of Lawrence	785-832-7800	dwagner@ci.lawrence.ks.us.
Mike Lawless	City of Lawrence	785 832-7862	mlawless@ci.lawrence.ks.us
Ed DILLINGHAM	KDHE	785.296.5513	edillingham@kdheks.gov
Steve Yonke	Burns & McDonnell	816-822-3102	syonke@burnsmcd.com

Technical Memorandum No. 4
Wastewater Collection System Rehabilitation Plan

Wastewater Facilities Master Plan

for

Lawrence, Kansas

City of Lawrence, Kansas

BMcD Project No. 54793

City P.O. 07629

Burns & McDonnell Engineering Company, Inc.
9400 Ward Parkway
Kansas City, MO 64114

City of Lawrence, Kansas

Wastewater Facilities Master Plan Technical Memorandum No. 4

Table of Contents

List of Tables

List of Figures

* = follows page number

A. Introduction

Technical Memorandum (TM) No. 4 was completed in partial fulfillment of the Lawrence, Kansas Wastewater Facilities Master Plan to develop the wastewater collection system rehabilitation plan and budgetary costs. The recommended scope and funding level of the rehabilitation plan is based on:

- The conclusions established by hydraulic modeling set forth in TM No. 2
- An inventory of the wastewater collection system
- Input from City staff.

B. Development of Prioritization and Funding Level

Prior to the completion of hydraulic modeling for TM No. 2, a preliminary prioritization schedule was developed in order to evaluate likely Inflow/Infiltration (I/I) sources within the collection system. The prioritization schedule was developed by taking a partial inventory of a portion of the system identified to have high rainfall-derived infiltration and inflow (RDII). The inventory categorized sewer age and material, maintenance issues and failures in order to identify likely sources of I/I.

As an outcome of discussing this prioritization schedule with City Staff, an 18 year rehabilitation project plan was developed that included all Vitrified Clay Pipe (VCP) sewers, brick manholes and other manholes in need of rehabilitation based on the City's GIS data base.

Upon completion of the hydraulic modeling for TM No. 2, the results of the modeling established that the older parts of the system located in close proximity to the Kansas River Wastewater Treatment Plant (KRWWTP) are a higher priority than other I/I sources within the overall system. Moreover, it was concluded that the reduction of the peak flow rate caused by rapid I/I sources (sources with short travel time) is more critical than the overall reduction of the total volume of wet weather flows. Based on this conclusion set forth in TM No. 2, the rehabilitation plan set forth in this TM addresses rapid I/I sources in close proximity to the KRWWTP (Rapid I/I Reduction Program area) as a higher priority in the overall 18-year rehabilitation plan.

The scope of the Rapid I/I Reduction Program was also set forth by TM No. 2 based on a targeted amount of I/I reduction for both public and private sources located in close proximity to the KRWWTP. Figure 4.1 identifies the drainage basins and sub-basins in close proximity to the KRWWTP that are recommended in TM No. 2 to be addressed by the Rapid I/I Reduction Program. The area includes the oldest parts of the collection system and was found by TM-2 to have relatively high levels of RDII/IDM of sewer. The objective of the Rapid I/I Reduction Program is an overall 35% reduction of I/I within the areas shown in Figure 4.1. This will amount to removal of approximately 19 MGD of peak I/I occurring during the 10 year design storm. In order for the public portion of the Rapid I/I Reduction Program to effectively achieve the target I/I reduction, a Sewer System Evaluation Survey (SSES) should be completed prior to rehabilitation work to identify and evaluate public sources of rapid I/I. The overall 35% reduction objective would be achieved in conjunction with the private portion of the Rapid I/I Reduction Program. The private portion of the program requires building inspections and program management to identify and eliminate private I/I sources. Once the public and private programs have been implemented, monitoring programs should be put into place to measure I/I reductions concurrently as rehabilitation progresses. Based on results of the monitoring programs, the scope and timeline of the Rapid I/I Reduction Program can be adjusted as required to achieve the targeted rapid I/I reduction objective(s).

Rehabilitation of the remainder of the collection system would be addressed in a Clay Pipe and Manhole Rehabilitation Program. The phasing and funding level of the Clay Pipe and Manhole Rehabilitation Program is based on continuing the City's current efforts to rehabilitate VCP lines concurrently with the higher priority Rapid I/I Reduction Program. Subsequent to reaching the reduction target of the Rapid I/I Reduction Program, the Clay Pipe and Manhole Rehabilitation Program would be ramped up to be completed within an overall 18-year timeline for both programs. The Clay Pipe and Manhole Rehabilitation Program would address all of the remaining VCP sewers, brick manholes and other manholes in need of rehabilitation throughout the system, including remaining sewers and manholes not addressed by the Rapid I/I Reduction Program within the program area.

C. Inventory of Collection System

The inventory of the existing collection system was derived from the City of Lawrence GIS wastewater collection system database. Vitrified Clay Pipe (VCP) sewers, brick manholes and other manholes in need of rehabilitation were filtered and extracted from the database for (1) the entire system (system-wide) and (2) the Rapid I/I Reduction Program area. Additionally, an inventory of the total length of sewers and number of manholes located within the Rapid I/I Reduction Program area was taken. VCP sewers that were indicated in the GIS database to have already been lined were excluded from the

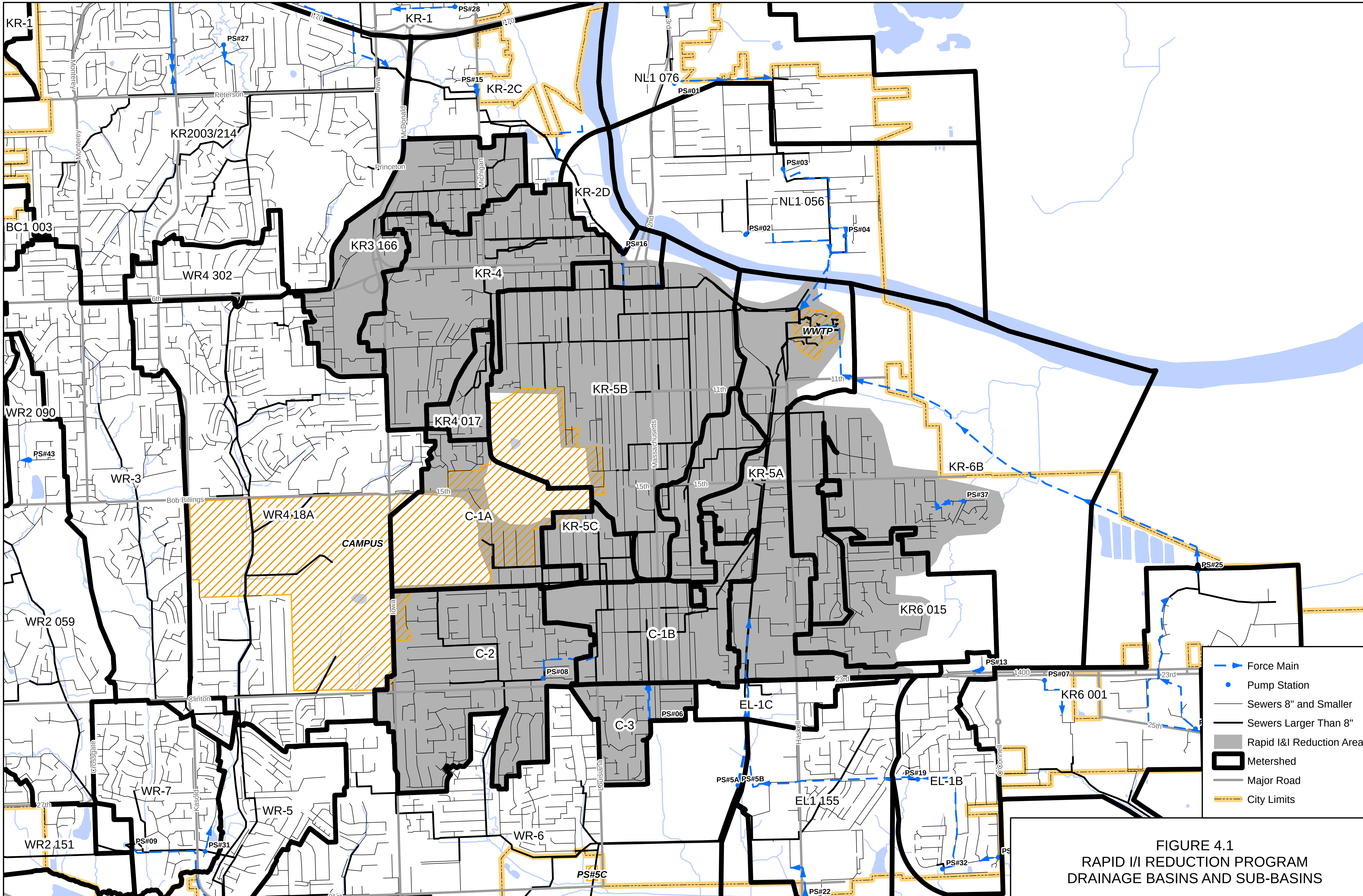


FIGURE 4.1
RAPID I/I REDUCTION PROGRAM
DRAINAGE BASINS AND SUB-BASINS

rehabilitation inventories. The entire system includes a total of 406 miles of gravity sewers of various materials, of which 111 miles are located in the Rapid I/I Reduction Program area.

The public portion of the Rapid I/I Reduction Program estimates there will be rehabilitation of 20% of the total length of pipes and manholes located in the Rapid I/I Reduction Program area. This level of rehabilitation is typical of system infiltration/inflow reduction programs based on a survey of cities where these programs have been completed. The remainder of pipe and manhole rehabilitation not addressed in the Rapid I/I Reduction Program would be addressed in the Clay Pipe and Manhole Rehabilitation Program to follow. The scope of the Clay Pipe and Manhole Rehabilitation Program was quantified by deducting the length of pipes and manholes rehabilitated in Rapid I/I Reduction Program from the system-wide rehabilitation inventory.

The Clay Pipe and Manhole Rehabilitation Program inventory quantified pipe lengths by diameter and manhole depths in order to estimate associated rehabilitation costs. Six-inch diameter lines extracted from the GIS database are included in the 8-inch diameter total as it is believed these are in fact 8-inch diameter lines per City Staff. Tables 4.1 and Table 4.2 below summarize the quantities of VCP pipe lengths and total vertical feet of manhole depths respectively. Both Tables show inventory totals for (1) the entire system, (2) the Rapid I/I Reduction Program, and (3) the resulting Clay Pipe and Manhole Rehabilitation Program totals.

Table 4.1

Total Lengths of VCP by Diameter (Linear Feet)

Area ↓	**Pipe Diameter**										
	8”	**9”**	**10”**	**12”**	**15”**	**18”**	**21”**	**24”**	**27”**	**30”**	**36”**
Entire System	547,580	3,028	33,699	58,643	27,987	16,508	9,391	40,738	5,687	3,815	374
Rapid I/I Reduction Program* *(20% of Rapid I/I Reduction Program Area)*	90,523	720	4,093	7,815	2,625	2,616	1,490	4,882	1,555	1,092	374
Clay Pipe Rehabilitation Program Totals *(Entire System – Rapid I/I Reduction Program)*	457,057	2,308	29,606	50,828	25,362	13,892	7,901	35,856	4,132	2,723	0

* Rapid I/I Reduction Program VCP lengths are shown for purposes of quantifying Clay Pipe and Manhole Rehabilitation Program Quantities only. VCP lengths were not used to develop Rapid I/I Reduction Program scope/budget. See Development of Probable Costs section in this TM for further information.

Table 4.2

Total Depths of Manhole Rehabilitation (Vertical Feet)

Area ↓	**Brick Manhole Depths**	**Depths of Other Manholes in Need of Rehabilitation**	**Total Manhole Rehabilitation Depths**
Entire System	19,581	2,571	22,152
Rapid I/I Reduction Program* *(20% of Rapid I/I Reduction Program Area)*	3,925	757	4,682
Manhole Rehabilitation Program Totals *(Entire System – Rapid I/I Reduction Program)*	963	2,154	17,470

* Vertical Footages of Manholes included in Rapid I/I Reduction Program are shown for purposes of quantifying Clay Pipe and Manhole Rehabilitation Program Quantities only. Vertical Manhole Heights were not used to develop Rapid I/I Reduction Program scope/budget. See Development of Probable Costs section in this TM for further information.

D. Development of Probable Costs

The Rapid I/I Reduction Program and Clay Pipe and Manhole Rehabilitation Program utilized different methods to develop budgetary costs. The budgetary costs for the Rapid I/I Reduction Program are based on a unit cost per foot of the 111 miles of sewers located within the Rapid I/I Reduction Program area. The budgetary costs for the Clay Pipe and Manhole Rehabilitation Program are based on actual quantities derived from inventories and corresponding unit costs.

The costs that were utilized in this report to develop funding levels for Rapid I/I Reduction Program and Clay Pipe and Manhole Rehabilitation Program are intended for budgetary purposes. The City should anticipate actual costs necessary to achieve the targeted I/I reductions set forth in TM No. 4 may require adjustments to the scope and/or duration of Rapid I/I Reduction Program and later phases of the rehabilitation plan based on (1) annual cost limits and (2) scope adjustments made to Rapid I/I Reduction Program based on measured I/I reduction as part of a monitoring program for both the public and private of the program.

Rapid I/I Reduction Program Costs:

The public portion of the Rapid I/I Reduction Program is based on a budgetary rehabilitation cost of $28.00 per foot of all sewers located within the Rapid I/I Reduction Program area. This budgetary figure includes costs to rehabilitate 20% of the total pipes and manholes within the Rapid I/I Reduction Program area, an SSES and other engineering services to identify and eliminate sources of I/I, and a monitoring program to measure I/I reduction and assess any required adjustment to the project scope. This budgetary cost is based on a survey of similar I/I reduction program costs experienced by several Midwestern wastewater utilities.

The private portion of the Rapid I/I Reduction Program is based on a budgetary program cost of $5.00 per foot of all sewers located within the Rapid I/I Reduction Program area. This budgetary figure includes costs for removal of private I/I sources such as downspouts, cleanout caps, and sump pumps, as well as an allowance for building inspections and management of the program, including a monitoring program to measure I/I reduction and assess any required adjustment to the project scope. These budgetary program costs are based on a similar program cost budget developed for a Midwestern wastewater utility. Budgetary costs for the Rapid I/I Reduction Program are summarized in Table 4.3 below.

Table 4.3
Rapid I/I Reduction Program Budgetary Costs

Public Rapid I/I Reduction Program Costs (588,939 ft.* x $28.00/ft.) = $ 16,490,292
*Total Sanitary Sewer Length within Priority 1 Program Area

Private Rapid I/I Reduction Program Costs (588,939 ft.* x $5.00/ft.) = $ 2,944,695
*Total Sanitary Sewer Length within Priority 1 Program Area

Rapid I/I Reduction Program Total Costs (Rounded):

Public Rapid I/I Reduction Program Costs =	$ 16,500,000
Private Rapid I/I Reduction Program Costs =	$ 2,900,000
Rapid I/I Reduction Program Total =	**$ 19,400,000**

Clay Pipe and Manhole Rehabilitation Program Costs:

The development of rehabilitation costs for the Clay Pipe and Manhole Rehabilitation Program is based on utilizing Cured-In-Place Pipe (CIPP) to address VCP lines and a cementitious manhole liner to address brick manholes and other manholes in need of rehabilitation. CIPP was discussed at the meeting with City Staff on June 24, 2011 as an acceptable method to rehabilitate VCP lines. The utilization of cementitious lining to rehabilitate manholes is subject to City's approval as an acceptable method.

As described above, the budgetary costs for the Clay Pipe and Manhole Rehabilitation Program are based on actual inventoried quantities and corresponding unit costs. The budgetary program costs that were developed include an allowance for inspection and program management. Unit costs for CIPP were developed utilizing average bid results from City of Lawrence 2010 CIPP Sanitary Sewer Rehabilitation provided by the City. The unit cost for cementitious manhole rehabilitation was based on three previous sewer rehabilitation projects that took place from 2005 to 2010 in northeastern Kansas. A budgetary cost of $172.00 per vertical foot of manhole depth was developed for the manhole rehabilitation portion of the program. A summary of the unit costs for the CIPP portion of the program are included in Table 4.4 below, followed by a summary of the overall Clay Pipe and Manhole Rehabilitation Program costs in Table 4.5.

Table 4.4
CIPP Unit Program Costs

Pipe Diameter	Clay Pipe Length (Feet)	Project Cost Per Foot	CIPP Cost Subtotals
8"	457,057	$35.76	$16,342,168
9"	2,308	$39.97	$92,260
10"	29,606	$43.23	$1,279,843
12"	50,828	$52.60	$2,673,765
15"	25,362	$65.99	$1,673,628
18"	13,892	$95.78	$1,330,593
21"	7,901	$111.33	$879,604
24"	35,856	$144.48	$5,180,445
27"	4,132	$156.07	$644,872
30"	2,723	$167.66	$456,528
36"	0	$208.05	$0
Total Cost of CIPP for Clay Pipe Rehabilitation =			**$30,553,705**

Table 4.5
Clay Pipe and Manhole Rehabilitation Program Costs

Clay Pipe and Manhole Rehabilitation Program Total Costs (Rounded):

CIPP Costs =	$ 30,500,000
*Manhole Rehabilitation Costs (17,470 V.F. x $172/V.F.) =	$ 3,000,000
Clay Pipe and Manhole Rehabilitation Program Total =	**$ 33,500,000**

*Manhole Rehabilitation Costs calculated by Total Vertical Feet (Table TM4.3) multiplied by the Unit Budgetary Cost discussed in this section of the TM above.

E. Recommended Rehabilitation Plan

The budgetary cost for the Rapid I/I Reduction Program is **$19,400,000**. The budgetary cost for Clay Pipe and Manhole Rehabilitation Program is **$33,500,000**. The total combined cost for both programs is **$52,900,000**. The Rapid I/I Reduction Program has been divided into an 8-year phased program to be implemented concurrently with the City's ongoing CIPP efforts. Subsequent to completion of the Rapid I/I Reduction Program, the annual budget has been allocated to the Clay Pipe and Manhole Rehabilitation. The first year of the Rapid I/I Rehabilitation Program costs have been reduced to account for time to implement a SSES of the Rapid I/I Reduction Program area before rehabilitation activities begin. The recommended funding level for the 18-year phased rehabilitation plan in 2012 dollars is summarized below in Table 4.6.

Table 4.6
18-Year Phased Rehabilitation Plan

Year	Rapid I/I Reduction Program	Clay Pipe and Manhole Rehabilitation Program	Annual Costs
1	$1,800,000	$400,000	$2,200,000
2	$2,515,000	$400,000	$2,915,000
3	$2,515,000	$400,000	$2,915,000
4	$2,515,000	$400,000	$2,915,000
5	$2,515,000	$400,000	$2,915,000
6	$2,515,000	$1,000,000	$3,515,000
7	$2,515,000	$3,000,000	$5,515,000
8	$2,510,000	$1,000,000	$3,510,000
9	$0	$4,000,000	$4,000,000
10	$0	$4,000,000	$4,000,000
11	$0	$0	$0
12	$0	$3,000,000	$3,000,000
13	$0	$3,000,000	$3,000,000
14	$0	$2,500,000	$2,500,000
15	$0	$2,500,000	$2,500,000
16	$0	$2,500,000	$2,500,000
17	$0	$2,500,000	$2,500,000
18	$0	$2,500,000	$2,500,000
Total			**$52,900,000**

* * * * *

Technical Memorandum No. 5
Capital Improvements Program

Wastewater Facilities Master Plan

for

Lawrence, Kansas

City of Lawrence, Kansas

BMcD Project No. 54793

City P.O. 07629

Burns & McDonnell Engineering Company, Inc.
9400 Ward Parkway
Kansas City, MO 64114

City of Lawrence, Kansas

Wastewater Facilities Master Plan Technical Memorandum No. 5

Table of Contents

List of Tables

* = follows page number

A. Introduction

Technical Memorandum (TM) No. 5 sets forth a program and schedule for capital improvements for the City's wastewater utility as recommended by prior master plan technical memoranda. Improvements are needed for current utility service area requirements, for serving growth and development forecast to occur by years 2020 and 2030, and for addressing new regulatory actions expected to occur over the next 10 to 20 years. This TM sets forth a schedule for capital improvements based on a combination of priorities as follows:

- Improvements needed for meeting current capacity needs or regulatory requirements first, followed by those necessary for providing capacity for future growth and development.
- Implement improvements that will achieve the greatest benefit for the money spent first, followed by those having a lower benefit relative to cost.
- Implement improvements in a manner that is most affordable to the utility's rate payers.

Improvements are grouped into three justification categories – growth, regulatory, and reliability.

B. Opinions of Probable Costs

Opinions of probable project costs are based on construction and other cost allowances including contingency, engineering, surveying, legal, and other related costs and are summarized in Table 5.1. Unit cost data and component cost information for the proposed improvements are based on historical projects and vendor's cost information. Unit costs are based on an Engineering News Record Construction Cost Index (ENR-CCI) of 10,500 Kansas City, Missouri for February 2012.

Project costs include construction costs, contingencies, and other costs. Land and right-of-way acquisition costs are not included. The total includes a contingency, which varies based on the project from 20 to 25 percent, and engineering and other costs, which vary by project. Contingency covers items that are not anticipated, changes in conditions, or other factors whose costs cannot be anticipated at this level of project development.

Table 5.1
Capital Improvements Opinions of Costs Summary

By Year	Classification	Category	Reason for Improvement	Item No.	Item	Quantity	Unit	Unit Cost	Const Cost w/ Contingency	Engineering/Other Costs	Subtotal Cost	Category Total
2020	Collection System	Gravity Sewers	3	1	21" Gravity Sewer to Eliminate PS 8	8,100	LF	360	2,916,000	584,000	3,500,000	
			3	2	KR-5C 12" Relief Sewer	2,300	LF	290	667,000	133,000	800,000	
			3	3	KR-6B 24" Relief Sewer	1,730	LF	340	588,200	111,800	700,000	
			1	4	15" Baldwin Creek West of K-10, Bore (BC-2) - Brink - (2)	3,300	LF	240	792,000	158,000	950,000	
					Subtotal - Gravity Sewers (2020)							5,950,000
		Pumping Stations	1,2	1	PS 9 Expansion to 14 MGD	1	LS	1,900,000	1,900,000	400,000	2,300,000	
			1,2	2	PS 32 Expansion to 1.7 MGD	1	LS	430,000	430,000	90,000	520,000	
			1,2	3	PS 25 Expansion to 4 MGD, Add 3rd Pump	1	LS	125,000	125,000	25,000	150,000	
			1,3	4	PS 23 Expansion to 0.1 MGD	1	LS	170,000	170,000	30,000	200,000	
					Subtotal - Pumping Stations (2020)							3,170,000
		Force Mains	1,2	1	PS 32 - 8" Force Main	3,600	LF	65	234,000	46,000	280,000	
					Subtotal - Force Mains (2020)							280,000
					Subtotal - Collection System (2020)							9,400,000
	New 2 MGD Wakarusa WWTP	Plant	1,2	1	Wastewater Treatment Plant	1	LS	25,000,000	25,000,000	5,000,000	30,000,000	
			1,2	2	Peak Flow Storage	1	LS	5,000,000	5,000,000	1,000,000	6,000,000	
			1,2	3	Roads, Utilities	1	LS	5,000,000	5,000,000	1,000,000	6,000,000	
					Subtotal - Plant (2020)							42,000,000
		Pumping Stations	1,2	1	New (Wakarusa) 11 MGD PS 5C	1	LS	4,330,000	4,330,000	860,000	5,190,000	
					Subtotal - Pumping Stations (2020)							5,190,000
		Force Mains	1,2	1	2 - 16" Force Mains	27,000	LF	240	6,480,000	1,030,000	7,510,000	
					Subtotal - Force Mains (2020)							7,510,000
					Subtotal - New 2 MGD Wakarusa WWTP (2020)							54,700,000
	Kansas River WWTP	Plant	3	1	Co-generation and Backup Power	1	LS	830,000	830,000	170,000	1,000,000	
					Subtotal - Kansas River WWTP (2020)							1,000,000
	Collection System Rehabilitation Plan	Gravity Sewers	2,3	1	Rapid I/I Reduction Program	8	Year	2,000,000	16,000,000	3,400,000	19,400,000	
			2,3	2	Clay Pipe and Manhole Rehabilitation Program	8	Year	750,000	6,000,000	1,000,000	7,000,000	
					Subtotal - Collection System Rehabilitation Plan (2020)							26,400,000
	Annual Maintenance	Plant	3	1	Annual Maintenance	8	Year	300,000	2,400,000	-	2,400,000	
		Pumping Stations	3	2	Annual Maintenance	8	Year	100,000	800,000	-	800,000	
		Gravity Sewers	1	3	Sewer Main Relocations for Road Projects	8	Year	260,000	2,080,000	320,000	2,400,000	
					Subtotal - Annual Maintenance (2020)							5,600,000
	Collection System Growth Related Projects	Baldwin Creek N of I-70 (BC-3)	1	1	8" Gravity Sewer, Bore	2,400	LF	220	528,000	102,000	630,000	
			1	2	10" Gravity Sewer, Bore	2,400	LF	240	576,000	114,000	690,000	
			1	3	15" Gravity Sewer, Bore	2,400	LF	295	708,000	142,000	850,000	
			1	4	21" Gravity Sewer	6,600	LF	205	1,353,000	277,000	1,630,000	
					Subtotal - Baldwin Creek N of I-70 (BC-3)							3,800,000
		Wakarusa US 59 & 1100 Rd (WRS-5)	1	1	36" Gravity Sewer	5,700	LF	360	2,052,000	428,000	2,480,000	
			1	2	1.0 MGD Lift Station	1	LS	825,000	825,000	165,000	990,000	
			1	3	8" Force Main	400	LF	65	26,000	4,000	30,000	
					Subtotal - Wakarusa US 59 & 1100 Rd (WRS-5)							3,500,000
					Subtotal - Collection System Growth Related Projects (2020)							7,300,000
2030	Collection System	Pumping Stations	1	1	PS 48 Expansion to 6.4 MGD	1	LS	250,000	250,000	50,000	300,000	
					Subtotal - Pumping Stations (2030)							300,000
		Force Mains	1,2	1	PS 25 Parallel 12" Force Main	13,800	LF	90	1,242,000	198,000	1,440,000	
					Subtotal - Force Mains (2030)							1,440,000
		Building	3	1	Collection System Field Operations Building	1	LS	3,330,000	3,330,000	670,000	4,000,000	
					Subtotal - Building (2030)							4,000,000
					Subtotal - Collection System (2030)							5,740,000
	Kansas River WWTP	Plant	2	1	Kansas River WWTP Nutrient Removal	1	LS	7,500,000	7,500,000	1,500,000	9,000,000	
					Subtotal - Kansas River WWTP (2030)							9,000,000
	Collection System Rehabilitation Plan	Gravity Sewers	2,3	1	Clay Pipe and Manhole Rehabilitation Program	10	Year	2,300,000	23,000,000	3,500,000	26,500,000	
					Subtotal - Collection System Rehabilitation Plan (2030)							26,500,000
	Annual Maintenance	Plant	3	1	Annual Maintenance	10	Year	600,000	6,000,000	-	6,000,000	
		Pumping Stations	3	2	Annual Maintenance	10	Year	100,000	1,000,000	-	1,000,000	
		Gravity Sewers	1	3	Sewer Main Relocations for Road Projects	10	Year	260,000	2,600,000	400,000	3,000,000	
					Subtotal - Annual Maintenance (2030)							10,000,000
	Collection System Growth Related Projects	Baldwin Creek N of I-70 (BC-1A)	1	1	8" Gravity Sewer	8,800	LF	155	1,364,000	246,000	1,610,000	
				2	0.5 MGD Lift Station	1	LS	625,000	625,000	125,000	750,000	
				3	6" Force Main	4,400	LF	45	198,000	42,000	240,000	
					Subtotal - Baldwin Creek N of I-70 (BC-1A)							2,600,000
		Baldwin Creek N of I-70 (BC-3A)	1	1	0.5 MGD Lift Station	1	LS	625000	625,000	125000	750,000	
				2	6" Force Main	4,800	LF	45	216,000	34000	250,000	
					Subtotal - Baldwin Creek N of I-70 (BC-3A)							1,000,000
		Wakarusa US 59 & 1100 Rd (WRS-3, 5 &6)	1	1	8" Gravity Sewer	3,500	LF	155	542,500	87500	630,000	
				2	15" Gravity Sewer	2,200	LF	200	440,000	90000	530,000	
				3	21" Gravity Sewer, Bore	2,300	LF	305	701,500	138500	840,000	
					Subtotal - Wakarusa US 59 & 1100 Rd (WRS-3, 5 & 6)							2,000,000
					Subtotal - Collection System Growth Related Projects (2030)							5,600,000
					Subtotal - 2020 Existing System Improvements							97,100,000
					Subtotal - 2030 Existing System Improvements							51,240,000
					Subtotal - All Existing System Improvements							148,340,000
					Subtotal - 2020 Collection System Growth Related Projects							7,300,000
					Subtotal - 2030 Collection System Growth Related Projects							5,600,000
					Subtotal - All Collection System Growth Related Projects							12,900,000
					GRAND TOTAL - ALL CAPITAL IMPROVEMENTS							161,240,000

Reason for Improvement
1 - Growth
2 - Regulatory
3 - Reliability

Engineering and other costs account for technical, professional and special services that are required to execute the project. These include environmental, technical, and geotechnical studies; land and right-of-way appraisals and negotiations, design and resident engineering fees, construction material testing, legal fees, project insurance, land surveying and legal descriptions, project design surveying, operation and maintenance manuals, and personnel training. Land and right-of-way costs for each improvement are not included in the cost opinions.

These order-of-magnitude cost opinions are based on experience and judgment as a professional consultant combined with information from past experience, vendors, and published sources such as RSMeans Construction Cost Data. Since Burns & McDonnell has no control over numerous factors which can affect the cost and pricing of construction work, economic conditions, government regulations and laws, competitive bidding or market conditions and other factors affecting such opinions or projection, Burns & McDonnell does not guarantee the actual rates, costs, etc. will not vary from the opinions and projections developed herein.

C. Capital Improvements Program – Existing System Improvements

1. Description

Recommended existing system capital improvements and the year they are planned to occur are summarized in Table 5.2 through year 2030. The program presents the improvements by the following categories:

1. Existing Collection System Improvements
2. Existing Collection System Rehabilitation
3. New Wakarusa Wastewater Treatment Plant
4. Kansas River Wastewater Treatment Plant Improvements
5. Annual Wastewater Utility Maintenance

The improvements included in each of these categories are described below.

Table 5.2

Capital Improvements Program Summary - Existing System Improvements

Item	Reason for Improvement			2012 Cost Opinion	2013 (1)	2014 (1)	2015 (1)	2016 (1)	2017 (1)	2018 (1)	2019 (1)	2020 (1)	2021 (1)	2022 (1)	2023 (1)	2024 (1)	2025 (1)	5 Year Period Ending 2030 (1)
1 Collection System**																		
a PS 9 expansion to 14 MGD	1, 2			$ 2,300,000								$ 3,147,700						
b PS 32 expansion to 1.7 MGD, 8" force main	1, 2			$ 800,000	$ 832,000													
c PS 25 expansion to 4.4 MGD, Add 3rd Pump	1, 2			$ 150,000							$ 197,400							
d PS 25 expansion to 6 MGD, parallel 12" force main	1, 2			$ 1,440,000														$ 2,917,200
e 21" gravity sewer to eliminate PS 8	3			$ 3,500,000					$ 425,800	$ 4,002,800								
f KR-5C 12" relief sewer	3			$ 800,000					$ 973,300									
g KR-6B 24" relief sewer	3			$ 700,000						$ 885,700								
h PS 23 expansion to 0.1 MGD	1, 3			$ 200,000	$ 208,000													
i PS 48 expansion to 6.4 MGD	1			$ 300,000												$ 480,300		
j Baldwin Creek West of K-10 (BC-2) - Brink - (2)	1			$ 950,000	$ 988,000													
k Collection System Field Operations Building	3			$ 4,000,000									$ 5,693,200					
Subtotal				$ 15,140,000														
2 New 2 MGD Capacity Wakarusa WWTP																		
a Wastewater Treatment Plant	1, 2			$ 30,000,000	$ 2,184,000	$ 6,489,600	$ 10,686,200	$ 11,698,600	$ 2,920,000									
b Peak Flow Storage	1, 2			$ 6,000,000	$ 499,200		$ 2,249,700	$ 3,509,600	$ 632,700									
c Roads, Utilities	1, 2			$ 6,000,000	$ 499,200	$ 2,995,200	$ 3,125,000											
d New (Wakarusa) PS 5C, 2 - 16" force mains	1, 2			$ 12,700,000	$ 924,600	$ 5,408,000	$ 5,624,300	$ 2,118,600										
Subtotal				$ 54,700,000														
3 Kansas River WWTP																		
a Nutrient Removal	2			$ 9,000,000											$ 13,855,100			
b Co-generation & Backup Power	3			$ 1,000,000	$ 600,000	$ 481,600												
Subtotal				$ 10,000,000														
4 Collection System Rehabilitation Plan																		
a Rapid I/I Reduction Program	2, 3			$ 19,400,000	$ 1,872,000	$ 2,720,200	$ 2,829,000	$ 2,942,200	$ 3,059,900	$ 3,182,300	$ 3,309,600	$ 3,442,000						
b Clay Pipe and Manhole Rehabilitation Program	2, 3			$ 33,500,000	$ 416,000	$ 432,600	$ 449,900	$ 467,900	$ 486,700	$ 1,265,300	$ 3,947,800	$ 1,368,600	$ 5,693,200	$ 5,921,000	$ -	$ 4,803,100	$ 4,995,200	$ 25,322,700
Subtotal				$ 52,900,000														
5 Annual Maintenance																		
a Wastewater Treatment Plant: 2013 - 2020	3	300,000	8	$ 2,400,000	$ 300,000	$ 312,000	$ 324,500	$ 337,500	$ 351,000	$ 365,000	$ 379,600	$ 394,800						
b Wastewater Treatment Plant - 2 Plants: 2021 - 2030	3	600,000	10	$ 6,000,000									$ 600,000	$ 624,000	$ 649,000	$ 674,900	$ 701,900	$ 3,953,900
c Pump Stations: 2013 - 2030	3	100,000	18	$ 1,800,000	$ 100,000	$ 104,000	$ 108,200	$ 112,500	$ 117,000	$ 121,700	$ 126,500	$ 131,600	$ 136,900	$ 142,300	$ 148,000	$ 153,900	$ 160,100	$ 901,800
d Sewer Main Relocations for Road Projects: 2013 - 2030	1	300,000	18	$ 5,400,000	$ 300,000	$ 312,000	$ 324,500	$ 337,500	$ 351,000	$ 365,000	$ 379,600	$ 394,800	$ 410,600	$ 427,000	$ 444,100	$ 461,800	$ 480,300	$ 2,705,400
Subtotal				$ 15,600,000														
Total				**$ 148,340,000**	$ 9,723,000	$ 19,255,200	$ 25,721,300	$ 21,524,400	$ 9,317,400	$ 10,187,800	$ 8,340,500	$ 8,879,500	$ 12,533,900	$ 7,114,300	$ 15,096,200	$ 6,574,000	$ 6,337,500	$ 35,801,000

(1) - 4% Inflation Used to Calculate 2013 to 2030 Costs

(2) - Cost allocation between property owners, developers and the City to serve Future Growth Areas has not been determined.

** Development Related Growth Projects Are Not Included in CIP

2013 - 2020 Total	**$ 112,949,100**

Reason for Improvement

1- Growth

2 - Regulatory

3 - Reliability

2. Existing Collection System Improvements

a. Expand Pumping Station No. 9

Pumping Station No. 9 requires expansion from 8.6 MGD to 15 MGD to accommodate existing wet weather peak flows and projected upstream growth and development. A portion of this expansion is necessary now in order to provide firm pumping capacity for the 10 year design storm event. The expansion could be done in stages, but it is considered to be more cost effective to complete the full expansion at one time. The existing structure and piping is designed to accommodate two more pumps. It may also be necessary to replace existing pumps to provide the needed capacity. The additional pumps and other station improvements should be configured to provide flexibility for pumping all flows east to the downstream collection system during dry weather periods, and pumping varying portions of wet weather peak flows east to the downstream collection system and to the existing wet weather peak flow storage basins. The division of capacities needs to be approximately 5 MGD east to the downstream collection system, and 10 MGD to the peak flow storage basins.

b. Expand Pumping Station No. 23

The existing system model predicts the 10 year design storm wet weather peak flow to Pumping Station No. 23 equals its firm pumping capacity. Pumping capacity will need to be increased to accommodate growth through 2030. Future development and flows tributary to this pumping station should be monitored and necessary expansions be done as dictated by actual development. Additional firm capacity for this station beyond what is forecast for 2030 should be considered when it is expanded.

c. Expand Pumping Station No. 32 and New Force Main

The existing system model shows the Pumping Station No. 32 existing firm capacity of 0.7 MGD is exceeded by the 10 year design storm wet weather peak flow rate and requires expansion. The design storm peak flow rate in 2030 to this pumping station is forecast to be 1.7 MGD. Expansion of Pumping Station No. 32 firm capacity to 1.7 MGD is recommended, which will also require installation of a parallel 8-inch force main to provide the necessary peak flow capacity.

d. KR-5C and KR-6B Relief Sewers

Gravity sewer surcharging remains at two locations following the Rapid I/I Reduction Program within the program target area due to inadequate flow capacities for conveying the 10 year design storm peak flow rate. In these instances, parallel gravity relief sewers are recommended to provide the additional peak flow capacity needed to convey the design storm peak flow rate. The 12-inch gravity sewer in Drainage Sub-basin KR-5C requires a 12-inch parallel relief sewer. The 21-inch gravity sewer in Drainage Sub-basin KR-6B requires a 24-inch parallel relief sewer. Recommended relief sewer sizes are based on 10 year design storm peak flow reductions that will occur as a result of implementing the Rapid I/I Reduction Program as explained below. As such, it is recommended that relief sewer construction occur after implementing the Rapid I/I Reduction Program.

e. Expand Pumping Station No. 25 and New Parallel Force Main

The 2020 system model predicts the 10 year design storm wet weather peak flow to this pumping station will exceed its firm capacity. As such, an initial expansion of Pumping Station No. 25 by 2020 to 4.4 MGD firm capacity by addition of a third pump is recommended. A further expansion of Pumping Station No. 25 to 6 MGD by the addition of a second, parallel 12-inch diameter force main is recommended by 2030. The diversion of Pumping Station No. 49 flows, which are now conveyed to Pumping Station No. 25, to the future WWWTP can be deferred until sometime after 2030. Actual development should be examined at the time it becomes necessary to expand Pumping Station No. 25 to confirm that is still appropriate to do so, or if actual growth and development in the East Lawrence Drainage Basin would instead dictate diverting Pumping Station No. 49 flows to the WWWTP.

f. Eliminate Pumping Station No. 8

Plans have been in place to eliminate Pumping Station No. 8 due to its age and condition and need for additional capacity. As such, a 21-inch diameter gravity sewer intercepting flows into Pumping Station No. 8 and conveying them south to the interceptor sewer tributary to new Pumping Station No. 5C (see below) is recommended. The recommended gravity sewer size is based on 10 year design storm peak flow reductions that will occur as a result of implementing the Rapid I/I Reduction Program as explained below. As such, it is recommended that elimination of Pumping Station No. 8 occur after implementing the Rapid I/I Reduction Program.

g. Expand Pumping Station No. 48

The 2030 system model predicts 2030 wet weather peak flows to this pumping station will marginally exceed its existing firm capacity. As such, future development and flows tributary to this pumping station should be monitored and necessary expansion be done as dictated by actual development. Additional firm capacity for this station beyond what is forecast for 2030 should be considered at the time it needs to be expanded.

3. Existing Collection System Rehabilitation

a. Rapid I/I Reduction Program

A Rapid I/I Reduction Program is recommended to achieve a targeted amount of I/I reduction for both public and private sources located in close proximity to the KRWWTP. The objective the Rapid I/I Reduction Program is an overall 35% reduction of I/I within the program area. In order for the public portion of the Rapid I/I Reduction Program to effectively achieve the target I/I reduction, a Sewer System Evaluation Survey (SSES) should be completed prior to rehabilitation work to identify and evaluate public sources of rapid I/I. The overall 35% reduction objective would be achieved in conjunction with the private portion of the Rapid I/I Reduction Program. The private portion of the program requires building inspections and program management to identify and eliminate private I/I sources. Once the public and private programs have been implemented, monitoring programs should be put into place to measure I/I reductions concurrently as rehabilitation progresses. Based on results of the monitoring programs, the scope and timeline of the Rapid I/I Reduction Program can be adjusted as required to achieve the targeted rapid I/I reduction objective(s).

b. Clay Pipe and Manhole Rehabilitation Program

Rehabilitation of the remainder of the collection system will be addressed in a Clay Pipe and Manhole Rehabilitation Program. The phasing and funding level of the Clay Pipe and Manhole Rehabilitation Program is based on continuing the City's current efforts to rehabilitate VCP lines concurrently with the higher priority Rapid I/I Reduction Program. Subsequent to reaching the reduction target of the Rapid I/I Reduction Program, the Clay Pipe and Manhole Rehabilitation Program would be ramped up to be completed within the desired timeline for both programs. The Clay Pipe and Manhole Rehabilitation Program would address all of the remaining VCP sewers, brick manholes and other manholes in need of rehabilitation throughout the system, including remaining sewers and manholes not addressed by the Rapid I/I Reduction Program within that program area.

4. Wakarusa Wastewater Treatment Plant

a. New Pumping Station No. 5C and Force Mains

A new Pumping Station No. 5C with a firm capacity of 11 MGD is recommended to provide sufficient peak flow capacity through year 2030. This same pumping station will also serve to divert dry weather flows to the new WWWTP. A location near the northwest intersection of 31st and Louisiana Streets would be preferred location for this pumping station. The new pumping station and force mains should be constructed and placed into service at the same time the future WWWTP is placed into service.

The force main from this pumping station will be routed west and then south and east to the future Wakarusa WWTP site. The range of dry and wet weather flows to be handled by this pumping station is wide, from as little as 1 to 3 MGD during dry weather periods up to the 11 MGD peak flow rate. As such, a dual force main is proposed, with one force main in service during dry weather periods, and both in service during peak wet weather flow conditions. Two 16-inch diameter force mains are recommended to provide sufficient flow velocity during dry weather flows when one force main will be in service.

b. New Wakarusa WWTP, Peak Flow Storage, and Support Systems

TM-3 concluded the future WWWTP should be constructed and in service by the time the service area population reaches 103,000 which is forecast to occur in 2018. A minimum initial treatment capacity of 2 MGD or more is recommended. A larger initial capacity may be appropriate given that the future WWWTP is expected to be put into service in 2018, and with a 2 MGD capacity would nearly be operating at its capacity 12 years later in 2030 based on the population forecast used for this plan. The final selection of treatment capacity remains to be determined by further planning for the WWWTP and will be based on costs and other factors concerning the most appropriate initial treatment capacity. The cost opinion included in this capital improvements program is based on 2 MGD capacity.

With a 2 MGD annual average daily flow capacity, the WWWTP could readily be designed to fully treat wet weather peak flow rates up to approximately 6 MGD. This will not be sufficient peak flow rate capacity for the Pumping Station No. 5C required firm pumping capacity of 11 MGD. As such, flows received at the WWWTP in excess of its peak flow capacity will need to be stored and then fully treated after flow rates return to less than its peak flow treatment capacity. The storage volume needed for the 10 year design storm event is estimated to be 4 MG.

The new WWWTP will require supporting systems such as utilities (water, electricity, natural gas) and road improvements to handle anticipated vehicular traffic related to plant operations. A cost opinion for these support systems is included in the capital improvements program.

5. Kansas River Wastewater Treatment Plant Improvements

At a meeting with KDHE arranged to discuss regulatory actions that may affect this master plan, KDHE informed the City that new effluent limits for nutrients (total nitrogen and phosphorus) should be anticipated at the time of the second 5 year renewal of the KRWWTP discharge permit. This would occur at approximately year 2020 with a compliance deadline likely to occur three years thereafter. This will require significant improvements to the KRWWTP as previously documented by others. A cost opinion for these improvements based on previous planning is included in the capital improvements program.

Other capital improvements at the KRWWTP are anticipated including new co-generation and back-up power systems. A cost opinion provided by City staff for these facilities is included in the capital improvements program.

6. Annual Wastewater Utility Maintenance

Various utility maintenance activities are required for reasons of reliability and in support of other City utility and road projects. They include replacement of mechanical equipment such as pumps and motors at lift stations and treatment plants. Relocations of sewers and force mains for road projects are also included. The capital improvements program includes cost opinions provided by City staff for these types of maintenance activities.

D. Capital Improvements Program – Service to Future Growth Areas

Some extensions of the existing collection system are needed to provide service to the future growth areas forecast to occur by years 2020 and 2030. Capital improvements and the year they are planned to occur are summarized in Table 5.3 through year 2030. Extensions are included for the following future development areas:

- Baldwin Creek West of K-10 (BC-2)

Table 5.3

Capital Improvements Program Summary - Service to Future Growth Areas

Item	Reason for Improvement	2012 Cost Opinion	2013 (1)	2014 (1)	2015 (1)	2016 (1)	2017 (1)	2018 (1)	2019 (1)	2020 (1)	2021 (1)	2022 (1)	2023 (1)	2024 (1)	2025 (1)	5 Year Period Ending 2030 (1)
1 Collection System Growth Related Projects**																
a Baldwin Creek North of I-70 (BC-3) - (2)	1	$ 3,800,000						$ 4,808,200								
b Wakarusa US 59 & 1100 Road (WR-5) - (2)	1	$ 3,500,000							$ 4,605,800							
c Baldwin Creek North of I-70 (BC-1A) - (2)	1	$ 2,600,000														$ 5,267,100
d Baldwin Creek North of I-70 (BC-3A) - (2)	1	$ 1,000,000														$ 2,025,800
e Wakarusa US 59 & 1100 Road (WR-3, 5, & 6) - (2)	1	$ 2,000,000													$ 3,330,100	
Subtotal		$ 12,900,000														
Total		**$ 12,900,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 4,808,200**	**$ 4,605,800**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 3,330,100**	**$ 7,292,900**

(1) - 4% Inflation Used to Calculate 2013 to 2030 Costs
(2) - Cost allocation between property owners, developers and the City to serve Future Growth Areas has not been determined.
** Development Related Growth Projects Are Not Included in CIP

Reason for Improvement
1- Growth
2 - Regulatory
3 - Reliability

- Baldwin Creek North of I-70 (BC-3)
- Wakarusa River South – US 59 & 1100 Road (WRS-5)
- Baldwin Creek North of I-70 (BC-1A)
- Baldwin Creek North of I-70 (BC-3A)
- Wakarusa River South – US 59 & 1100 Road (WRS-3, 5 & 6)

The collection system extensions to future growth areas are based on a conceptual level of development and representative of one possible development alternative that is subject to change. Actual pipe sizes, alignment, and schedule will be determined as development occurs. The cost allocation between property owners, developers, and the City to serve future development areas has not been determined.

* * * * *