Kentucky American Water

Lexington Service Area

Comprehensive Planning Study - 2013

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Comprehensive Planning Study

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INTRODUCTION

This Comprehensive Planning Study (CPS) details the capital improvement recommendations (found in **Table E-1)** for Kentucky American Water's water system for projection years 2013 through 2030. This plan presents a strategy for facility improvements to ensure that Kentucky American Water (KAW) can continue to provide safe, adequate and reliable service to its customers.

The service area is divided into two separate systems. The Central System serves approximately 120,000 customers and accounts for nearly 98% of the total water demand in Fayette, Bourbon, Clark, Harrison, Jessamine, Scott and Woodford counties, while the Northern System serves approximately 3,882 customers and accounts for the remaining 2% of demand in Owen, Grant, Gallatin and Franklin counties.

The Central System is currently serviced by the three water treatment plants: Kentucky River Station I ("KRS-1"), Richmond Road Station ("RRS"), and Kentucky River Station II ("KRS-2"). The Northern System is currently serviced by the Owenton Water Treatment Plant, however a pipeline to connect the Northern District to KRS-2 is under construction, with plans to take the Owenton Water Treatment Plant off line in early 2014. The Central and Northern Systems include the following characteristics as shown in **Table E-2**:

Table E-2
System Characteristics

Description	Central System	Northern System
Total Service Connections	120,000	3850
Avg day demand 2006 - 2011 (mgd)	40.5	0.860
Max day demand 2006 - 2011 (mgd)	67.2	1.175 7/23/2011
Min day demand 2006 - 2011 (mgd)	27.4	0.431 or 0.454 3/3/2012 or 4/3/2006
Number of pressure gradients	1	(--)
Number of Storage Tanks and Reservoirs	18	11
Number of Pump stations.	15	4
Distribution Main (miles)	1,848	270
Number of Wells	0	0

Current criteria for the three water treatment plants are listed in **Table E-3**:

Table E-3 Water Treatment Plants			
Description	**KRS-1**	**RRS**	**KRS-2**
Overall Rated Capacity (mgd)	45 mgd(1)	25 mgd	20 mgd
Source of Supply	Kentucky River Pool 9	Kentucky River Pool 9 & Jacobson Reservoir	Kentucky River Pool 3
Permitted Allocation	30 to 63 mgd(2)	16 mgd(3)	20 mgd (June – Aug) 6 mgd (Sept – May)
Type of Treatment	Conventional Treatment via Aldrich Units	Conventional Treatment	Conventional Treatment

(1) KRS I has a temporary rated capacity of 45 mgd dependent on raw water quality
(2) Dependent upon river level.
(3) RRS obtains its water from Pool 9 and Jacobson Reservoir. Jacobson Reservoir has a permitted allocation of 16 mgd.

CPS PROCESS

This CPS was conducted in four steps. First, a system-wide evaluation of the District's water systems was conducted using American Water's Standard Planning Criteria for analysis of the water systems. The American Water Planning Criteria, which is enclosed as **Appendix A**, provides review, analysis guidance, and methodology for the following key areas:

- Engineering Criteria
    - Water demand projections
    - Source of supply analysis
    - Source water quality and watershed protection
    - Treatment facility evaluation
    - Electrical service and standby power evaluations
    - Chemical feed, storage and containment evaluations

- Distribution Piping, Pumping, and Storage Evaluation Criteria
    - Distribution system analysis
    - Distribution system computer modeling
    - Distribution system main replacement program evolutions

Also, the water facility improvement and upgrade needs, which were identified by the prior steps were subject to an alternative analysis and prioritization process. The alternative analysis process identified the available options for satisfying each of the identified needs and recommended a solution that cost-effectively met the requirements of the given needs. Next, each of the identified solutions was prioritized utilizing several factors including the ranking of the given solution relative to system safety, reliability, and regulatory compliance.

Finally, KAW's system was reviewed for potential opportunities to improve energy efficiency and reduce carbon footprint as part of separate study conducted in parallel with the CPS. The highlights of the energy study are included in the CPS and capital investment projects that would reduce energy costs and/or energy consumption and carbon footprint are included.

The CPS recommendations are presented in **Section 1** of the CPS. **Section 2** provides a summary of the American Water Comprehensive Planning Process. **Section 3** addresses KAW projected water demands. **Section 4** evaluates water supply. **Section 5** assesses the condition of each water facility from a production perspective. **Section 6** address distribution system issues and **Section 7** summarizes the highlights of the energy optimization evaluation.

Supplemental information, such as the Planning Criteria and Regulations, is included in **Appendix A**. Detailed cost estimates are included in **Appendix B**. Site and Plant photos are included in **Appendix C**. The pump efficiency recommendation for KAW is located in **Appendix D**. The Hydraulic analysis is included in **Appendix E**.

FINDINGS AND CONCLUSIONS

Demand Projections - The KAW Central System has experienced slow to modest rate of growth. The Kentucky American Water's primary service area lies mostly in Fayette County, Kentucky. Nearly all of Fayette County is served by KAW. In addition, service is provided to parts of six of the seven surrounding counties. Bourbon, Clark, Harrison, Scott, Woodford and Jessamine Counties are supplied in part by Kentucky American Water, either directly to customers or indirectly through sales to other water utilities. The Counties are projected to have a moderate growth rate from 2015 through 2030, similar to historical growth, except Jessamine and Scott Counties, which are expected to show a higher level of growth of 34% and 65%, respectively. Jessamine and Scott Counties are located approximately 10 miles south and north of Central Lexington, respectively. The main service area in Fayette County has a 14% projected growth rate over the planning period.

Continued growth, although at rates less than those in the past, is expected in the residential and commercial water use categories, which can be related directly to population changes. Industrial growth is anticipated to be maintained near current levels as the remaining industrial customers are expected to post gradual small water use increases in those cases where production is expanded. Industrial water conservation and reuse practices are already fairly well implemented but will continued by most of the large industrial users through prior efforts to reduce municipal sewer charges, which are based on water usage.

The Northern System had an average day demand of approximately 1.0 mgd and a maximum day demand of approximately 1.3 mgd in 2010. Since the system was acquired in 2005, an increase in demands has been observed throughout the Northern System. It is assumed that the 2025 average day and maximum day demands for the Northern System will increase to 1.3 mgd and 2.0 mgd, respectively.

In summary, water needs are expected to moderately grow, reflecting the population trends but tempered by more conservation effects as the use of low-flow and high efficiency fixtures continue and as customers learn to become more efficient in their water use. The demand projections ranges for the planning period are listed in **Table E-4.**

Table E-4
Historic and Projected Demand Summary

Demand Scenario	Historic				Projected			
	All Time[1]	2000	2005	2010	2015	2020	2025	2030
Normal Weather								
Average Day Demand - Central Division		41.02	44.30	40.73	41.22	43.19	44.68	46.19
Average Day Demand - Northern Division		N/A	N/A	1.00	1.10	1.20	1.30	1.30
Average Day Demand - Total		41.02	44.30	41.73	42.32	44.39	45.98	47.49
Maximum Day Demand - Central Division		66.37	69.65	61.36	73.33	76.03	79.17	82.44
Maximum Day Demand - Northern Division	71.82	N/A	N/A	1.50	1.67	1.83	2.00	2.00
Maximum Day Demand - Total		66.37	69.65	62.86	74.99	77.87	81.17	84.44
Hot, Dry Scenario								
Average Day Demand - Central Division		-	-	-	44.31	46.02	48.01	50.08
Average Day Demand - Northern Division		-	-	-	1.17	1.27	1.38	1.38
Average Day Demand - Total		-	-	-	45.48	47.29	49.39	51.46
Maximum Day Demand - Central Division		-	-	-	77.85	80.67	83.94	87.35
Maximum Day Demand - Northern Division		-	-	-	1.77	1.94	2.12	2.12
Maximum Day Demand - Total		-	-	-	79.62	82.61	86.06	89.47

(1) All time high Aug 5, 2002.

Water Supply - Raw water for the Kentucky American-Central System is obtained from three sources: the Kentucky River, Jacobson Reservoir on East Hickman Creek, and Lake Ellerslie on West Hickman Creek. The Kentucky River is the predominant supply of raw water for the Kentucky-American system. The Kentucky River is utilized at Pool 9 and at Pool 3. About 80% of the service area's daily consumption is obtained from the river.

Raw water for the Kentucky American-Northern System can be obtained from two sources: Severn Creek near the confluence of the Kentucky River in Pool 2, and Lower Thomas Lake.

The Commonwealth of Kentucky Division of Water (DOW) administers permitted withdrawals from surface water supplies. The existing permit information is presented in **Table E-5**.

Table E-5 Allocations from Surface Water Supplies								
Source (1)	Date Last Revised	Permit No.	Allocation (mgd)	Anticipated Withdrawal (mgd)	Passing Flow Reqm't	Summer (mgd)	Winter (mgd)	Drought (mgd) (2)
Kentucky River at Pool 3	1/10/07	1572	6 - 20(3)	6 - 20	None	20	6	20
Kentucky River at Pool 9	9/17/99	200	45.0	45.0	Yes	45	60	45-30
Jacobson Reservoir	5/26/89	201	16.0	16.0	None	16	16	16
Lower Thomas Lake	8/29/06	0874	0.80 - 0.90(4)	0.80 - 0.90	None	0.80	0.9	0
Severn Creek	1/27/12	0863	1.1 - 1.2(5)	1.1 - 1.2	none	1.2	1.1	0
Current Total Supplies						82.8	87.1	65-50

(1) Not included in this table is Lake Ellerslie, which has poor water quality but is available as an emergency supply.
(2) Drought condition assumes Phase 6 worst case scenario occurring between June and August.
(3) Jan - May = 6mgd; June - Aug = 20 mgd; Sep - Dec = 6 mgd
(4) Jan - Apr = 0.80 mgd, May - Jun = 0.85 mgd; Jul - Aug = 0.090 mgd
(5) Jan - May = 1.1 mgd, Jun - Aug = 1.2 mgd

Comparison of supplies with demands presented in **Section 3** indicates that under most scenarios, supplies are adequate to meet current and projected future demands through the planning period. Only under severe droughts would limitations on allocation pose challenges in meeting the projected drought average day demands if elevated demands are left unabated. KAW has in place a comprehensive drought management plan that includes demand management measures aimed at mitigating such circumstances and reducing demands to effectively fall within allocation limits. This demand management approach in conjunction with KAW's flexibility in supply/treatment options is considered adequate and cost-effective in meeting current and projected demands for the service area. Therefore, no recommendations for additional supply or treatment capacity are needed at this time.

Production Facilities - Each of the water treatment plants was evaluated for production adequacy. A summary of the major areas is discussed below:

KRS-1 - KRS-1 is a conventional coagulation/flocculation/sedimentation/filtration plant. That was originally constructed in 1928. KRS-1 has a rated capacity of 40 mgd with temporary re-rating to 45 mgd by the DOW during optimal raw water quality periods. The filters consist of ten circular Aldrich units. Raw water is obtained from Pool 9 of the Kentucky River via six 1,250 hp pumps. Residual sludge from sludge blowdown and washwater is discharged to one of four sludge lagoons and supernatant is discharged back to the river. A review of the data and discussions with KAW staff indicated the following areas of focus:

1. Residuals
2. Chemical Storage and DBP Control
3. Pumping Modifications

Residuals Improvements - With the absence of sludge blankets, there is currently no effective flocculation in the Aldrich units. The original design included the maintenance of a sludge blanket, which served this purpose, however filter upsets were not uncommon and the operation of a sludge blanket was deemed too sensitive given the fluctuations in flow experienced at the plant. The compensating effect is more coagulant and more frequent sludge blow downs and filter backwashes - ultimately resulting in more sludge generation and an overloading of the residuals process. The risks with overloading the residuals system are overflows into a nearby creek and/or excessive turbidity discharge to the river. Adjustments to operations and added flocculation via tube settlers has been recommended as a solution, also allowing for higher loading rates.

Chemical Storage and DBP Control - KRS-1 utilizes high volumes of polyaluminum chloride (PACl) for coagulation. Under the current operation, there is not adequate PACl storage to comfortably meet maximum day demands. Also under current operations, DBP formation is increasing in the system, partially due to decreasing Total Organic Carbon (TOC) removal at the plant and also due to a presumed high chlorine demand in the system. After consideration of process alternatives, the recommendation is to consider several forms of polymer and polymer blend (including ferric), while at the same time commencing an enhanced distribution system flushing program to remove organics in the distribution network piping. It is believed that the organic removal in the pipes will decrease chlorine demand, ultimately decreasing DBP formation and improving water quality in the tanks. The alternative chemical blends that are being considered will also decrease PACl use and free up existing chemical storage. Should the new coagulant prove ineffective, additional coagulant storage will be needed.

Pumping Modifications - Pumping modifications will help with better flow matching and energy costs:

- Flow Matching - KRS-1 and RRS high system delivery is currently governed by raw water flow (KRS-1 low service pumps also deliver water directly to RRS on occasion). The low service pumps ramp up and down on constant speed motors in increments of approximately 14 mgd. In order to smooth out operations and facilitate more effective and efficient production of water, variable frequency drives are recommended for the low service and high service pumps, as well as the transfer pumps.
- High Energy Use and Demand - Energy charges can be split into the two main categories of energy use (efficiency of flow) and energy demand (rate of energy being used at any point in time). KAW has been addressing energy use as part of the Energy Use Intensity (EUI) initiative, therefore some of the recommendations from that evaluation include the replacement of inefficient pumps. Hydraulic modeling identified savings from reducing throttling which dovetails well with the above recommendations to install variable frequency drives (VFDs) and motors to the existing pumps and decrease high amplitude changes in flow. A detailed review of energy demands indicates several areas of improvement through operations alone, including load shifting and possibly utilizing VFDs for demand trimming (another benefit to VFD installation). Minimal improvements to real time demand monitoring via control room instrumentation could have significant economic benefit.

RRS – RRS was built in 1924, site of the original plant from 1885 and underwent expansions and improvements in 1937, 1988, 1992, and 2003 to bring its current rated capacity to 25 mgd. It is a conventional surface water treatment plant that consists of chemical pretreatment followed by rapid mixing, flocculation, sedimentation, and conventional filtration. Primary disinfection is achieved with free chlorine through the pretreatment processes with ammonia added at the end of the process to create chloramines for secondary disinfection in the distribution system. With the exception of the filter building, the plant is in good condition and produces high quality water that meets all drinking water standards.

The area of focus at RRS was the filter building. Filter building equipment (including valves, operators, electrical, etc.) and piping in the filter gallery are in poor condition due to corrosion. The filter pipe gallery is extremely congested, which makes working in this area difficult. Inadequate ventilation and dehumidification has accelerated the deterioration of the piping and valve actuators. Additionally, there is visible cracking of the filter walls and leaking in the filter gallery. There also appears to be leaks between the filter gallery and the chlorine contact chamber that is below the filters as there is a chlorine odor in the room. Expedited corrosion of equipment and piping could be a result of the chlorine. In addition, to the equipment and pipes, the aggressive environment has resulted in the deterioration of the concrete and structural elements in the filter gallery. Recommendations include limited improvements to address imminent structural concerns in the filter gallery, and a near term project to replace the filter building.

KRS-2 – KRS-2 was constructed and placed into service in 2010. It is a conventional surface water treatment plant with a reliable capacity of 20 mgd. The treatment facility consists of four, three stage flocculators, four sedimentation basins, and five concrete box filters. The raw water and high service pump stations each consist of four pumps with a reliable capacity 24 mgd. The water supply is obtained from Kentucky River Pool 3. No issues were identified in the study due to the fact that it was recently built. Therefore, no improvements are recommended at this time.

Distribution System and Standby Power - The distribution system section includes an analysis of storage adequacy, backup power and distribution system pressure and water quality.

Distribution System Storage – The Central System consists of sixteen treated water storage tanks (2 standpipes, 8 ground tanks, and 6 elevated tanks) with a total storage capacity of approximately 25.20 MG. The Northern System consists of eleven treated water storage tanks (9 standpipes and 2 elevated tanks) with a total storage capacity of approximately 1.74 MG. . The Northern System needs to be evaluated for optimal utilization of storage.

A system is required to meet the equalization storage volume and fire flow reserve needs per American Water standards. The equalization storage volume is assumed to be 15% of maximum day demands. The fire flow reserve need is based on providing up to a maximum of 3,500 gpm for up to three hours, a calculated volume of 0.63 MG. Based on this criteria, the Central System has adequate storage provided throughout the planning period. Capital improvements are underway to serve the Northern System from KRS-2. Improvements include two new elevated storage tanks (one 300,000 gallons and one 600,000 gallons) and a 2 MG booster station. Once improvements are completed, the pressure zones that are currently connected to the Owenton pressure zone will have adequate storage, including Owenton, Monterey, Rockdale/New Columbus, Bromley, and Sparta. The Carroll Co./Wheatley and Glencoe pressure zones have adequate storage via Carroll Count Water Department and Gallatin County Water Department interconnections, respectively.

Distribution System Emergency Supply – As agreed between the Public Service Commission (PSC) and KAW, KAW is required to maintain a storage capacity equal to 50% of the average day demand and standby distributive pumping capacity equal to 50% of the average day demand in the Central Division. The Northern Division is required to comply with the original requirement of Title 807 Chapter 5 – Utilities, Section 4 – Continuity of Service, paragraph (4) where "the minimum storage capacity for systems shall be equal to the average daily consumption."

The Central Division has a storage capacity of 25.2 MG and a backup distributive pumping capacity of 26.5 MG, both of which exceed the 50% average day demand of 20.37 MG in 2010, and will satisfy the 50% average day demand of 23.10 MG in 2030 (the highest demands in the planning period). In addition, the Northern Division has adequate storage in each pressure zone to meet the required 100% average day demand in storage capacity for the entire planning period (this does not include the new 300,000 gallon and 600,000 gallon tank)

Distribution System Water Quality and Pressure **-** KAW operations staff worked closely with the hydraulic modeling effort in helping to identify areas of concern in the distribution system. Two predominant issues include areas of poor water quality due to elevated water age, or areas of low pressure during peak demands.

Central System - Water quality issues were identified at several locations throughout the Central Division under both current and future day conditions. Modeling corroborated the water quality issues via model runs that indicated high water age. Two primary areas of concern are the Briar Hill area and the Sadieville Tank areas, which both exhibited nitrification. Project recommendations are provided for both of these areas and for the distribution system in general as a means of addressing system chlorine demand and nitrification.

Projects were also identified to address the development of low pressures in the system under peak demand conditions. Under such conditions, analysis has shown that pressures can fall to below the Public Service Commission (PSC) standard of 30 psi in some locations. Several pipeline projects are recommended for mitigating these conditions. The CPS also includes several distribution pipeline projects to coordinate with an anticipated Kentucky Department of Transportation project to expand US-25, in conjunction with future population increases in that area. Such coordination will take advantage of construction efficiency opportunities and result in overall cost effective pipeline project during highway construction.

<u>Northern System</u> - Key issues in the Northern System include low pressures around and north of the New Owenton 0.6 MG tank. A project has been developed to address this. Also, the area around Fairgrounds Tank has been identified as a priority after the KRS-2 connection is completed. This project will allow the Fairgrounds tank to be taken offline for maintenance and address pressure and tank filling issues resulting from the KRS-2 connection. Additional future modeling will be completed to observe current and future pressures based on the Northern System being served by KRS-2.

Energy Evaluation and Plant Optimization - KAW's system was evaluated with the purpose of optimizing the delivery of water by saving energy and operational costs. A separate report was developed in parallel with this CPS (*Kentucky American Water Energy Optimization, January 2013)*. Highlights from the document are summarized below:

- KAW has over 100 separately billed accounts. Bills were reviewed for the period from June 2011 through July 2012, including 15-minute demand interval data.
- A review of the data indicated that the following large pumps (i.e. "major pumps") consumed approximately 86% of KAW's overall energy for the duration of the period from July 2011 through June 2012:

- Transfer Pumps
- Jacobson Pumps
- RRS High Service Pumps
- KRS-2 Low Service Pumps
- KRS-2 High Service Pumps
- Woodlake Booster Pumps
- Owenton High Service Pumps
- Severn Low Service Pumps

- Total electric billings, including energy plus demand, rolled up to $3.7M over that time period, and approximately $3.2M for the "Major Pumps" category.
- A breakdown of pure energy use and demand use (not including other charges on the bill) indicated that approximately 67% of total energy cost was from Energy Use (kWh) and 33% due to Energy Demand (kW). When other charges are factored into the entirety of the bill, they represent 58% and 28% of the bill, respectively.
- Tariff structures vary between facilities, depending upon the electric service provider and type of structure, as shown below in **Table E-6**:

Table E-6
Facility Tariffs

Location	Energy Provider	Tariff
KRS 1	Kentucky Utilities (KU)	Time of Day - Primary
RRS	KU	Power Service - Secondary PF Adj
Jacobson Reservoir	KU	Time of Day - Primary
Woodlake Pump Station	KU	Time of Day - Primary
KRS 2	Owen Electric	Large Industrial Rate LPB1

Energy Use Analysis - Three different routes of water delivery were examined for energy use per million gallons (kWh/MG), indicating KRS-2 as the most expensive and RRS as the least expensive. Note that RRS delivery varies tremendously depending on whether water is pumped directly from the Jacobson Reservoir or pumped directly to RRS from KRS-1, or "double pumped" from KRS-1 and the Jacobson Reservoir. The unit energy per MG below is actual billed data. Unit costs are summarized in **Table E-7**.

Table E-7 Actual kWh/MG Consumed (Jul '11 through Apr '12)			
	kWh	MG Treated	kWh/MG
KRS-1[(1)]	19,667,878	5,746	3,300 - 3,500
RRS[(2)]	7,247,122	3,436	2,109
KRS-2[(3)]	9,233,296	2,036	4,534

1. "KRS-1" billings only.
2. "Transfer + Jacobson + RRS" billings.
3. "KRS-2 + Woodlake" billings.

<u>Energy Use Savings Through Pump Improvements</u> - Several pumps have been identified that are consistently operating at low efficiencies and have substantial annual run times. Replacing these pumps can result in approximately 3% savings on the total energy bill.

<u>Energy Use Savings Identified in Hydraulic Model</u>- The hydraulic distribution model was updated with pump curves and efficiency curves and analyzed for energy savings. Under average day conditions. up to 2% total billed energy cost savings could be achieved with the installation of VFD's in certain parts of the system.

<u>Energy Demand Analysis and Peak Energy Savings</u> **-** An analysis of load factors indicate that KRS-2 and RRS seem to be operating within a reasonable range, whereas KRS-1, Jacobson, the Transfer pumps and Woodlake appear to have a high number of demand spikes. High demand spikes can impact demand load charges for the 11 months following that billing cycle. A limited analysis of KRS-1, Jacobson, the transfer pumps and Woodlake indicates that various forms of operational controls (i.e., load shifting, staggered equipment run time, real time SCADA monitoring) could result in a range of 10 to 20% per year in demand costs (roughly 3 to 5% in total billed energy charges), assuming that the operational proposals are acceptable to KAW staff.

<u>Chemical Usage</u> **-** The maximum cost from each month was summarized, then each monthly max was averaged for the year. The costs ranged from $90/MG (KRS-1) to $137/MG (RRS).

<u>Carbon Footprint</u> **-** The carbon footprint for KAW was calculated to be approximately 32,630 tons CO2e, excluding Scope 3 emissions, as shown below in **Table E-8**.

Table E-8 Carbon Emissions	
Emissions Source	**Emissions (metric tons CO2e)**
Direct Emissions	
Stationary combustion	73
Mobile sources	23
Process/fugitive	0
Refrigerant	4
Subtotal	**99**
Indirect Emissions	
Electricity	32,531
Purchased steam	0
Purchased chilled water	0
Subtotal	**32,531**
TOTAL	**32,630**

Alternative Energy Source Evaluation - Four technologies were screened for technical and economic feasibility, solar, wind, fuel cells and microturbines. With the exception of microturbines, alternative energy does not appear to be cost effective due to the very low cost of energy in the Kentucky region. This could change with credits and government subsidies, but, even then, because the Kentucky region benefits from such a low energy cost, this is also unlikely. If a suitable combination of pressure, flow and proximity to a receiving power source can be found in KAW's system, microturbines may be feasible. An initial screening of that criteria did not indicate that there is currently an application in the system.

Average and Maximum Day Operation Without RRS - Part of the analysis included a limited evaluation of supplying water with only KRS-1 and KRS-2 (ie; taking RRS off line) by using the WaterGems® hydraulic model. It is important to note that the model is a skeletonized model and includes only 50% of the total piping. Though supply assumptions can be made with this limited model, it is not possible to draw conclusions on whether a minimum pressure can be maintained - a full pipe model would be required to fully assess supply and pressure. Given those constraints, results indicated the following:

- Under Average Day Conditions (46 mgd), with RRS off line, the system is able to meet water demand, but uses about 13% more energy by doing so. Though the model is unable to accurately estimate electric demand charges, it is likely that demand charges would go up as well, as system headloss increases and pumping at higher heads is needed.

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RECOMMENDATIONS

A summary of the CPS recommended improvement projects for the KAW System is presented in **Table E-1**. All cost estimates are presented in year-2013 dollars, and should be inflated to the year of future construction. The locations of the KRS-1 recommended improvements are shown on **Exhibit 1-1**, RRS on **Exhibit 1-2.** The pipeline projects are shown on **Exhibit 1-3** and **Exhibit 1-4**.

TABLE E-1

KENTUCKY AMERICAN WATER

COMPREHENSIVE PLANNING STUDY - PRIORITY A & B PROJECTS

PROJECT No.	FACILITY	DESCRIPTION	PAGE	PROJECT DURATION (MONTHS) TOTAL	PERMIT/ DESIGN	CONSTR.	PROJECT COSTS
PRIORITY A PROJECTS							
A-1	RRS	Jacobson Reservoir Pump Station Improvements (Under Construction)		8	Completed	8	$3,300,000
A-2	Northern	Owenton Northern District Improvements (Under Construction)		12	Completed	12	$15,000,000
A-3	RRS	Filter Building Replacement		30	12	18	$14,100,000
A-4	Central	Leestown Pike Near Midway Interconnect		11	6	5	$1,900,000
A-5	KRS1	Residuals Improvements		24	12	12	$10,500,000
A-6	Piping/Tanks	Storage Tank and System Nitrification		17	3	14	$400,000
A-7	KRS1	Chemical Storage and Feed Improvements (includes potential bldg addition)		12	6	6	$1,200,000
A-8	KRS1	Pumping Modifications		18	6	12	$2,300,000
A-9	Central	Georgetown Bypass and US 25 Area Northern Feed Improvement		17	6	11	$2,500,000
				TOTAL PRIORITY A PROJECTS			**$51,200,000**
PRIORITY B PROJECTS							
B-1	RRS	Evaluation of Jacobson Reservoir Safe Yield		N/A	N/A	N/A	$65,000
B-2	Central	Briar Hill Tank Area Water Age and Pressures		17	3	14	$6,000,000
B-3	Central	North of Sadieville Water Quality and Modeling		13	3	10	$2,100,000
B-4	Northern	Areas Along KY-22 Future Max Day Pressures		13	3	10	$2,000,000
B-5	Central	State Highway Project Upgrades (Georgetown Bypass and Newtown Pike)		21	6	15	$8,600,000
				TOTAL PRIORITY B PROJECTS			**$18,765,000**
RECURRING (R) PROJECTS							
R-1	Distr. System	Increase Replacement Rate of Annual Main Repalcement Program		On-going			$9,900,000/yr
				TOTAL RECURRING PROJECTS			**$9,900,000/yr**

SECTION 1
RECOMMENDATIONS

1.1 PROJECT DESCRIPTIONS

This section provides a description of each of the recommended capital improvement projects for the KAW system. The estimated project duration and cost are included, followed by a detailed description of the project scope, purpose, and alternatives. **Exhibit 1-1 (KRS-1), Exhibit 1-2 (RRS), Exhibit 1-3 (Central System Distribution)** and **Exhibit 1-4 (Northern System Distribution)**, included at the end of this section, show the location of each of the proposed projects for the KAW Plants and System.

Until each project can be funded, there is an associated risk to either the customer service or other operating parameters. However, the need to balance the benefits of the capital improvement projects with the rate impact on customers and affordability is a reality. This Comprehensive Planning Study has attempted to prioritize the projects with most benefit in a logical sequence. The priority category has been included to briefly discuss the projects benefits and associated risk that the project implementation will alleviate.

1.2 RECOMMENDED CAPITAL PROGRAM

Through the analysis of historic operating data, discussions with KAW staff, analysis of demand patterns and projections, and with the aid of computer model to simulate various operating scenarios, the primary needs of the Kentucky system were found to be:

- Residuals and chemical improvements at KRS-1,
- Filter building improvements at the RRS,
- Distribution system water quality,
- Northern System connection to KRS-2,
- Main replacement program.

The Capital Improvement Program presented in this report seeks to address these system needs. Projects are identified as Priority A or B. Priority A designates high priority projects recommended for construction due to projected near term treatment, storage and pumping deficits, and associated transmission reinforcements. Priority B projects are recommended to enhance fire flow capacity and to provide various system improvements, and serve as a place holder for potential future projects based on actual system growth. Estimated scheduling for design and construction of each project is also presented. The design horizon includes design, permitting and document preparation.

Project A-1

JACOBSON RESERVOIR PUMP STATION IMPROVEMENTS

Design and Permitting:	complete		
Construction:	8 months	Project Cost:	$ 3,300,000

Need for Project:

The Jacobson Reservoir Pump Station (JRPS) is used to transfer water from Jacobson Reservoir to Kentucky American Water's (KAW) Richmond Road Station (RRS). The existing pumps are worn, which reduces pumping capacity and results in a significant loss of energy efficiency. Also, due to the lack of VFDs in this pump station, flow control is achieved during certain periods by recirculating flow back to the suction side of the pumps, which wastes energy. The existing electrical switchgear is deteriorated and includes two different operating voltages (2300V and 480V). Also, although one of the pumps is equipped with a backup diesel engine, supply from the station is limited and could be unavailable if the single standby pumping unit is out of service during a power outage event. Lastly, KAW seeks to eliminate the manual handling of dry potassium permanganate pails, to reduce labor requirements, as well as lower the risk of operator injury and accidental chemical spills into the reservoir.

Background:

The JRPS is equipped with three horizontal split-case pumps that draw suction from a nearby intake structure in the reservoir. The intake structure is accessible via an elevated catwalk from the earthen embankment that forms part of the reservoir dam. Because the safe yield of Jacobson Reservoir is limited, the JRPS typically operates, on average, about six months per year. The average pumping rate when in service is 12 million gallons per day (mgd), which also equates to the average rate of production at the RRS. Total annual pumpage from JRPS to the RRS amounts to approximately 2 billion gallons per year. Additional raw water supplies to RRS are obtained from the Raw Water Transfer Station at KAW's KRS-1 Plant. **Table 1** provides a summary of pertinent information for the existing pumping equipment at JRPS:

Table 1 JRPS Existing Pumping Equipment						
Raw Water Pump	**Driver**	**Pump Voltage (V)**	**Motor HP**	**Individual Field Rated Capacity (MGD)**	**Current Wire-to-Water Efficiency**	**Station Max/Reliable Capacity (MGD)**
Low Service Unit No. 1	Electric	460	100	6.0	63%	16.8 / 12.0
Low Service Unit No. 2	Electric	460	100	6.0	58%	
Low Service Unit No. 3	Electric Diesel	2,300	400 368	16.0	57%	

Pumps 1 and 2 were originally installed in 1966, and wire-to-water efficiencies are currently only 63 and 58%, respectively. Pump No. 3 was installed in 1956 and also appears to be nearing the end of its useful life. The pump casing is damaged and wire-to-water efficiency is currently only 57%. Although the original combined rated capacity for the three pumps was over 25 mgd, which is the rated capacity of RRS, the current actual pumping capacity with all three units in service is estimated to be only 16.8 mgd because of impeller wear and head loss conditions that occur at higher flows. In the event of a power outage, operators must go to the station to manually switch Pump No. 3 to diesel power in order to operate the pump station.

The existing power system includes both 2300-volt and 480-volt equipment. Pumps 1 and 2, as well as the existing reservoir blower, are powered from a 480-volt transformer. Pump 3 is powered from the existing 2300-volt service. The existing switchgear and motor controls are exhibiting signs of corrosion due to its location in the moist atmosphere of the pump station building. In fact, the switchgear for Pump 2 failed recently, and this pump is out of service until a new pump and switchgear are installed as part of this project.

Potassium permanganate is fed at the raw water intake from Jacobson Reservoir for the purpose of oxidizing iron and manganese and also for taste and odor control. Over the last five years, it has been fed over 40% of the time with an average dose of 0.5 mg/L. Granular potassium permanganate is delivered in 55-pound pails to the RRS where it is stored before being transferred to the JRPS every few days. To prepare batches of potassium permanganate, several pails of dry potassium permanganate are manually fed into a hopper every few days. The hopper then feeds the permanganate into a dissolver/mix tank from which the solution is applied to the intake well. The JRPS is located several miles from RRS, so it is time consuming and labor intensive for operators to travel to the pump station to refill the hopper. Also, operators manually carry the pails out to the intake enclosure and empty the pails into the dry hopper feeder. Lifting and pouring the 55-pound pails of permanganate puts workers at risk for injuries and creates some risk of a spill into the reservoir during the handling operation. The steel operating floor of the intake well is also exhibiting signs of corrosion.

Recommended Solution:

It is recommended that all three of the existing raw water pumps be replaced with new horizontal split-case pumps with 480V motors. Two (2) of the pumps should be equipped with VFDs, to allow efficient flow control over a wide flow range. Each VFD shall include a reduced voltage solid state (RVSS) bypass starter integrated with the drive, but be in an isolated/separate enclosure to allow servicing of the drive while operating in standby mode. A third pump should be constant speed, equipped with an RVSS starter. The proposed pumping capacity should be 25 mgd, with all 3 pumps in service, and a reliable 16.7 mgd with one pump out of service. Also, pumps should be selected which will allow efficient operation at an average pump station flow of 12 mgd (2 pumps, reduced speed).

The existing motors, motor controllers, and power distribution equipment are to be replaced. The existing aeration compressor unit is to remain and be served from the new power distribution equipment. The existing diesel engine and drive shall be removed and provisions for a new diesel-engine driven electrical generator shall be provided. Depending upon the budget, backup power could be provided either by an existing portable generator, or a new permanent generator installed as part of this project. The generator should be sized to provide power to two (2) pumps concurrently, the compressor, and all low voltage building equipment being served.

Lastly, the existing dry permanganate feed system should be replaced with a liquid sodium permanganate storage and system. It is proposed that the bulk tank for the liquid sodium permanganate be located in an outdoor containment structure adjacent to the pump station, and the day tank and chemical feed pumps be located inside the existing JRPS building in space that will be vacated after removal of the electrical switchgear. Consideration was given to other forms of bulk dry permanganate feed systems, but a liquid sodium permanganate system was identified as the preferred option. Other miscellaneous site improvements, including a reservoir inflow flow meter, a chemical injection vault, and access road improvements for chemical deliveries are also proposed.

Output and Benefits:

Replacement of the existing pumps, motors, and drives at the JRPS will increase the efficiency and improve the operability of the station. In particular, the application of VFDs on the new pumps will allow the output from JRPS to better meet the flow demands at RRS and further reduce energy consumption by eliminating the recirculation pumping. The proposed standby power generator will allow for automatic transfer to back-up the utility power in the event of a power outage, which will increase reliability and also improve the operation of the RRS. Replacement of the 2,300-volt pump (and other aged 480-volt equipment) with new power distribution equipment will eliminate potential safety and maintenance concerns of having medium voltage equipment located within the pumping station. Also, by upgrading or replacing the existing equipment and support systems at this facility, reduced overall energy demands and improved maintenance and operations is anticipated. Installing a sodium permanganate system will significantly reduce labor requirements, along with the potential risk of operator injury and spills from manual handling of 55-pound chemical pails.

Options:

One option considered was to limit the scope to a mechanical rehab of each pump, with the goal of improving the efficiency and hydraulic performance. However, this option would not address the energy loss that occurs while recirculating flow back to the suction side of the pumps, the reliability and safety concerns of using aged electrical equipment, and the issues with the existing potassium permanganate feed system.

Budget Discussion:

A Design-Build contract was awarded following a competitive bidding process. The budget outlined for this project is based on contracted amounts for Design and Construction Administration, and a 60% Target Cost for Construction.

Risks:

If this project is not completed, this pump station will continue to have poor pumping efficiencies and wasted energy due to recirculation, reliability and safety concerns due to aging equipment and multiple voltages, and a labor-intensive chemical feed process.

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
311 – Pumping Equipment	100	Asset Renewal poor condition	100

Project A-2

NORTHERN SYSTEM IMPROVEMENTS

Design and Permitting:	Done		
Construction:	12 months	Project Cost:	$15,000,000

<u>Need for Project:</u>

- The Owenton Water Treatment Plant (WTP) operates with a single treatment process train from the raw water transfer through the sedimentation process into two filters resulting in no redundancy and limited reliability.
- Chemical storage facilities are inadequately sized. Chemicals must be purchased in small batch quantities due to inadequate containment, including lack of chlorine containment.
- The Owenton WTP has no provisions for residuals processing and the lack of discharge monitoring has been identified by Kentucky Division of Water (DOW) inspectors as a risk for a violation to the discharge permit.
- There are only two filters at the plant - both are shallow and both are required for operation. While one is out of service for backwash, the plant is generally unable to keep up with normal system demands. The shallow filters are also limited in their ability to remove turbidity. Prior to the acquisition the plant was frequently unable to meet the current THM and HAA standards for disinfection by-products.

<u>Background:</u>

The Owenton facility was acquired by KAW in 2005 and serves KAW's Northern System. A 2009 engineering evaluation conducted by KAW compared the cost of supplying water to the northern district via the Owenton facility or via a 16 mile transmission main from newly constructed KRS-2. The assessment evaluated unit processes at the Owenton facility and assessed such factors as regulatory compliance, reliability, safety and efficiency, among others. The assessment concluded that retirement of the Owenton water treatment plant and construction of a 16 mile transmission main was most cost effective and met the aforementioned factors more effectively than maintaining the current Owenton facility.

Recommended Solution:

Installation of the new pipeline and the Owenton water treatment plant retirement is the recommended solution. Supplying the Northern System from the KRS-2 WTP entails constructing approximately 16 miles of 16-inch main along US-127 from the KRS-2 WTP to the intersection of KY-22/US-127 in Owenton and then proceeding east along KY-22 to near the intersection of Old Monterey Road and KY-22. The project phases are described below.

Phase I of the Northern System Connection project includes the construction of a 16-inch transmission main from the KRS-2 WTP to the north of Monterey. This includes approximately 39,620 linear feet of a 16-inch transmission main and appurtenances. This transmission main will supply flow from KRS-2 to Monterey and enable connections that will allow service to residents who are currently served by the Owenton WTP and that reside south of Monterey along US-127. When all phases of construction are complete, the transmission main's primary purpose will be to supply water to the new 600,000 gallon elevated storage tank that will be constructed outside of Owenton (see Phase III below).

Phase II continues the 16-inch transmission main north along US-127 from Monterey and connects into the Northern System in three locations: into an existing 6-inch line near the intersection of KY-845 and US-127, into an existing 8-inch line on US-127 near the intersection of US-127 and KY-22, and into an existing 6-inch line on KY-22 near Thomner Trailer Park Road. This includes approximately 44,945 linear feet of a 16-inch transmission main and appurtenances.

Phase III includes the construction of two elevated storage tanks and a booster pump station. The first storage tank will be constructed on the north side of Monterey and will be 300,000 gallons. The second elevated storage tank will be constructed outside of Owenton and will be 600,000 gallons. The new booster pump station will be rated for 2 mgd and will pump directly out of the 300,000 gallon elevated storage tank through the new 16-inch transmission main to the Northern System.

Also included in the overall improvements is the rehabilitation of the Fairgrounds Tank. The Fairgrounds tank is a very old tank in substantial need of restoration but can’t be taken off line because the system loses pressure - the tank controls the hydraulic grade line in Northern System. Once KRS-2 is online, the HGL in the area will increase and the project is needed to fill the Fairgrounds Tank, which is needed to service the rest of the Northern System. Under future demand conditions, the tank is therefore a bottleneck. Rehabilitation includes the upgrade approximately 3,000 lf of existing 6-inch main on US-127 from intersection of US-127 and KY-22 to new 12-inch main. Upgrade approximately 2,400 lf of existing 8-inch main on Seminary Street to new 12-inch main. Rehabilitate and repaint the interior and exterior of the Fairgrounds Tank.

Output and Benefits:

The new transmission main and outlined improvements will address redundancy and reliability issues that are currently a concern at the newly acquired Owenton WTP and allow for the consolidated treatment as a more efficient means for addressing future water quality regulations.

Options:

Three options were evaluated in total in order to arrive at this project 1) the current project extending a pipeline from KRS-2, 2) Upgrading the Owenton WTP and, 3) Constructing a pipeline for an interconnection the Bullock Pen Water District. Taking capital and operating expenditures in to account through 2020, Option 2 was estimated to cost $17,670,000 and Option 3 was estimated to cost $11,700,000 (for a booster station and 7 miles of transmission main).

Budget Discussion:

Costs are based on a 2009 evaluation conducted by KAW staff and reviewed by AW Corporate Engineering. Costs have been updated to 2013 dollars.

Risks:

A risk includes schedule impacts from permitting and implementing the several phases of work.

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
331 – Trans & Distribution Mains	100	Rel/Qual customer (pres taste etc.)	100

KENTUCKY AMERICAN WATER
RICHMOND ROAD STATION

Project A-3			
RRS REPLACEMENT OF FILTER BUILDING			
Design and Permitting:	18 months		
Construction:	18 months	Project Cost:	$ 14,100,000

Need for Project:

The filter piping gallery at the Richmond Road Station has reported structural issues and requires upgrades in order to remain in service. Significant corrosion and delaminating of the filter operational floor support beams has been observed and pose a safety to personnel working within the building. In addition a significant amount of pipe, pipe fittings, valves, and electrical equipment located in the filter gallery is in need of replacement due to corrosion. The cause of the severe corrosion in this area is likely due to a combination of chlorine vapors from the clearwell below the filters and the moisture in the room from leaking pipe joints, valves and filter walls. The corrosive environment needs to be addressed in order to prevent future corrosion. Additionally, the filter piping gallery is so congested that it makes the maintenance required to keep the facility in full operation extremely difficult resulting in diminished operating capacity of the filters.

Background:

Four of the filters (Filters 11 to 14) at the Richmond Road Station were constructed in 1924. Since then, several additions were made to expand the filter capacity. Six filters (Filters 15 to 20) were added in 1937, two more filters (Filters 21 and 22) were added in 1938 and the final four filters (Filters 23 to 26) were added in 1953 to bring the total number of filters to 16. The filter gallery is located beneath the operating floor of the filter building, between the two rows of filters. The clearwell is located directly beneath the filters and filter gallery.

The filter gallery has been the source of significant maintenance issues for many years. Corrosion has impacted the concrete, piping, and equipment in the filter gallery. Based on the presence of a chlorine odor in the filter gallery, it appears that some of the corrosion is caused by chlorine vapors in the room. It is likely that the source of the chlorine vapor is from the clearwell which is located directly beneath the filter gallery. In addition, standing water can be found in the filter gallery as a result of leaks in the piping and valves in the filter gallery as well as from seepage from the concrete filters which abut the filter gallery on both sides. As a result of the corrosion, significant time and money is spent on maintaining the equipment in the filter building. Although all 16 of the filter effluent valve operators were replaced within the last four years at a cost of approximately $175,000, six of the operators have failed already and need to be replaced.

Structural, mechanical, and electrical evaluations of the filter gallery were performed in 2012 as part of this CPS. The following is a summary of the findings of each of the inspections:

<u>Structural Inspection</u>

A structural evaluation of the existing filter building structure was performed in order to provide a preliminary assessment of the existing structure. It was concluded that the filter gallery and exterior walls between Filters 11 to 22 can be classified as in poor to severe condition and extensive repairs are needed. The condition of Filters 23 to 26, the bottom slab of the pipe gallery and the steel frame structure above the operating floor can be classified as satisfactory. Some immediate repairs were identified that are addressed in Project A-1.

In addition to the immediate repairs, a number of other improvements are needed in order to keep the filter building in service for the duration of the planning period:

- Repair of cracks in the walls of the filter gallery to stop leaks and minimize future deterioration of both the concrete and reinforcement.
- Application of corrosion resistant protective coating to all concrete in the filter gallery.
- Replacement of all corroded pipe supports in the filter gallery.
- Replacement of all steel ladders in the filter gallery so that they are compliant with OSHA standards.
- Repair of the top slab of Filters 19 and 20 by either strengthening the slab or replacing it
- Repair of the exterior northern walls of Filters 20 and 22 by either strengthening the wall or replacing it.

- Further evaluation of the roof slab and exterior walls of the filters for those areas that were not accessible for testing.

It should also be noted that the condition of the roof slab for Filters 16, 20, and 22 (located on the west side of the filter building) was found to be in poor to serious condition. It is recommended that access to the roof for these filters be restricted.

Mechanical Inspection

An inspection of the mechanical components (piping, valves, fittings, and mechanical valve operators.) was also conducted. The following observations were found with regard to the mechanical components:

- None of the piping is in danger of imminent failure, however, there are serious signs of corrosion and tuberculation along the pipe. Further examination is required to determine the extent of corrosion and type of repair.
- Of particular concern are the connections associated with the piping. The connector bolts in many of the mechanical joints and flanges are severely deteriorated.
- The overall condition of the valves is hard to assess since internal inspections were not possible. A reasonable assumption is that the valves will need replacement over the next 5-10 years at increasing frequency due to their age and the corrosive environment they are present in. The operations staff indicates that removal of the valves is possible through existing access hatches and front and back egresses. However, it is possible that in some locations, the only possible means for removal may be through a hole in the operations floor.

Electrical Inspection

In addition to the filter valve operator electrical equipment, the electrical service for the filter building and transfer switch for the emergency generator are located in the filter gallery. Most of this equipment is corroded as shown in the **Exhibit 1**.

Exhibit 1
Corroded Electrical Equipment in Filter Gallery (2)

According to the operations staff, all of the valves and actuators associated with the filter are operational. However, it has been noted that they are in need of constant repair due to their age and the corrosive environment that exists in the piping galley. Some of the operators are relatively new having been replaced within the last five (5) years and can be considered in fair/good condition. However, the remaining operators should be considered in poor/fair condition and will need replacement over time (within next 5 – 10 years).

The following improvements are needed for addressing the issues with the electrical equipment:

- Replace the electrical service and transfer switch and locate the equipment in new building, located outside of the filter building as illustrated in **Exhibit 2.**
- Replace the filter effluent electric valve operators with new electric valve operators constructed with corrosion-resistant materials.
- Replace all other electrical valve operators with new pneumatic valve operators. The duplex air compressor and solenoid valves for the pneumatic operators should be located on the filter operating floor or in the new electrical building described above.

Exhibit 2
Proposed Location of New Electrical Building

It should be noted that the filter gallery is a congested area with little room to walk or work as illustrated in **Exhibit 3**. In order to perform many of the improvements described above, valves, pipes, or pipe supports will likely need to be temporarily moved or relocated. This will make the work difficult and time consuming which will result in a significant increase in the cost of rehabilitation.

Exhibit 3
Congested Filter Pipe Gallery (2)

HVAC

In 2003, a dehumidifier was installed in the filter building to try to control the humidity in the filter gallery. However, there is still a considerable amount of moisture in the filter gallery. It is unknown how much of the moisture in the filter gallery is due to air flow conditions and pipe sweating and how much is related to leaking pipes and filters. There also appear to be leaks between the filter gallery and the chlorine contact chamber that is below the filters as there is a chlorine smell in the room. Expedited corrosion of equipment and piping could be a result of the chlorine vapors.

In order to reduce the amount of chlorine vapors in the filter gallery and, thereby reduce corrosion, the following HVAC improvements are needed:

- Install one roof intake fan and one roof exhaust fan, vented to the atmosphere, on the clearwell underneath the filters to create a negative pressure space for limiting the amount of chlorine fumes that leave the clearwell.

- Install one wall exhaust fan and a louver in the filter gallery in order to move fresh air through the space in order to minimize the amount of chlorine fumes in the room.

Recommended Solution:

Due to the extensive improvements needed to address the deficiencies noted above, it is recommended a new filter building be installed as described in Option 2 under the “Options” section of this project description below. The size of the building is estimated to be 120 feet by 50 feet based on the installation of 10 filters with space for a wider filter piping gallery and room for the electrical equipment and blowers for the filter air scour system. The existing clearwell will remain in service so site piping will be required to convey the filtered water to the clearwell. It is assumed that the new filter building will utilize as much of the existing site piping layouts as possible which should not be difficult given the proximity of the proposed new building to the existing building.

Output and Benefits:

The improvements will provide filters with a service life of at least 50 years. The new building will include a more spacious filter gallery which will reduce the risk to employees working in the space.

Options:

The options that were explored were rehabilitation of the filter building and pipe gallery as described above or replacement of the existing filter building with a new filter building. A present worth analysis was performed to help make this determination. For comparison purposes, the operation and maintenance costs (O&M) associated with each alternative is based solely on maintenance costs and labor costs associated with maintaining the filter building.

Option 1 – Rehabilitation of Existing Filter Building

Based on the structural, mechanical, and electrical inspections described above, the following improvements are included in this option in order to keep the filter building in service for the planning horizon (15 years):

- Repair and reinforce concrete beams in the filter gallery.
- Recoat the concrete roof slab in the filter gallery at selected places by removing the existing concrete cover, applying a corrosion inhibitor, reestablishing the concrete cover with repair mortar, and applying a corrosion-resistant protective coating.
- Replace corroded pipe supports in the filter gallery.
- Repair of cracks in the walls of the filter gallery and exterior walls.
- Apply corrosion resistant protective coating to all concrete in the filter gallery.
- Replacement of all steel ladders in the filter gallery so that they are compliant with OSHA standards.
- Repair of the top slab of Filters 19 and 20 by either strengthening the slab or replacing it
- Repair of the exterior northern walls of Filters 20 and 22 by either strengthening the wall or replacing it.
- Further evaluation of the roof slab and exterior walls of the filters for those areas that were not accessible for testing.
- Replace the nuts and bolts on the pipe mechanical joints in the filter gallery
- Replace the butterfly valves in the filter gallery (48 valves in all)
- Replace the filter effluent electric valve operators with new electric valve operators constructed with corrosion-resistant materials (16 operators, total)
- Replace all other electrical valve operators with new pneumatic valve operators. The duplex air compressor and solenoid valves for the pneumatic operators should be located on the filter operating floor or in the new electrical building described above. (32 operators, total)
- Replace the electrical service and transfer switch and locate the equipment in new building outside of the filter building.
- Install a roof mounted exhaust fan with a roof mounted intake supply fan on the clearwell and a sidewall mounted exhaust fan with louver in the filter piping gallery.

The capital cost of the improvements described above is estimated to be $9.4 million. It should be noted that the structural and mechanical evaluations, recommend that further study be done to assess the condition of the filter building, piping, and equipment. It is possible that the costs presented herein do not represent the full scope of the needed improvements.

Operators at the RRS estimate that approximately 20% of all maintenance costs at RRS are spent maintaining the valves in the filter gallery. In 2011, $112,000 was spent on maintenance at the RRS, which means an estimated $22,000 was spent on the valves in the filter gallery. Additionally, it was estimated that approximately 832 labor hours are spent on the filter building which equates to about 16 hours per week. Assuming an hourly cost of $50 for an operator (including benefits), this equates to $42,000 per year for a total of approximately $64,000 spent on the filter gallery, annually. For comparison purposes, it is estimated that the improvements described above will reduce the maintenance time and money spent in the filter gallery in half so the annual O&M is assumed to be about $32,000.

Option 2 – Replacement of Filter Building

It was assumed that the new filter building would be sized to treat 25 mgd and would have allowable filter loading rates up to 5 gpm/sf which is the maximum allowable loading rate by the DOW. The individual filter sizes are based on the dimensions of new filters recently installed in Hopewell, Virginia in order to develop an estimate based on recent actual costs. As such, the size of each filter was assumed to be 430 sf which means that 10 filters will be needed in order to provide a reliable capacity of 25 mgd at 5 gpm/sf.

Exhibit 4 provides a recommended size and location of the new building with the building sized to accommodate the filters, a sufficiently sized filter gallery, and an electrical room for the electrical service and emergency generator transfer switch which also houses blowers for filter air scour.

The capital cost of the new filter building is estimated to be $14.1 million. With the other immediate repairs needed in the existing filter building, the total capital cost for this option is $14.9 million. It is assumed that the new filter building will greatly reduce the amount of maintenance cost and labor that is currently associated with the filter gallery. Therefore, it is assumed that the O&M cost for this alternative will be approximately $14,000 per year.

Exhibit 4
Proposed Size and Location of New Filter Building

Cost Analysis of Alternatives

Table 1 shows the capital cost for each alternative with the estimated O&M cost associated with maintenance of the filter gallery. A present worth cost was estimated for each alternative based on a 15 year period, which is the estimated project life of the rehabilitation project, at a 7% interest rate.

Table 1 Cost Analysis of Alternatives		
Option	**Capital Cost ($ million)**	**O&M cost ($ per year)**
Option 1 – Rehabilitate existing filter building	9.4	32,000
Option 2 – New filter building	14.1	14,000

Additional Analysis of Alternatives

The advantages and disadvantages of each alternative is provided in **Table 2** below.

Table 2 Advantages/Disadvantages of Alternatives		
Option	**Advantages**	**Disadvantages**
Option 1 – Rehabilitate existing filter building	• Lower capital costs • Can perform work in phases so costs can be spread out over multiple years • Limited design work required	• Higher operation and maintenance costs • Risk of unidentified capital costs • Employee safety risks due to congested work environment and other structural issue • Limited control of humidity/corrosion in building • New filter building will likely be needed after ~15 years
Option 2 – New filter building	• Lower operation and maintenance costs • New building can be designed to meet KYAW needs and safety issues • Reduced number filters due to higher efficiency filters • Filter building life expected to be 50+ years • Work can be performed with minimal risk to disruption of service	• Higher capital costs • Work needs to be performed at one time (can't spread out costs over multiple years)

Based on the discussion evaluation presented above, the recommendation is to install a new filter building at the site. Although capital costs are higher for this alternative, the recommendation is based on the benefits of the new building and the risks associated with the upgrade to the existing building.

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
320 – Water Treatment Plant Equipment	100	Asset Renewal Poor Condition	100

KENTUCKY AMERICAN WATER
CENTRAL SYSTEM

Project A-4

LEESTOWN PIKE NEAR MIDWAY INTERCONNECT (12,600' of 12")

Design & Permitting:	6 months	
Construction:	5 months	Project Cost: $ 1,900,000

Need for Project:

The Central System has an 8-inch PVC pipe installed in the 1950s. The main has had 3 to 4 breaks in the last two years that has interrupted service to customers along the main. The main is the only main that allow for KAW to deliver water to Midway.

Background:

The 8-inch PVC main has experienced several main breaks in the past two years. It is believed that the main breaks are due to either poor bedding material and/or high pressures in that area, or a combination of the two. The main breaks have interrupted service for the customers along the main along with one of KAW's bulk water purchasers, the City of Midway.

Recommended Solution:

Install approximately 12,600 LF of new 12-inch main along Georgetown Road from Ironworks Pike to I-64. The new pipe will connect directly into the 42-inch transmission main coming from KRS-2, providing a second feed to Midway.

Output and Benefits:

The primary benefit is reliability to one of KAW's bulk wholesale customers.

Options:

An option is to not replace the main and be prepared for quick response to further main breaks.

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

n/a

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
331 - Trans & Distribution Mains	100	Rel/Qual customer (pres taste etc)	100

Project A-5

KRS-1 RESIDUALS IMPROVEMENTS

Design and Permitting:	12 months		
Construction:	12 months	Project Cost:	$10,500,000

<u>Need for Project:</u>

Project need is the result of an overloading of the residuals system. Due to the excess loading of the sludge lagoons from the wash water holding tanks (WWHTs) and Aldrich units, KAW runs the risk of overflowing one or more of the lagoons in to a nearby creek which runs through a sensitive conservation area, or exceeding the NPDES discharge limit for total suspended solids (TSS). The mode of operation also requires more manpower and is generally a high risk operation.

During periods of high flow, WWHTs and sludge lagoons are stressed with excess sludge and backwash. Due to the high frequency of backwashes, the WWHTs are quickly filled to capacity. In order to make room for upcoming backwashes, the WWHTs need to be drained before settling, so they discharge directly to the sludge lagoons rather than the supernatant discharging to the river. Adding to the excess WWHT loading is the fact that the influent valve is left open during a backwash, resulting in additional flows. This is done in order to maintain a steady flow to the remaining active units such that steady filter runs will be maintained.

Also during periods of high flow, the sludge in the Aldrich units is blown down at a higher frequency in order to maintain settled water turbidities less than 4 NTU, resulting in additional loading to the sludge lagoons.

<u>Background:</u>

<u>Backwash</u>: KAW runs all ten filters concurrently and backwashes one filter at a time while the remaining nine filters are operating. During periods of high flow, the backwash volume of the

WWHT is sometimes discharged directly to the sludge lagoons in order to make room for the next backwash. The typical mode of operation is to discharge the supernatant to the river, however settling time is sometimes not achieved at high flows. Also during a backwash, the influent water valve is left open in order to maintain an uninterrupted flow to the remaining units. This results in additional flow in to the wash water holding tanks (WWHT). There is currently no method of turning the other influent water valves down while the backwash influent valve is shut other than manual operation.

Each of the ten KRS-1 filters are sized at 727 sf. For granular media filtration, a typical filter backwash volume is 150 gpm/sf per wash. Based on this typical value, each backwash at KRS-1 would produce 109,000 gallons of wastewater. Backwash data indicates approximately 70,000 gallons used per filter cycle, or 90 gal/sf, which is somewhat less than expected.

The two wash water holding tanks are each sized with a 70 ft inside diameter and can hold 28,770 gallons per foot of depth. With the overflow elevation of 886.50 ft and a stop recycle elevation (top of the sludge cone) = 880.00 ft, there is 6 ft usable depth (assuming 6" freeboard to avoid overflow to Lagoon # 3). The 6 ft usable depth = 172,600 gallons per tank. These calculations show the WWHTs to be sized appropriately for typical operating conditions.

The practice of washing filters with the (clarifier) influent valve in the open position produces additional wastewater. For example, a filter rate (and influent rate) of 3 gpm/sf = 2,200 gpm (3.2 mgd) of additional wastewater totals to an additional 33,000 gallons of wastewater over a 15 minute wash period, approximately 20% of the usable volume. This unintended flow consumes the reserve volume in the WWHTs, and most likely causes overflows to Lagoon No. 3, and puts additional load on other liquid waste processes.

<u>Aldrich Unit Sludge Blowdown</u>**:** The Aldrich units were originally designed to operate in a sludge blanket mode, however this mode of operation has been abandoned and staff currently operates the units as upflow sedimentation without flocculation instead. The units can operate well up to 1 gpm/sf with a sludge blanket, whereas straight sedimentation loading rate is limited to around 0.5 gpm/sf. The settling capacity limitation is partially compensated for with operational adjustments, including an increase in the frequency of sludge blowdowns and operating in an enhanced coagulation mode.

At the rated plant capacity of 40 mgd, or 4 mgd/filter, the hydraulic loading rate (HLR) on each filter unit is approximately 1.0 gpm/sf. At this high HLR (1.3 gpm/sf at peak capacity of 52.4 mgd), settled water turbidities more easily reach 4 NTU, even at low raw water turbidities. Data from 2006-2010 indicates average settled water turbidities equal to 4.7 NTU, with a range of 1.5 to 9.4 NTU. When settled water turbidities start increasing, the current mode of operation is to blow down the sludge (open drain lines) until settled water turbidities begin to drop. The inlet valve is also closed. Sludge blow down exits through a new 12-inch, motor controlled valve via a drain line. Sludge blow down goes directly to the sludge lagoons, with the option of discharging to the WWHTs.

Sludge Lagoon #3 is usually reserved for overflow from various processes throughout the plant, while Lagoon #4 is essentially too small. Lagoon #1 is the furthest down gradient lagoon and closest to the stream and thus more sensitive and risky. This narrows KAW's options down to Lagoon #2 for most of the waste streams.

Operations will typically blow down a filter unit when settled water turbidities start exceeding 4 NTU. Particles do not have time to settle out at high HLRs, so the inlet valve is shut and sludge is blown in order to reduce settled water turbidities.

Sludge Lagoon Influent Piping**:** The Aldrich units and WWHTs currently drain to Lagoon #3 during maintenance, however the high volume and flow displaces a large volume of air in the pipe between Lagoon #3 and Lagoon #4. The release of air is violent and blows off manholes, potentially doing damage to the pipes.

Recommended Solutions:

The recommendations listed below will work conjunctively to decrease overall waste stream loading and minimize risk and manpower needs.

A) Aldrich Units - Retrofit with Tube Settlers – Tube settlers will decrease the need for high volume blowdowns because they allow a higher HLR than conventional sedimentation. HLRs for conventional sedimentation are typically around 0.5 gpm/sf, however they are currently exceeding 1 gpm/sf during high flows. Additionally, there is no flocculation, having previously relied on the sludge blanket for flocculation. The current operational adjustment has been an increased frequency of sludge blow down in order to decrease settled water turbidities. Tube settlers can achieve HLRs greater than 1 gpm/sf, and will reduce settled water turbidities to less than 4 NTU at a high HLR, reducing the need for high volume blowdowns. Note that this has been done successfully at other AW sites, including NJAW Swimming River WTP. The tube settlers will compensate for the loss of the sludge blanket, which was the original design intent of the Aldrich units. Note that the proposed tube settlers will function similarly to plate settlers at KRS-2.

It is recommended that KAW pilot two Aldrich units as a first step.

B) Total Suspended Solids (TSS) Monitoring – Install several TSS meters downstream of the Aldrich units, backwash basins, and sediment holding basins, especially the common drain line (the drain line is being doubled as a drain line and sludge blowdown line) in order to better monitor sludge concentrations. Blowdown was formerly done manually and visually in the sludge pit, however the 3-inch lines flowing in to the sludge pit are no longer utilized. By monitoring TSS, there will be more control over knowing when to close the sludge valve and terminate the sludge blowdown operation. The monitoring will also provide a stronger basis for recommendation for project A-1A (Tube Settlers). Each operator conducts a sludge blowdown differently, so there may be situations where the valve is left open longer than it needs to be, resulting in unnecessary lagoon loading.

C) Sludge Lagoon Influent Piping – Install a gate valve on the common 30 inch lagoon influent line between Lagoons #2 and 3. The valve will eliminate the air and water downstream downstream of the discharge lagoon.

D) Aldrich Units - Influent Valving – The valves and actuators are 26 years old and are suspected to be a contributing factor to leaks in to the Aldrich units. In addition to having surpassed their useful life, they are likely not as heavy duty as current actuators. While the concern for potential filter upset is warranted, it is also recommended that KAW consider closing the influent valve during a backwash while closely monitoring any potential impact on the remaining filters. It is our understanding that this is currently done during a sludge blowdown, thus the impact on the remaining filters, if any, should be the same. The influent valves are currently motor operated and are able to be controlled from the KRS-1 control room. Incorporating logic into the influent valve will automate this process while freeing up additional volume in the WWHTs during backwash. The addition of the tube settlers will also add buffering capacity to the increased flow across the active filters. [Note also that VFDs on the raw water pumps will dampen the initial influx of water].

Output and Benefits:

A decrease in flows to the Aldrich units with influent valve modifications and control will result in overall less flow in to the WWHTs, potentially freeing up 20% more additional volume.

Of the recommended improvements, a retrofit of the Aldrich units with tube settlers will have the most impact. The tube settlers will be able to accommodate a high HLR, thus decreasing settled water turbidities and decreasing blow down frequency from the Aldrich units. With fewer blow downs, there will be less stress on the sludge lagoons.

TSS monitoring will give operations much more transparency during sludge blow down, minimizing the potential of exceeding the blow down duration and overloading the sludge lagoons with supernatant.

A gate valve between Lagoons #2 and #3 will mitigate the violent displacement of air and water during the draining of the Aldrich units.

Options:

A) Option to Aldrich Units - Retrofit with Tube Settlers

- Operational adjustments to backwash (i.e., timing and duration of the sludge release, closing the influent valve) and wash water holding tanks (additional volume should open up if there is decreased backwash frequency). Operational adjustments will be better determined from the TSS monitoring downstream of the tanks.
- Treat the waste stream with plate or tube settlers downstream of the Aldrich units or sludge lagoons. Note that settled water turbidities would remain high and the overall waste volume would not decrease, but the waste would be clarified and dampened before discharge to the lagoons. A separate building would need to be constructed for this option.
- Develop a sludge blanket for flocculation, rather than tube settlers.

C) Option to Sludge Lagoon Influent Piping

- Install an air release valve rather than a surge relief valve. The surge relief preferred and more effective because the medium being displaced is water and not air.
- Rather than installing a gate valve between Lagoons #2 and #3, gradually fill the line in anticipation of the next draining of the WWHTs and Aldrich units.

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

There may be a risk to sizing the surge valve correctly for discharge to the lagoon. Safety precautions must be considered when timing a basin cleaning.

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
320 – Water Treatment Plant Equipment	100	Asset Renewal Poor Condition	100

Project A-6

STORAGE TANK AND SYSTEM NITRIFICATION AND MODELING

Design and Permitting: 3 months
Construction: 14 months

Project Cost: $400,000

Need for Project:

Modeling and field observation have indicated several storage tanks and areas in the KAW distribution system that are experiencing nitrification and overall poor water quality.

Background:

The Briar Hill and Sadieville tanks experience nitrification and the areas around these tanks have elevated water age compared to other portions of the system. The high water age results in depleted chlorine residual, which is also a regulatory concern. The Sadieville tank is currently off line and the Briar Hill tank nitrifies about once annually, which results in low to no chlorine for customers fed from this tank, including two consecutive systems (i.e., North Middletown and Judy Water). Samples were obtained from the tank during the period of 2008 to 2012 and the results are shown below. As can be seen, the maximum nitrite activation level is well above the 0.010 mg/L value that is indicative of nitrification.

Samples were obtained from the tank during the period of 2008 to 2012 and the results are shown below.

Table 1 Nitrate/Nitrate Concentrations in Briar Hill Tank - 2008 - 2012		
	Nitrite (mg/l) MCL=1.0 [Activation Level = 0.01 mg/l [(1)]]	**Nitrate (mg/l) MCL=10**
MAX	0.256	0.59
MIN	0.005	0.06
AVG	0.02	0.33

(1) The concentration where nitrification starts. Samples obtained approximately 4x/year.
(2) The Sadieville area and Russell Cave tank also experience elevated water age. The Sadieville tank is currently off line due to nitrification.

Recommended Solution:

Briar Hill Tank and Distribution System Water Quality: Tank nitrification is a result of residual available ammonia ("RAA") that is available for nitrification and denitrification. The high chorine demand could be due to a biofilm, corroded pipes and/or tuberculation and can be addressed with an aggressive distribution maintenance program.

An aggressive distribution system flushing program in conjunction with in-tank remediation is proposed. It is also recommended that residual ammonia leaving the plants be monitored closely for optimization, as well as other water quality adjustments (i.e., a higher pH if possible) in order to decrease the RAA. It is recommended that KAW staff develop an aggressive and annual (possibly semi-annual) flushing program to remove biofilm and deposits. There are several technologies that have been proven effective in biofilm removal, including unidirectional flushing (higher velocities than standard flushing), ice flushing (utilizes ice to abrade the pipe walls), and free chlorine flush (higher residual chlorine concentrations).

Treatment of the actual nitrification in the Briar Hill tank (and Sadieville/other tank(s) if brought back on line) may be addressed via in-tank mixing technologies. Similar to the distribution flushing program, there are several technologies available to address this, ranging from aeration to vertical mixing to inlet/outlet diffusion. The latter will likely be less effective, however, as it is dependent on the tank being filled and emptied.

North of Sadieville Area and Northern System: Similar to the Briar Hill area, elevated water age and nitrification have been recorded in the Sadieville tank and the area north of the tank. This area is also at the dead end of a long piping run from the Central System and experiences low circulation and high water age. Additional modeling and/or field sampling is recommended for this area to determine means and methods to increase circulation. Given it's proximity to the Northern System, one possibility may be connecting the two systems at this location, which may also help in circulating water in the Owenton System. There are staff concerns that the Owenton System may also experience nitrification once it is chloraminated with KRS-2 water, so this potential adverse impact should be considered during the modeling/field sampling effort. In addition, the modeling should address other issues in the Northern System, including pressures.

Output and Benefit:

If the measures taken above are implemented, nitrification and taste and odor issues are expected to decrease in the overall Central System, specifically the Briar Hill and Sadieville tanks. The Owenton System will also benefit from a closer evaluation of nitrification potential once that system is chloraminated.

Options:

If the above measures are unsuccessful, capital improvements have been identified specifically for the Sadieville and Briar Hill Tanks that will alter the hydraulics of those areas and improve water circulation and age. These are "B"-phase projects "Briar Hill Tank Area Water Age and Pressures" and "North of Sadieville Water Quality and Modeling".

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

There is risk that the tank retrofits and distribution system will not address the nitrification issue and the issue is primarily due to hydraulics and elevated water age, in which case more extensive capital improvements will be needed.

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
330 – Dist Reservoirs & Standpipes	100	Rel/Qual customer (pres taste etc.)	100

Project A-7

KRS-1 CHEMICAL STORAGE AND FEED IMPROVEMENTS

Design and Permitting:	6 months	Project Cost:	$ 1,200,000
Construction:	6 months		

Need for Project:

This project incorporates several components of chemical storage and delivery and TOC removal designed to enhance the robustness, reliability of KRS-1 operations, and minimizing the risk of plant shutdown due to insufficient chemical storage and feed. The issues include the following:

- There is currently only one feed line for PACl, KAW's primary coagulant. A single feed line puts the facility at risk for meeting filter effluent turbidities,
- Chlorine is fed in the hydrotreaters via rotameters that are mostly corroded or non-existent. There is also no means of measuring or monitoring the chlorine feed. Too little chlorine could compromise CT and disinfection. Standard operating protocol includes the ability to measure and monitor disinfectant dosage,
- An evaluation of chemical storage revealed that there was insufficient PACl storage (KAW's primary coagulant) to meet high demands, putting the facility at risk for insufficient coagulation, high turbidities and ultimately compromised filter effluent quality.

Background:

A) Chemical Feed Line Redundancy – PACl is the primary coagulant used at KRS-1 and is needed under all conditions in order to meet filtered water requirements. In the event that PACl feed is compromised, water production will also be compromised. A recent line break resulted in pot-holing throughout the site to investigate a PACl leak. The criticality of a primary coagulant warrants redundancy and an additional chemical feed line.

B) Intermediate Chlorine Feed – Several of the hydrotreaters do not have rotameters so there is no way of quantifying chlorine delivery other than sampling above the filters. Of the rotameters and piping that are still there, the rotameters do not work and the piping is in poor condition. Replacement of rotameters and associated piping will allow more control and quantification of chlorine dosing.

C) PACl/Coagulant Storage - An evaluation of PACl storage during varying plant flows and dosing concentrations indicated that PACl storage was insufficient. Staff utilized ferric chloride in the past but that resulted in red staining of the equipment and sometimes in the river, which resulted in more PACl use. With the additional PACl usage there was a commensurate loss of storage. Staff is finding that PACl alone may not be sufficient in removing TOC (see above), so they are currently piloting a PACl/Polymer blend with aluminum chloro-hydrate (ACH) addition during high TOC events.

Recommended Solution:

A) Chemical Feed Line Redundancy – The primary coagulant should have redundancy to ensure adequate filter effluent quality at all times. The project would entail a new connection to the existing (or new polymer blend) day tank, new metering pump, new building piping and new yard piping. The outside work would consist of excavation, new penetration and piping in to the piping vault, and a new penetration or tee in to the existing PACl feed line at the rapid mix facility.

B) Intermediate Chlorine Feed – The project entails replacing the piping and rotameters in five individual valve vaults located between the hydrotreaters. The project involves cutting out existing piping and/or existing piping and rotameter, and replacing with new rotameter and piping. Chlorine dosing can then be monitored and flow paced from the control room. The hydrotreaters should be fed with a common chlorine supply line. Cost assumes ten new panels and assumes gaseous chlorine will be fed at the units, as chlorine solution accelerates degradation of new meters due to the high corrosivity. The project does not include replacement of unit chlorine rings.

C) PACl/Coagulant Storage - Coagulant storage will depend upon the success of the coagulant piloting. If the piloting indicates that the new coagulant is effective, there will be sufficient storage available. If it is not successful, it is recommended that KAW staff consider the use of ferric chloride to ensure TOC removal, and also increase the storage for PACl. A cost for a new tank and building addition has been included in the event that the pilot testing does not provide sufficient TOC removal.

Output and Benefits:

There will be redundancy for the primary coagulant which will add to the reliability of the system, as well as a more reliable chlorine feed. The current powdered activated carbon system is extremely labor intensive when it is needed, so the installation of a sack system is a reasonable modification to improve operations and working conditions.

TOC removal is currently being addressed by piloting a new coagulant. If the new coagulant is effective, then it will reside in one of the two 16,450 bulk tanks that is currently occupied by ferric and PACl. If it does not prove effective in removing TOC, then the ferric and ferric bulk tank is still on hand. If the latter is embraced, then a new bulk storage tank for PACl will be needed. A proposed addition to the existing chemical building will provide the floor space for the new tank. A cost has been included for the addition and new tank and appurtenances.

Options:

Another option for TOC removal is ozonation. Based on an ozonation project at the 15 mgd Canoe Brook WTP facility, the capital cost at KRS-1, assuming 40 mgd capacity, could be as high as $9 million. Operating costs would be approximately $25 per MG (a "rough" estimate) for power and liquid oxygen.

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

There is risk that the pilot testing of the polymer blend is not successful in achieving DBP reduction.

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
320 – Water Treatment Plant Equipment	100	Asset Renewal Poor Condition	100

KENTUCKY AMERICAN WATER

KRS-1 PUMPING MODIFICATIONS

Project A-8

PUMPING MODIFICATIONS

Design and Permitting:	6 months	Project Cost: (Transfer Pump VFD Only)	$ 2,300,000
Construction:	12 months		

Need for Project:

Pumping efficiency was evaluated at KAW from four perspectives - 1) operational perspective, 2) energy optimization, 3) energy efficiency and 4) energy demand. The analyses indicated that there is room for improvement both operationally and from an energy perspective through pump modifications.

Background:

An evaluation of KAW's system was conducted from three perspectives:

- Operations - Each of the low service pumps at KRS-1 is rated at 14.4 mgd, serving both KRS-1 and RRS. Most of the time the plant delivery requirements at KRS-1 and RRS (either combined or individually) do not match the incremental output of the raw water pumps. This results in either throttling of the transfer pumps to accommodate for mismatched RRS high service delivery, or frequent on/off operation to accommodate mismatched KRS-1 high service delivery.

- Energy Optimization - Plant delivery was evaluated at a macro level scale in order to evaluate system performance from the perspective of energy consumption for the three plants. The model indicated that KRS-2 was the least efficient delivery route, followed by KRS-1, then RRS. If RRS double pumps from Pool 9 and Jacobson Reservoir, then RRS is the least efficient. Specifically, the model indicated an energy savings if the transfer pumps were not throttled.

- Energy Efficiency - As part of the EUI initiative, individual pumps were evaluated by AW engineering to assess where the pumps are operating relative to their Best Efficiency Point (BEP). An efficiency and operation analysis of the KAW distribution system and pumps indicates that there are several pumps, in KAW's system where energy savings may be achieved by rehabilitating or replacing low efficiency pumps as shown below in **Table 1** (excerpted from the May 11, 2012 pump rehabilitation memo attached as **Appendix D**).

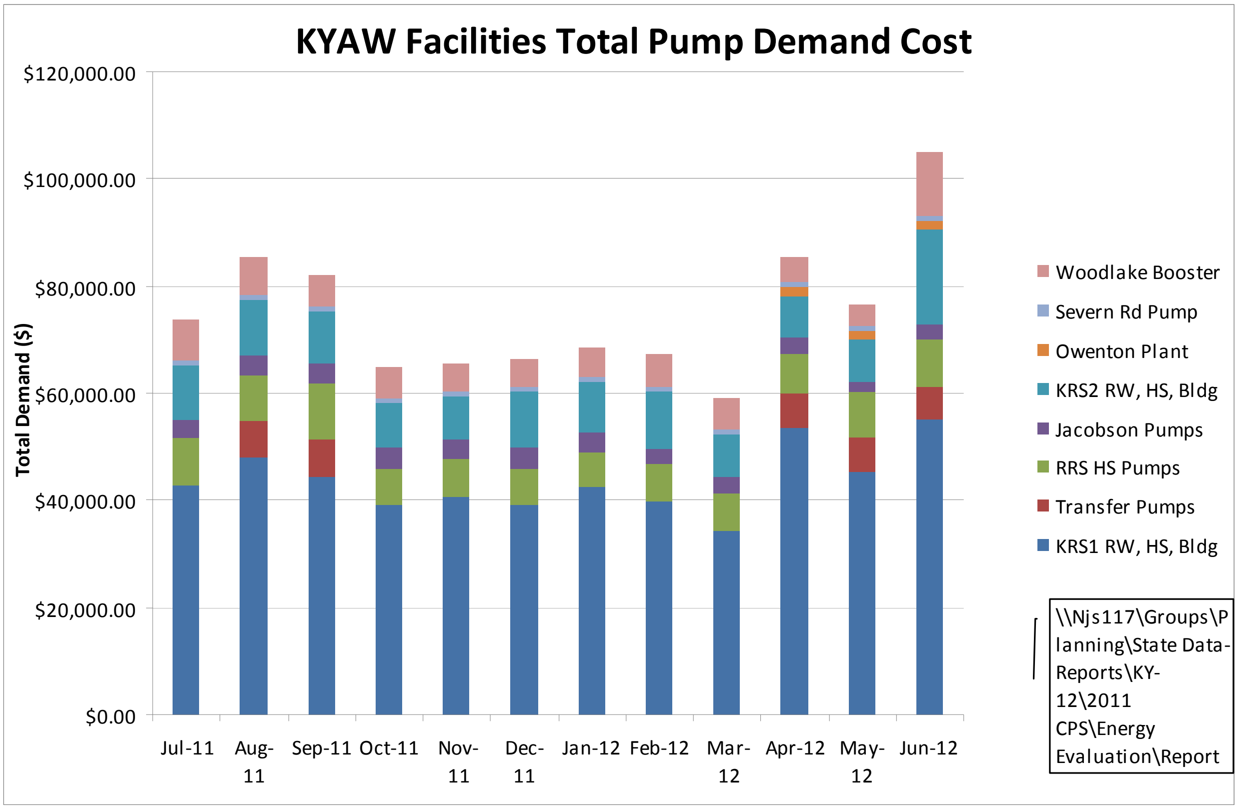
Table 1

Summary of Existing Pumping Equipment

Pump No.	Pump Description	Pump Type	Design Flowrate (MGD)	Nameplate Motor Size (HP)	Wire-To-Water Efficiency	Historic Annual Runtime (Hrs)	Percent of Time in Service	Estimated Annual Pumpage (MGD)	Estimated % of Annual Volume Delivered	Estimated Current Pump EUI (MWH/MG)
KENTUCKY RIVER STATION NO. 1										
1	Raw Water	VTP	14.4	1,250	69	2,997	34%	4.9	17%	1.69
2	Raw Water	VTP	14.4	1,250	74	4,656	53%	8.2	29%	1.58
3	Raw Water	VTP	14.4	1,250	72	2,729	31%	4.7	16%	1.62
4	Raw Water	VTP	14.4	1,250	72	1,738	20%	3.0	10%	1.62
5	Raw Water	VTP	14.4	1,250	73	2,858	33%	5.0	18%	1.60
6	Raw Water	VTP	14.4	1,250	71	1,630	19%	2.8	10%	1.64
	SUBTOTAL		**86.4**	**7,500**	**N/A**	**16,608**	**N/A**	**28.5**	**100%**	**1.62**
8	Transfer	HSC	18.1	1,000	N/A	1,106	13%	N/A	N/A	N/A
9	Transfer	HSC	18.1	1,000	N/A	2,132	24%	N/A	N/A	N/A
	SUBTOTAL		**36.2**	**2,000**	**N/A**	**3,238**	**N/A**	**N/A**	**N/A**	**N/A**
10	High Service	VTP	8.0	700	60	3,838	44%	2.5	14%	2.09
11	High Service	VTP	8.0	700	61	5,617	64%	3.5	19%	2.06
12	High Service	HSC	8.1	700	70	4,258	49%	3.5	19%	1.79
13	High Service	HSC	10.0	800	73	2,599	30%	2.4	13%	1.72
14	High Service	VTP	10.0	800	75	5,861	67%	4.5	25%	1.67
15	High Service	VTP	10.0	900	56	2,611	30%	1.9	11%	2.24
	SUBTOTAL		**54.1**	**4,600**	**N/A**	**24,783**	**N/A**	**18.4**	**100%**	**1.89**
RICHMOND ROAD STATION										
1/C	Jacobson Resv.	HSC	4.0	100	63	3,100	35%	1.4	20%	0.30
2/H	Jacobson Resv.	HSC	4.0	100	58	3,300	38%	1.5	20%	0.34
3/F	Jacobson Resv.	HSC	12.0	400	57	3,800	43%	5.2	60%	0.46
	SUBTOTAL		**20.0**	**600**	**N/A**	**10,200**	**N/A**	**8.1**	**100%**	**0.40**
6	High Service	HSC	6.5	250	70	1,695	19%	1.2	12%	0.86
7	High Service	HSC	12.0	500	73	6,473	74%	8.7	85%	0.85
8	High Service	HSC	4.0	300	63	594	7%	0.3	3%	0.95
10	High Service	HSC	5.5	250	54	0	0%	0.0	0%	1.11
	SUBTOTAL		**28.0**	**1,300**	**N/A**	**8,762**	**N/A**	**10.2**	**100%**	**0.85**

- Energy Demand - An analysis was conducted on energy demand at the three facilities. As shown below on **Exhibit 1** (excerpted from energy study), KRS-1 comprised over 50% of the demand costs between July 2011 and June 2012 (billings include the combination of the KRS-1 plant, the low service pumps and high service pumps).

Exhibit 1
Energy Demand Summary



Recommended Solution:

- Pump Rehabilitation or Replacement - It is recommended that KAW proceed with rehabilitation or replacement of KRS-1 high service pumps 10, 11 and 15. While Table 1 indicates that there are nine pumps that would benefit from either rehabilitation or replacement, KAW has opted to address the KRS-1 low service pumps separately from CPS, therefore recommendations for those pumps are not made (the pumps efficiencies are also relatively good compared to new pumps). The RRS Jacobson pumps are in the process of being replaced, so similarly, recommendations are not made for these pumps. RRS high service pumps 8 and 10 indicated field tested efficiencies of 63% and 58%, respectively, however they are only used for a very small percentage of the time.

- Design Points - It is recommended that KAW consider revising the pump curve design points to better match the high service pump flow points at KRS-1 and

RRS. System discharge pressures have decreased since the original pump design due to distribution system improvements.

- VFDs - It is recommended that KAW proceed with the installation of VFDs on three KRS-1 low service pumps and three KRS-1 high service pumps, and one transfer pump, in order to better facilitate flow matching. The four analyses converged on the primary need to better modulate flow between the low service and high service pumps at KRS-1 and RRS. By slowly ramping KRS-1 raw water pumping flow up and down rather than the current surged on/off operation, there is better flow matching between the low service pumps and high service pumps at KRS-1, and also better flow matching between the KRS-1 low service pumps and the RRS high service pumps (via the transfer pumps).

Output and Benefits:

From an operational perspective, the installation of VFDs provides more flexibility with KRS-1 and RRS plant operations, allowing an even flow modulation between low and high service pumping. The even flow distribution will also decrease demand costs by mitigating energy demand “spiking”.

The recommended pump rehabilitation and/or replacement will increase energy efficiency at the KRS-1 high service pumps.

Options:

VFD installation will be the more costly recommendation. There would still be considerable benefit from installing VFDs at only the KRS-1 low service pumps, both operationally and from energy efficiency and demand. Consideration could be given to eliminating the recommendation for VFDs on the KRS-1 high service pumps.

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

There is risk that sitting for the low service pump VFDs may become problematic due to access and space issues to the existing drives. The VFDs would also need climate control in both locations. Space constraints may also be an issue for the VFDs on the high service pumps.

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
311 – Pumping Equipment	100	Asset Renewal Poor Condition	100

KENTUCKY AMERICAN WATER
NORTHERN SYSTEM

Project A-9

GEORGETOWN BYPASS AND US 25 AREA

Design & Permitting:	6 months	
Construction:	11 months	Project Cost: $ 2,500,000

Need for Project:

The Muddy Ford tank is 25 years old and can not be taken out of service for maintenance. The Muddy Ford Tank serves Toyota and has had only external inspections conducted on it. This project will help facilitate taking the tank out of service by supplying Toyota in the interim. It will also provide a second supply line to Toyota to assure uninterrupted and reliable service. The project also provides a backup supply line to Scott County and the northern area if there were a break in the Newtown main.

Background:

KAW staff have unsuccessfully tried taking the Muddy Ford Tank off line to conduct maintenance on this tank. This will be more easily facilitated with this project and will provide another source of water to Toyota while Muddy Ford is off line.

The northern area of the Central System is essentially served by one main extending up Newtown Road. Should something happen to this main, the northern portion of the Central System and Toyota would not be provided supply. This project will provide that redundancy.

The project also improves pressures in this area under future max day demand conditions.

Recommended Solution:

Upgrade the existing 6-inch main to approximately 15,700 LF of new 12-inch main on Lisle Road between US-25 and Lemons Mill Road.

Output and Benefits:

The project will assure an uninterruptible source of supply to Toyota, one of KAW's largest customers. It allows the Muddy Ford tank to be taken off line, which is the primary service facility for Toyota. The project will also increase reliability by providing a second larger supply line to the northern area and Toyota.

Options:

An option is to not implement the project, but eliminate the possibility of taking the Muddy Ford tank off line.

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

Marginal benefit would actually be provided by the project.

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
331 - Trans & Distribution Mains	100	Rel/Qual customer (pres taste etc)	100

Project B-1

EVALUATION OF JACOBSON RESERVOIR MANAGEMENT AND OPERATION DAM STABILITY

Study:	6 months	Project Cost:	$65,000

Need for Project:

The Jacobson Reservoir provides the least expensive source of water to the entire KAW system due to the fact that it does not require the significant pumping that KRS-1 and KRS-2 do. It is also the smallest capacity source of water as well as requiring the most monitoring and upkeep. The reservoir is currently monitored frequently for water quality while requiring aeration and chemical addition to address algal formation.

Background:

Jacobson Reservoir was constructed to impound East Hickman Creek in 1914 with an estimated gross storage capacity of 619 MG. The dam is an earth fill structure with a concrete core wall and spillway. The crest of the spillway is at elevation 967.3 ft. Depth-capacity surveys were conducted in 1964, 1977 and 1991 with resulting gross capacities determined at 818 MG, 745 MG and 619 MG respectively. The current estimated usable storage capacity is 500 MG based on the 1991 study.

This project recommends that an Engineering Consulting firm qualified in reservoir operation be retained to conduct a review of the water quality of the reservoir and capacity. The estimated cost includes funds for Water Company supervision of the project. The project should be accomplished early to help improve operational efficiencies at Richmond Road Station.

Recommended Solution:

It is recommended that Kentucky American retain a consultant to determine if there is a greater efficiency available in the management of the Reservoir, either through additional algal treatment, or partial dredging. The study would also update the estimated volume of the reservoir, while potentially allowing for more volume and increased water quality.

Output and Benefits:

An evaluation of the reservoir by an expert may provide a water quality alternative to the current aeration and chemical addition, both of which are costly. From a water quantity perspective, additional capacity will provide KAW with added low cost capacity.

Options:

Operating as-is.

Budget Discussion:

Cost is based on a 6 month study by an outside consultant.

Risks:

There is not much risk to the actual study.

Purpose Codes and Drivers:

Asset Type	**%**	**Purpose Code**	**%**
305 – Collect & Impound Reservoirs	100	Rel/Qual customer (pres taste etc.)	100

Project B-2

BRIAR HILL TANK AREA WATER AGE AND PRESSURES

Design and Permitting: 3 months
Construction: 14 months

Project Cost: $6,000,000

Need for Project:

[Note: This project is a recommended as a backup solution to the "Storage Tank and System Nitrification and Modeling", if the recommended approach for that project is not effective]. Modeling and field observation have indicated the Briar Hill Tank Area as having elevated water age compared to other portions of the system, as well as low pressures during future max day demands. The high water age results in zero chlorine residual, taste and odor issues and tank nitrification, and the need to empty the tank once per year. During this period, the tank is down for approximately four days, chlorinated and brought back on line.

Background:

Due to the elevated water age in this area, chlorine residual drops to zero, resulting in taste and odor issues as well as nitrification in the tank. Samples were obtained from the tank during the period of 2008 to 2012 and the results are shown below.

Table 1
Nitrate/Nitrite Concentrations in Briar Hill Tank – 2008-2012

	Nitrite (mg/l) MCL=1.0 [Activation Level = 0.01 mg/l (1)]	Nitrate (mg/l) MCL=10
MAX	0.256	0.59
MIN	0.005	0.06
AVG	0.02	0.33

(1) The concentration where nitrification starts.

It is possible that the water coming from KRS-2 displaces water from the western portion of the system toward the east, where the Briar Hill tank is located. At low flow conditions, the water becomes stagnant due to difficulty with tank turnover and this may be the cause of the elevated water age. A solution to address the issue hydraulically was modeled by Strand in their 2012 hydraulic modeling report.

Recommended Solution:

In order to increase the turnover rate in the Briar Hill tank, and assuming the aforementioned improvements are not successful, the following sections of water main are recommended for installation:

1. Install approximately 4,000 feet of 8-inch water main on Bryan Station, from Briar Hill Road to Muir Station Road.
2. Install approximately 8,300 feet of 6-inch water main on Rockwell Road, from Clintonville Road to Mimosa Drive, replacing existing 4-inch water main.
3. Install approximately 27,700 feet of 12-inch water main on North Cleveland Road from Briar Hill to Todds Road, replacing existing water mains.
4. Install approximately 11,200 feet of 12-inch water main on Todds Road from North Cleveland Road to Interstate 75, replacing existing water mains.

In addition to the water main, the following valves should be closed/opened, isolating the Briar Hill Tank Area as its own gradient.

Valves to be closed:

1. Intersection of Houston Antioch Road and Paris Pike
2. Intersection of Rolling Hills Drive and North Cleveland Road
3. Install a new valve (to be closed) at the intersection of Bryan Station Road and Muir Station Road.
4. Install a new valve (to be closed) at the intersection of Winchester Road and North Cleveland Road.

Valves to be opened:

1. Intersection of Muir Station and Paris

Output and Benefit:

The benefit will be higher tank residual, less nitrification and a reduction in taste and odor complaints at Briar Hill.

Options:

As noted above, this project is a recommended as a backup solution to the "Storage Tank and System Nitrification and Modeling**"** project, if the recommended approach in that project is not effective.

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

n/a

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
331 - Trans & Distribution Mains	100	Rel/Qual customer (pres taste etc)	100

KENTUCKY AMERICAN WATER
CENTRAL SYSTEM

Project B-3

NORTH OF SADIEVILLE WATER QUALITY AND MODELING

Design & Permitting:	3 months	
Construction:	10 months	Project Cost: $ 2,100,000

Need for Project:

[Note: This project is a recommended as a backup solution to the “Storage Tank and System Nitrification and Modeling**”** project, if the recommended approach in that project is not effective]. The Sadieville area is experiencing elevated water age, manifested via nitrification and elevated nitrite/nitrate levels. The Sadieville tank has been off line for approximately eight years. There is also concern that the Northern System will experience high water age and nitrification when it is converted to chloramines.

Background:

Sadieville is located in the northernmost part of system, approximately 26 mile north of the RRS treatment facility. There is a single line extending north to the Sadieville tank for approximately 4 miles. Due to the nature of a single line extending that long distance, the Sadieville area has numerous dead ends. The Sadieville tank has been periodically removed from service over the past eight years due to nitrification. The Sadieville area is still able to maintain pressure with Sadieville off line. In addition to dead ends and nitrification in this area, there is concern that the Northern System will experience high water age and nitrification when it is converted to chloramines.

Because the Sadieville area is so close to the Northern System, a connection between the two systems may benefit both systems.

Recommended Solution:

KAW staff has concurrent concerns with water quality in the northern Sadieville area and Northern System and has considered making a connection from the area north of Sadieville in to the new Northern System. This proposal includes the installation of approximately 19,000 LF of new 8-inch main on KY 330 between Corinth and KY 607, which is the $2.5M cost associated with this project. The most effective means of addressing these issues will be addressed in the “Storage Tank and System Nitrification and Modeling**”** project.

Output and Benefits:

The improvements will address nitrification and water age north of the Sadieville tank and in the Northern System in the vicinity of Sadieville.

Options:

[Note: This project is a recommended as a backup solution to the “Storage Tank and System Nitrification and Modeling**”** project, if the recommended approach in that project is not effective].

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

If the new 8-inch were installed without some sort of prior validation (modeling and/or field measurements), there is risk that the installation addresses neither the Sadieville tank nitrification or the Northern System nitrification. The Northern System is new and untested with respect to water age may require further investigation.

Purpose Codes and Drivers:

Asset Type	**%**	**Purpose Code**	**%**
331 - Trans & Distribution Mains	100	Rel/Qual customer (pres taste etc)	100

Project B-4

AREAS ALONG KY-22 FUTURE MAX DAY PRESSURES

Design & Permitting:	3 months	
Construction:	10 months	Project Cost: $ 2,000,000

<u>Need for Project:</u>

Pressures in the vicinity of KY-22 are below 45 psi and there are pockets below 30 psi under future max day demand conditions. There are no customer complaints in the areas as this is a future condition.

<u>Background:</u>

The project area is located to the east of the Fairgrounds tank and exhibits low pressures during future maximum day demand conditions due to the headloss exhibited in the existing 4-inch main.

<u>Recommended Solution:</u>

Upgrade approximately 19,150 LF of existing 4-inch main along KY-22 from KY-227 to KY-845 to new 6-inch main.

<u>Output and Benefits:</u>

Alleviate low pressures in the vicinity of the new 0.6 mgd Owenton tank.

<u>Options:</u>

n/a

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

n/a

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
331 - Trans & Distribution Mains	100	Rel/Qual customer (pres taste etc)	100

KENTUCKY AMERICAN WATER
CENTRAL SYSTEM

Project B-5

STATE HIGHWAY PROJECT UPGRADES
(GEORGETOWN BYPASS AND NEWTOWN PIKE)

Design & Permitting:	6 months	
Construction:	15 months	Project Cost: $ 8,600,000

Need for Project:

These two projects will increase the flow capacity in the central portion of the system and also slightly improve the pressures to above 45 psi under future demand conditions. The projects also help to provide redundancy to the northern area. If not for future improvements to the roadways that the piping is in, the benefit of the projects would be marginal. The State project, however, provides KAW the opportunity to increase the size of the mains in these two areas and increase future demand pressures and flows to the northern portion of the Central System.

Background:

Under future demand conditions, future pressures within the area are below 45 psi. Kentucky Department of Transportation has plans to increase highway capacity at the locations of both of these projects and will pay for in kind replacements. If KAW were to increase the diameter of the piping in these areas, they would be responsible only for the difference in diameter of the pipe, thus making the projects more attractive.

Recommended Solution:

Construct approximately 17,800 LF of new 24-inch main along US-25 from the existing 42-inch transmission main on Ironworks to Kearney Ridge Boulevard connection to the existing 16-inch main at the intersection of US-25 and Kearney Ridge Boulevard. In addition, upgrade the existing 16-inch main to approximately 11,800 lf of new 24-inch main on Newtown Pike between I-75 and New Circle Road.

Output and Benefits:

The projects will provide additional flows and added pressure during future maximum day conditions, and also increase redundancy to the Northern System.

Options:

Do not include the projects.

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

Risks:

Projects are dependent on implementation and timing of the State Highway project.

Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
331 - Trans & Distribution Mains	100	Rel/Qual customer (pres taste etc)	100

Project R-1

INCREASE REPLACEMENT RATE OF MAIN REPLACEMENT PROGRAM

Design and Permitting:	On-going	Project Cost:	$9,900,000
Construction:	On-going		

Need for Project:

An adequately funded and proactive program to replace water mains on an on-going basis will help reduce the overall risk and consequences associated with main breaks, including but not limited to: avoiding potential insurance claims due to damages associated with main breaks, maintaining reliable service to customers, and avoiding the potential for contamination while repairing pipe. Replacing small diameter main would increase hydraulic capacity and improve water quality by replacing pipe with tuberculation and/or corrosion. In addition, it will help reduce non-revenue water in the system.

Background:

The Kentucky American Water System consists of approximately 2,100 miles of main. American Water's Asset Investment Strategy identifies achieving sustainability of the performance of its assets as a key long-term goal. It is recommended to ensure each system's current level of pipeline replacement investment levels are adequate in maintaining adequate, sustainable levels of service. Kentucky's main replacement program replaced approximately 2 to 4 miles of main per year from 2009 to 2011. This equated to a replacement rate of approximately 0.11% to 0.21% per year, or a complete replacement of the distribution piping once every 475 to 900 years. An optimal replacement rate of 1% per year, or once every 100 years, would more closely align the replacement rate with the anticipate lifetime of the distribution main in the system.

A main replacement program achieves American Water's long-term goal of maintaining adequate, sustainable levels of service while also improving system reliability, hydraulic capacity, and water quality in the system as well as decreasing non-revenue water.

Recommended Solution:

It is recommended to increase the main replacement program from the current level of 0.21% per year to a replacement rate up to 1.0% per year. The annual main replacement program should continue to focus on critical areas within the system such as those that have experienced service disruptions, water quality problems, low operating pressures, low fire flow, or the location of a critical customer (larger users, school, hospital, etc.).

Output and Benefit:

The output and benefits of the main replacement program include:

- Reduced general risk of operating the water system,
- Increased hydraulic capacity,
- Improved water quality,
- Increased fire protection, and
- Increased pressures.

Options:

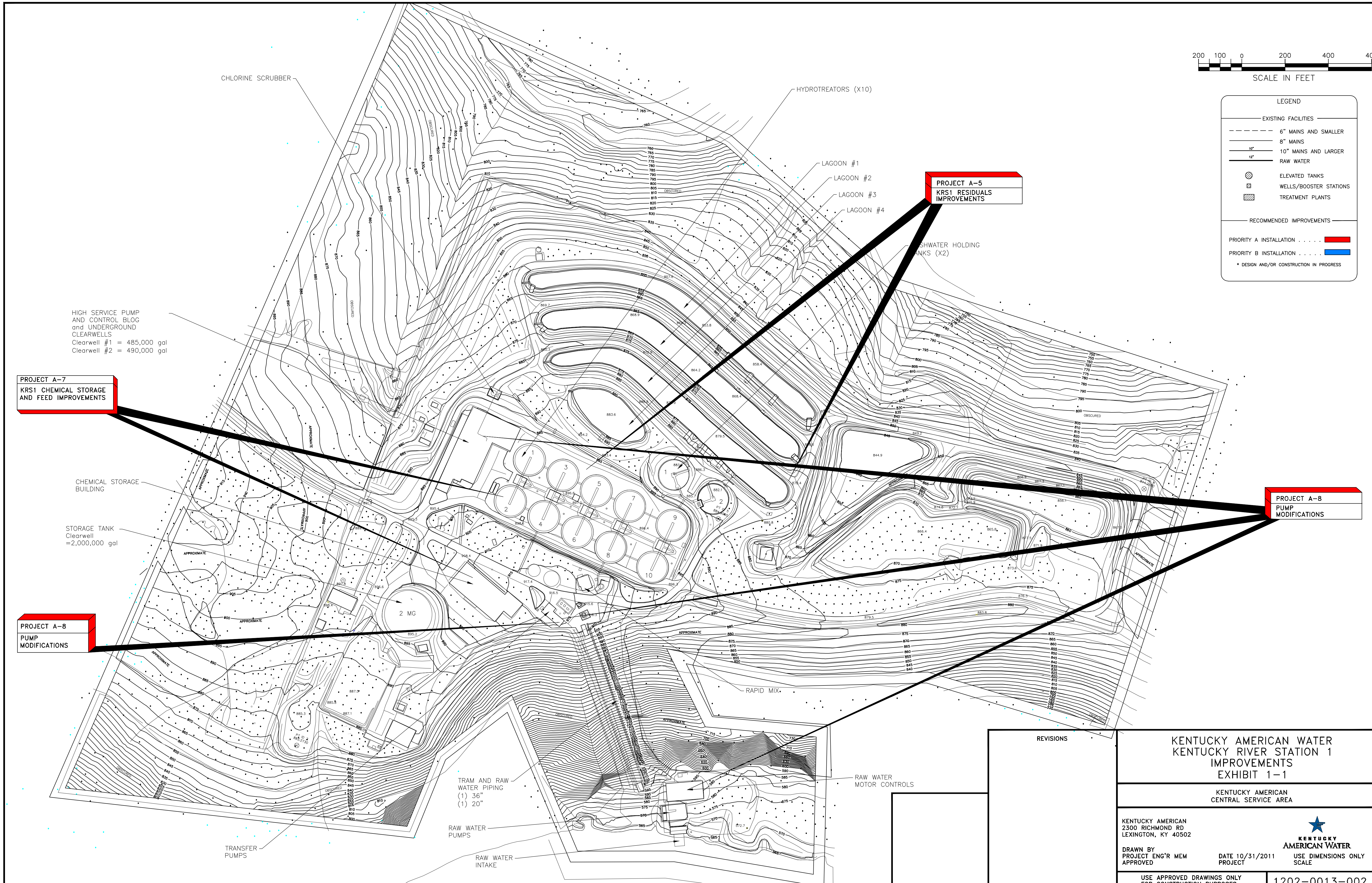
Doing nothing or maintaining lower replacement rates could increase the number of outages in the system; decrease water quality, fire protection, and pressures; result in customer complaints due to the lack of maintaining a reliable system; and will result in an unsustainable pipeline distribution network to maintain adequate levels of service for the long-term future.

Budget Discussion:

Costs include installed materials, plus 30% for legal, engineering, admin, AFUDC, overhead and permitting, and 20% for contingency, outlined in **Appendix B**.

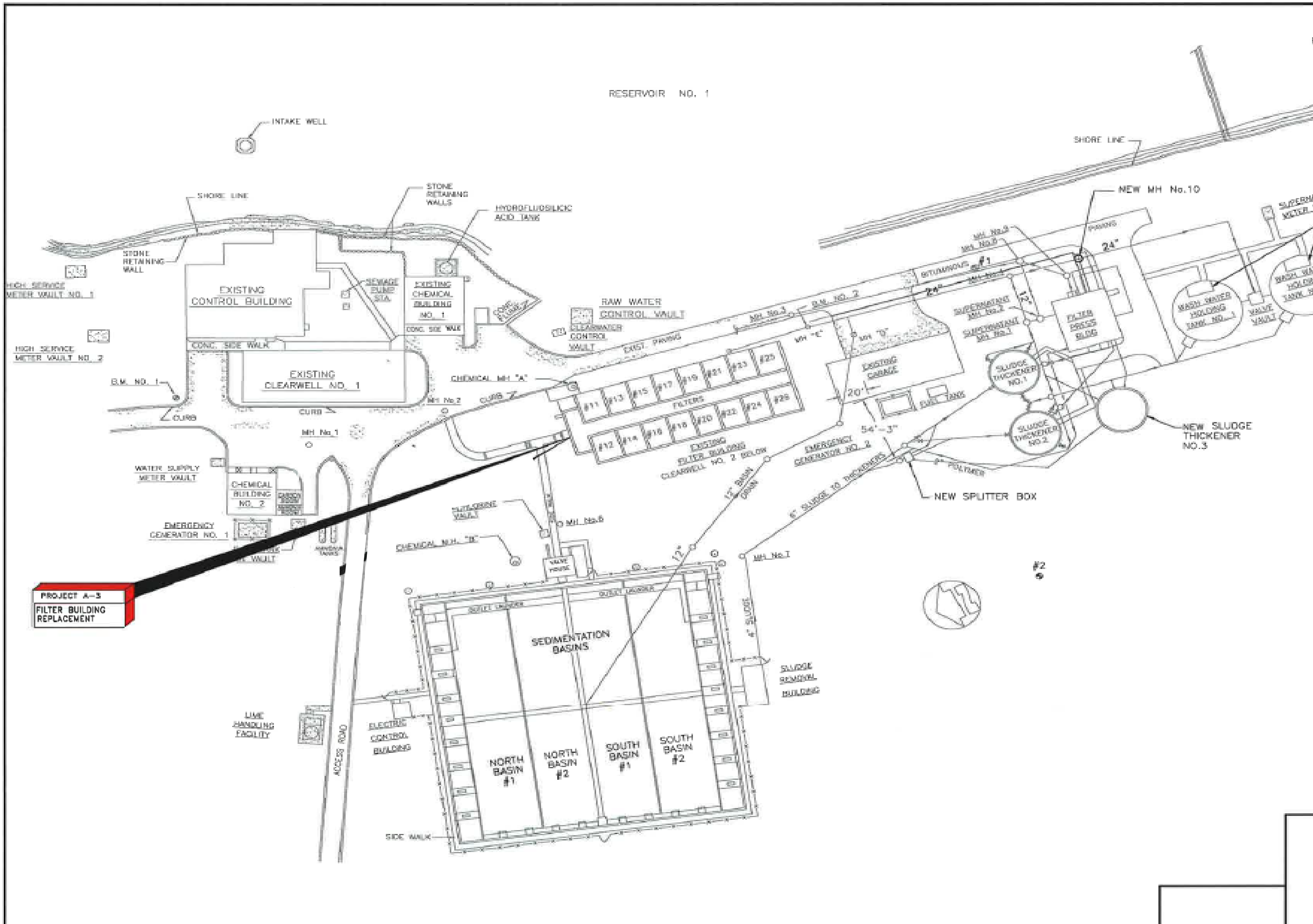
Purpose Codes and Drivers:

Asset Type	%	Purpose Code	%
331 - Trans & Distribution Mains	100	Rel/Qual customer (pres taste etc)	100



FOR COMMENTS

12020013002

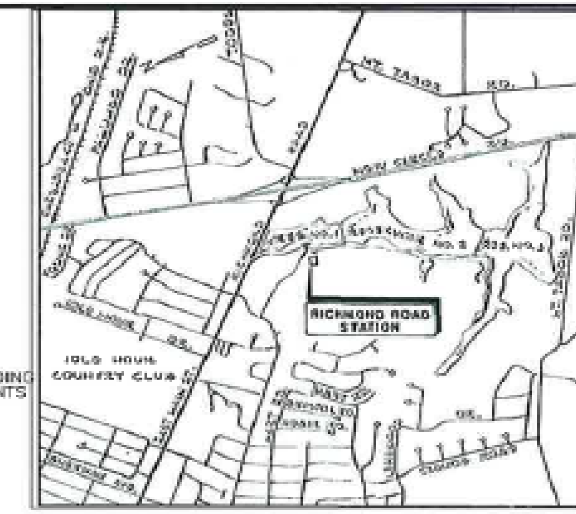


SITE PLAN

1" = 40'

40 0 20 40 80 120 160

SCALE IN FEET



LOCATION PLAN

1" = 2000'

NOTE:

1. TOPOGRAPHIC SURVEY PREPARED BY WILLIAM H. RANNIS & ASSOCIATES DATED JULY 1965 A.W.W. SERV. CO. INC. DWG Nº 350-524.
2. ALL ELEVATIONS SHOWN ARE U.S.G.S.
3. BENCH MARK Nº 1-DISC SET IN CONC. ELEV. 983.77.
4. BENCH MARK Nº 2- PAINT MARK TOP OF WALL ON SOUTH EAST CORNER OF STORM SEWER M.H."E" EL. 984.05'.

LEGEND

EXISTING ITEMS & PIPING TO BE REMOVED

EXISTING ITEMS & PIPING TO BE ABANDONED

EXISTING ITEMS & PIPING TO REMAIN IN SERVICE

ITEMS & PIPING TO BE INSTALLED

REVISIONS

KENTUCKY AMERICAN WATER
RICHMOND ROAD STATION
IMPROVEMENTS
EXHIBIT 1-2

KENTUCKY AMERICAN WATER
LEXINGTON WATER SERVICE AREA

AMERICAN WATER ENGINEERING
3906 CHRUCH ROAD
MT. LAUREL, NJ 08054

DRAWN BY R. BEATTY
PROJECT ENG'R M. McDONALD DATE 10/17/2011
APPROVED PROJECT IP

KENTUCKY AMERICAN WATER

USE DIMENSIONS ONLY
SCALE 1" = 100'

USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES

1202-0013-004

FOR COMMENTS

12020013001

Exhibit 1-3
Kentucky American Water
Comprehensive Planning Study - Central Distribution System Improvements

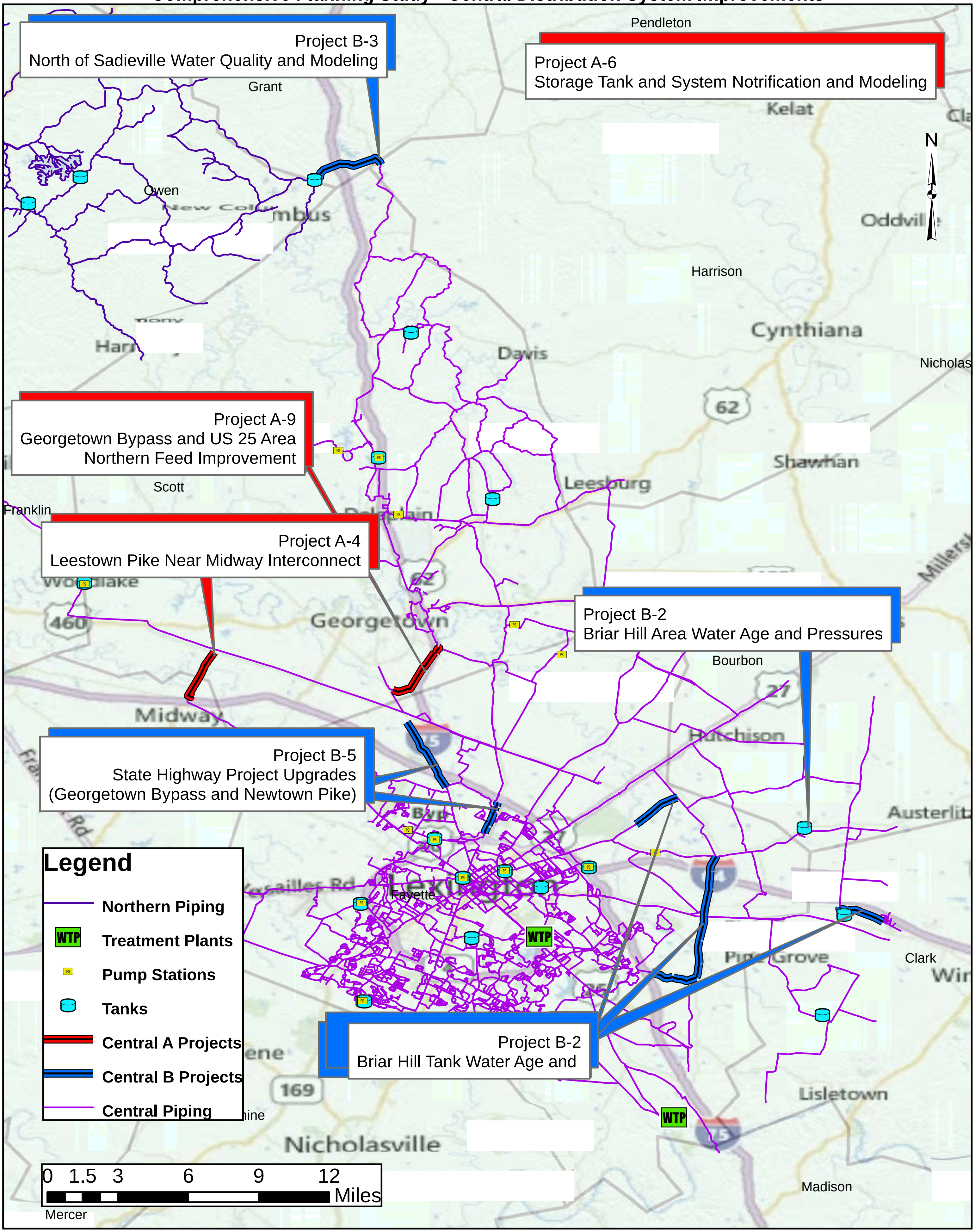
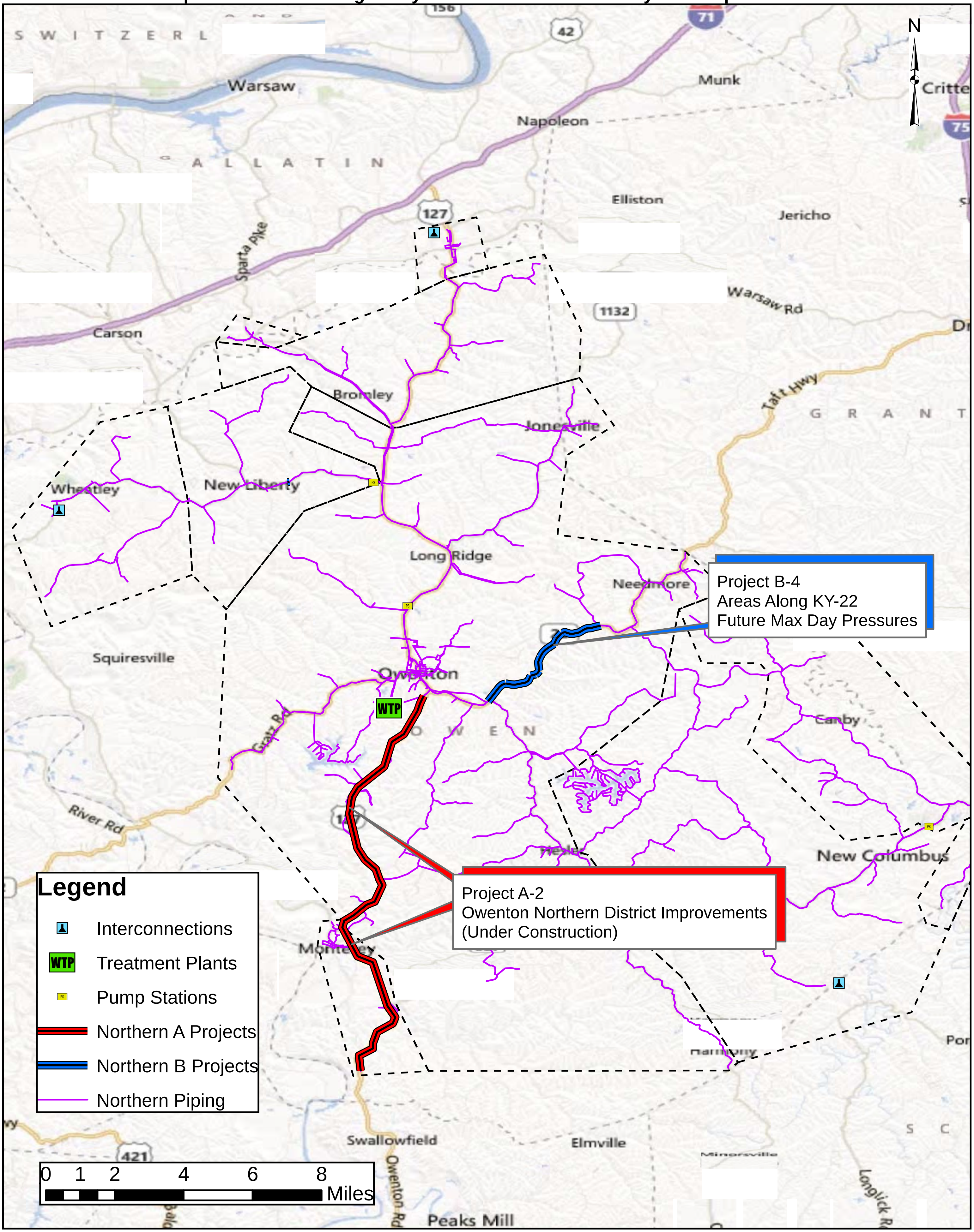


Exhibit 1-4
Kentucky American Water
Comprehensive Planning Study - Northern Distribution System Improvements



SECTION 2
COMPREHENSIVE PLANNING PROCESS

2.1 OVERVIEW

This Comprehensive Planning Study recommends capital improvements that are necessary in order for Kentucky American Water (KAW) to continue to provide safe, adequate and reliable service to its customers. The improvements will also ensure that KAW will continue to supply domestic, commercial and industrial customer demand; meet federal, state and local regulatory requirements; and provide fire protection capability. The criteria used for evaluating the various system components are summarized in the following subsections. In addition, water resource management, national, state and local trends, and their role in the planning process are discussed.

A brief overview of the planning criteria utilized is provided in this section. Specific details regarding the criteria and applicable regulations are contained in **Appendix A**. Recommendations included in this Comprehensive Planning Study address improvements that contribute toward meeting the planning criteria described below. Improvements are also recommended in this report where structural or mechanical problems with existing facilities are evident.

The purpose of this Comprehensive Planning Study is to provide an engineering analysis which management can utilize, among other tools, to assist in the long-term planning process and operation of the company. The priorities and recommendations contained herein are based on conditions that exist and are known as of the date of the report and should not be construed as a recommendation as to the appropriate management decision with regard to implementation of the recommendations at any particular time. Any such management decisions must also consider a wide range of additional and current factors, which are beyond the scope of this report, including, but not limited to general economic conditions, changing regulatory and environmental requirements, and customer impacts.

It is beyond the scope of this Comprehensive Planning Study to attempt to identify the end of the useful life of each piece of Water Company equipment — for example, the many miles of pipeline within a distribution system. It should be assumed that capital expenditures will occur over time due to normal aging and operational wear on existing equipment. For this and various other reasons, it is anticipated that the Water Company may encounter additional capital expenditures beyond those identified in this Comprehensive Planning Study.

2.2 ENGINEERING CRITERIA

In planning the needed water facilities, accepted engineering standards and practices have been utilized to evaluate facilities. Using these standards and practices to evaluate the following areas, an assessment is made to determine if adequate capacity and an appropriate level of reliability are present for domestic, commercial, industrial and fire protection needs. **Appendix A**, Planning Criteria, provides a more detailed discussion of the criteria and regulations used in the evaluation of the water facilities.

2.2.1 Customer and Demand Projections

Demand projections provide the basis for evaluating future system needs. Projections of the total number of customers and their associated demands are developed for the water system over a fifteen year planning horizon for this study.

Since each water system is unique, the specific techniques used to project both customers and demand varies, as appropriate. In general, the projections are developed based on a review of population trends, historic customer and demand data, and local planning commission forecasts. The effects of water conservation are considered in the demand projections along with the analysis of historic water consumption trends. A base growth demand scenario was considered as the most likely projection of demand for this study.

2.2.2 Sources of Supply

Water Company sources of supply should have the necessary quantity of water to reliably meet the projected system demand, even in the event of failure/malfunction of one unit of mechanical equipment. The quality of the water from source of supply is regularly monitored and should provide finished water after treatment that complies with all Federal and State regulations. Sources of supply should also have sufficient allocation rights to permit average and maximum demands to be met. For this reason, build-out demands were evaluated to determine supply adequacy in advance of allocation needs as the permitting process can last multiple years and could delay the development of new sources of supply.

The Water Company conducts water resource management activities and programs that are designed to protect, maintain and monitor the efficient use of supply sources and the finished product. These measures include managing water resources from both the supply and demand side. Continuation of these practices will assist in providing high quality service to the customers.

2.2.3 Water Treatment Facilities

The goal of the Water Company is to continue to produce high quality water that meets or surpasses Federal and State water quality standards. Treatment facilities are designed to meet projected maximum day demands and to comply with water quality regulations at all times. Individual components are sized with appropriate standby capacity that allows the facilities to meet maximum day demands under varying operating conditions.

Recommendations for capital improvements are developed after evaluating the Water Company's ability to provide a reliable and high quality water supply. This ensures continued compliance with existing and anticipated federal and state water quality and environmental regulations, and the ability to meet projected customer demands.

The ability to provide continuous service during a power outage is critical to a system's reliability and depends on several factors including: the nature of the electrical service (i.e., service from one vs. two substations), the presence of any floating storage within a pressure zone, standby electrical generating capacity, and the availability of pumps which can be driven by diesel fuel or natural gas.

The Partnership for Safe Drinking Water is a voluntary cooperative effort between USEPA, American Water Works Association (AWWA) and other drinking water organizations to help ensure the safety of America's drinking water. As promoted by AWWA, "The Partnership provides a new measure of safety by implementing prevention programs where legislation or regulation do not exist. The preventative measures are based around optimizing plant performance, and thus increasing protection against microbial contamination in America's drinking water supply."

2.2.4 Pumping, Water Distribution And Storage

The analysis of Water Company facilities includes an evaluation of pipelines, storage tanks, booster stations, and emergency power provisions. These distribution system components are analyzed to determine their ability to provide safe, adequate and reliable service to customers under forecast conditions.

Pumping facilities are designed to meet projected maximum day demands with the largest single unit assumed out of service. This design standard provides an appropriate measure of reliability in the event of a mechanical failure, or if maintenance is required for a pump. Pumping facilities may also be an important component of the fire protection system.

Pipelines are designed to provide adequate working pressures in the distribution system under normal conditions of flow. The minimum service pressure is established at 30 psi. Pipelines are also designed to maintain a minimum pressure of 20 psi at all points in the distribution system under all demand conditions. Fire flows may impose the heaviest demand on the piping network, and should be considered when sizing new mains. Water quality, fire flow delivery and local pressure limitations are also considered in the analysis of distribution system pipelines, where applicable.

Distribution storage facilities are designed to provide the recommended volume of water to equalize the pumping rate at a treatment plant or booster station during the projected maximum day demand event. The volume of water necessary for fire protection needs is also evaluated. Additionally, on a site-specific basis, state regulations on storage volume are evaluated, and storage facilities may need to provide a reserve volume for reliability purposes in the event of a power failure, main break or other emergency.

2.3 THE WATER COMPANY'S ROLE IN REGIONALIZATION

Regionalization can often provide economies of scale, avoid duplication of facilities, and provide more effective service to customers. For example, water systems within a specific geographic area can regionalize to benefit from shared sources of supply, treatment facilities or distribution system facilities. Likewise, the availability of interconnections between water systems can improve reliability and enhance fire protection.

Regionalization opportunities are evaluated to determine if a consolidated solution to water supply problems in a particular area is feasible, or if management services opportunities are viable. In the case of management services, expertise within the Water Company can be utilized to improve other area water supplies and benefit the State's residents.

2.4 CONSIDERATION OF NATIONAL, STATE, AND LOCAL TRENDS

In developing a Comprehensive Planning Study, it is beneficial to review national, state and local trends that can affect future planning. Nationally, there has been a strong trend toward increased and tougher regulations affecting water purveyors. Examples include: increased protection of sources of supply; more stringent water quality regulations of finished water; additional regulation of treatment plant residuals; increased frequency of required water quality monitoring; increased water and energy conservation requirements; and more extensive environmental laws affecting new construction and source development.

Typically these new regulations are passed down from the federal to the state level to be incorporated as state laws. Often, states have passed additional legislation that address issues specific to their individual circumstances, and can be more stringent than federal laws. Many states, including Kentucky, have primary enforcement responsibility (primacy) of the drinking water regulations as outlined in the Statutes set by the state.

SECTION 3
DEMAND PROJECTIONS

3.1 INTRODUCTION

This chapter describes the methods, procedures, and the driving factors utilized in developing demand projections for the Kentucky American Water CPS. Accurate demand projections are critical to the planning process. The greater the degree of accuracy in the demand projections, the more closely capital improvements can be tailored to system needs. Underestimating future demands can lead to inadequate source, production, and distribution facilities which have detrimental impacts on the customer service levels. Conversely, overestimating future water demands can lead to oversizing the facilities, which are a financial burden on water consumers.

Kentucky American Water Company is keeping abreast of the most current demand projection techniques through development and use of computer aided forecasting of customer demand. Past trends are analyzed to determine existence of data relationships which can be used along with predicted data causal variables to provide more accurate demand projections. Disaggregation of customer demands into smaller units of homogeneous usage categories is part of the forecasting approach. Sensitivity analyses can now be conducted on any modeled variable to determine what the demand variation will be for each change in the selected variables. Several such analyses were made as part of the development of the demand projections for this CPS.

Creating a model that accounts for all variables affecting water consumption and all information impacting water system growth can be a complicated process that is very difficult. The goal is to analyze demand relationships to the extent that will derive the best possible forecasts, with optimum disaggregation of data gathering. Prospective changes in modeling must always be tested with a sensitivity analysis to insure that the resulting incremental costs of generating such data are justifiable in terms of the estimated improvement in forecasting accuracy.

The present model is a spreadsheet program which allows maximum flexibility in adapting the model to changing patterns of water consumption and "what-if" scenarios that was developed in 1991. Modifications to the program can be made on a continuing basis by Water Company personnel. Improvements and enhancements have been implemented since then on a periodic basis. Performance of the model to date has been very good which depends on the accuracy of the population projections. One significant concern with previous versions of the model was the understatement of maximum day demands during hot and dry weather. Changes have been implemented so that the model provides a more realistic range of maximum day demands by incorporating trends in recent peak usage patterns, and considers a range of potential average day demands which more effectively consider weather variables.

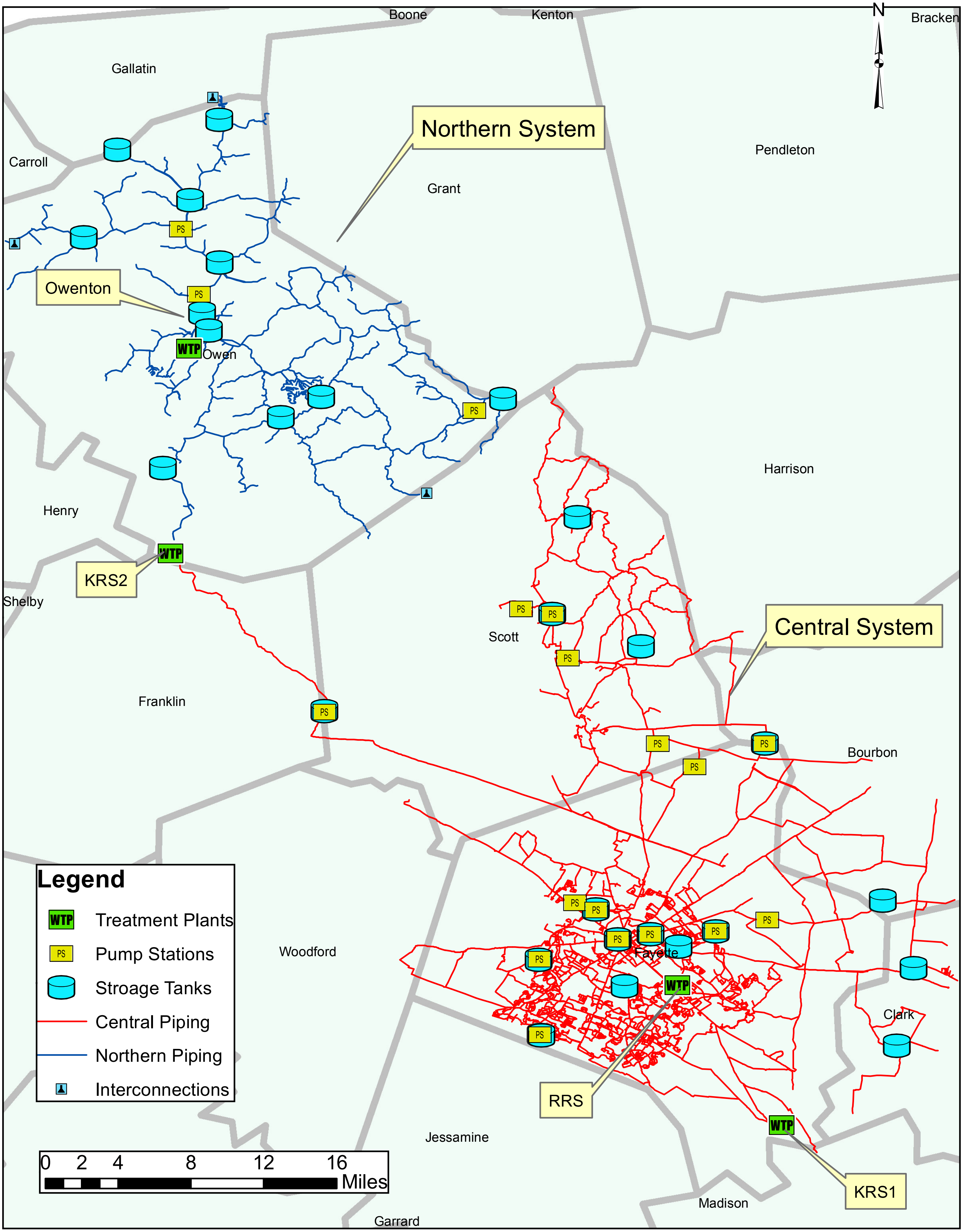
3.2 REGIONAL GROWTH OVERVIEW

The Kentucky American Water Company's primary service area lies mostly in Fayette County, Kentucky. Nearly all of Fayette County is served by the Water Company. In addition, service is provided to parts of six of the seven surrounding counties. Bourbon, Clark, Harrison, Scott, Woodford and Jessamine Counties are supplied in part from Kentucky American Water, either directly to customers or indirectly through sales to other water utilities. One neighboring county, Madison County across the Kentucky River to the southeast, is not served by Kentucky American Water. Fayette County and the surrounding counties can be seen on **Exhibit 3-1**.

County populations are presented in **Table 3-1**, including the projected number of inhabitants through 2030 as obtained from the Kentucky State Data Center at the University of Louisville. Long term economic conditions in the central Bluegrass Region are expected to remain stable with continued growth. The Kentucky State Data Center's population forecast for Fayette County in the year 2020 is 312,190. This would add 26,269 persons to the 2010 census count or a 9.2% gain in the intervening 10 years. The past and projected populations of Fayette County and the surrounding key counties are shown in **Table 3-1**. Each of the counties shown in **Table 3-1** are projected to have a moderate growth rate from 2015 through 2030.

Continued growth, although at rates less than the past, is expected in the residential and commercial water use categories, which can be related directly to population changes. Industrial growth is anticipated to be flat as remaining industrial customers are expected to post gradual small water use increases in those cases where production is expanded. Industrial water conservation and reuse are already fairly well implemented but will continue by most of the large

Exhibit 3-1
KAW Distribution System And Surrounding Counties



industrial users through prior efforts to reduce municipal sewer charges, which are based on water usage.

In summary, the prediction of continued increases in population in key counties should result in an overall favorable business climate. Water needs are expected to moderately grow, reflecting the population trends but tempered by more conservation effects as the use of low-flow and high efficiency fixtures continues and as customers learn to become more efficient in their water use.

3.3 DEMAND PROJECTION MODEL

This section describes the methods and procedures utilized in developing water demand projections for this CPS for the Kentucky American Water Company. Significant improvements and refinements have recently been added to update the demand model and achieve a better analysis of past demand trends and more accurate future demand forecasts.

As part of the 1992 CPS effort, Kentucky American Water Company retained Brown & Caldwell Consultants to provide expertise in the development of the demand projections. Brown & Caldwell recommended enhancements to improve demand modeling and prediction techniques which were implemented. These included changes in price elasticity calculations, development of weather normalization data to separate the effects of unusually hot and dry or cool and wet conditions from the base scenario, and refinements in the analysis and prediction of the relationship between average day and maximum day demands. Water use in the single family residence category was disaggregated into indoor usage and outdoor usage to better predict residential normal base usage versus residential peak usage. Price elasticity relationships were revised to include the effects of sewer costs in addition to water costs alone, and to adjust for inflation.

In 1999, the demand model was further refined to improve the estimated effects of low-flow fixtures. The demand model now generates a base case projection for average day demand. Then a projection is made for extreme weather conditions that provides a maximum day demand with a 95% confidence interval.

The effects of future conservation applications can also be analyzed using the demand model. Conservation will be discussed separately in a later section of this report.

3.4 DEMAND FORECASTS

Demand projections for the Kentucky American Water system are made with the aid of a computerized spreadsheet model. The model is driven primarily by population forecasts. The parameters used to predict water growth in the various disaggregate water use categories are outlined in the following paragraphs.

3.5 CENTRAL SYSTEM DEMAND

3.5.1 Single Family Residential Usage

The current model uses the population data, together with the actual single family residential usage data, to calculate historic gallons per capita per day (gpcd) usage. The model tracks usage back through 1987 and shows a steady decline of per capita use for single family residential water use within Fayette County. The five year average unit usage was then adjusted for price elasticity including factors for the Consumer Price Index inflation rate and unit price increases in water and sewer rates. Separate elasticity factors were applied to indoor and outdoor usages. Per capita usage for single family residential customers is projected to be approximately 78.91 gpcd through 2030 with new single family homes having an average per capita usage of 68.45 gpcd. Based on projected growth trends, the resulting single family residential customer average demand projection for Fayette County is an increase from 13.58 mgd in 2010 to 14.56 mgd by the year 2030. The single family residential usage category represents approximately 34% of the 2030 average day demand and is the largest single usage category in the system.

3.5.2 Apartment Usage

Historic apartment usage is taken from billing information for multi-family residential units, garden apartments, and high rise apartments. Total apartment per capita usage is calculated based on the estimated apartment population taken from 2010 census data and pro-rated through 2030. Elasticity considerations are accounted for in a similar fashion as for single family residential usage. Based on projected population increases, apartment usage is estimated to increase from 5.71 mgd in 2010 to 7.37 mgd in 2030.

3.5.3 Commercial Usage

The commercial usage category includes restaurants, motels, hospitals, laundries, farms, golf courses, car washes, and other similar types of water users. Historical commercial usage is based on the actual commercial metered usage less apartment usage. It is assumed to increase in direct proportion with the increase in overall population. Elasticity considerations are incorporated into commercial usage in a manner similar to that used for residential usage. The commercial use is projected to increase from 8.69 mgd in 2010 to 11.09 mgd in the year 2030.

3.5.4 Industrial Usage

The overall industrial usage is also projected to increase in the future. Industrial usage can be significantly impacted by Toyota. Toyota is the largest industrial customer in the service area. In 2010, Toyota usage averaged 0.87 mgd, which represents 57% of the total industrial demand. Therefore, for purposes of analysis, Toyota has been separated from the balance of the industrial demand component.

Industrial demand is projected to increase slightly, from 1.55 mgd in 2010 to 1.69 mgd in the year 2030. Price elasticity has not been applied to industrial use. The large industrial users have indicated that recycling and conservation efforts have already been implemented to reduce water and sewer costs and will continue to be explored as they are deemed cost effective.

3.5.5 University of Kentucky

The University of Kentucky is queried annually as are the top industrial customers. Information obtained in recent years indicates that the number of students living on campus is expected to grow through 2020. Because accurate student numbers are difficult to track, consumption is disaggregated into two categories, basic usage on the main campus and per student use. Consumption is projected to remain constant for the foreseeable future. Increases in student enrollment and dormitories are expected to be offset through continued efforts of efficiency on campus.

3.5.6 "Other Public" Usage

This category of consumption in the projection model includes municipal facilities, schools, and other public type users. "Other Public" usage is projected to increase slightly from 1.15 mgd in 2010 to 1.37 mgd in the year 2030 with population projections.

3.5.7 Surrounding Counties Usage

Parts of five neighboring counties are served by the Central System, either directly to individual customers on Water Company main extensions or through bulk sales by Kentucky American Water to other water purveyors. Demand projections are separated into two categories, individual customers and bulk water sales. Individual customers are served in Woodford, Scott, Bourbon, Jessamine, and Harrison Counties. Bulk sale customers are served in Woodford, Jessamine, Bourbon, Scott, Franklin, and Harrison counties.

Water demand projections for each of the above listed counties are modeled at growth rates parallel to the projected population growth in each county. The combined bulk sales are expected to grow slightly from 1.20 mgd in 2010 to 1.58 mgd in the year 2030.

Individual customers served by Kentucky American Water in surrounding counties are expected to grow from 1.44 mgd in 2010 to 1.98 mgd in 2030.

3.5.8 Losses and Non-Revenue Use

This usage category includes unmetered uses such as sewer flushing, street cleaning, main flushing, fighting fires, leaks, and metering errors or inaccuracies. Kentucky American Water maintains an aggressive program of leak detection and repair, and estimating and recording known but unmetered consumption in an effort to account for the maximum possible quantity of non-revenue uses. Over the past five years, total combined unaccounted-for water and non-revenue usage has varied between 18.8% of total system delivery in 2006 to 12.03% in 2011. Unaccounted-for water can be expected to vary from month to month and year to year. Kentucky American Water will continue an aggressive program to control unaccounted-for water to as low a level as possible. The demand projections, however, must not be based on projected unaccounted-for water levels that are too low to be reliably achieved and sustained even with ongoing leak detection activities. In the base case projection contained in this report, unaccounted-for water is projected at 12.0% through the planning horizon. Non-revenue usage is projected a 1.8 %.

3.5.9 Total Demand

The various categories of disaggregated consumption for the Kentucky American Water System are shown in **Table 3-2**. For purposes of this study a 20-year planning period is used to size long range facility needs such as production and distribution facilities, with emphasis on the first three years ending in 2015. The population projections of the Kentucky State Data Center were made in 2010 every 5 years to 2050.

The key projection year average day forecasts for the CPS are shown in the model. The base case is forecast with an average consumption. A second forecast appears in the **Table 3-2** under the "Hot and Dry Scenario". The demand projections contained in this report do not include potential acquisitions or other regionalization programs. If Kentucky American Water were to provide service to or acquire additional water systems, the demand projections shown here must be modified accordingly.

Weather normalization was used to prepare the base case projection of average day consumption. Analysis of hot and dry years revealed an average consumption which exceeded normal years by a factor of 6%. The future hot and dry scenario projection therefore has average days 6% above the base case.

3.5.10 Plumbing Code Change

In 1988, with Company support, the Kentucky State Plumbing Code was amended to require 3.5 gallons per flush (gpf) or less "water saver" toilets (not to be confused with the 1.6 gpf or less "low-volume" or "ultra low flow" toilets), 1.5 gpf or less urinals, 4.0 gallons per minute (gpm) or less showerheads, and lavatory and sink faucets that use 3.0 gpm or less cold water and 0.75 gpm of hot water. The provision applies to fixture installations in new construction, renovations, and replacements in existing structures. In 1996, the national plumbing code standards were implemented.

The total estimated water savings to date in the Kentucky American Water System due to the new plumbing code is estimated to be between 1.3 and 3.2 mgd, based on an annual fixture replacement rate of 2 to 5%. Annual water savings can be expected to continue for another 0 to 20 years, at a rate that tracks population growth and new housing starts.

Recent years have seen an advent of high efficiency appliances that are continuing to reduce water consumption in dishwashers and washing machines. Beginning in 2020, these efficiencies are rolled into the per capita usage as part of the trend of water use.

3.5.11 Projected Demands

Overall, each of the counties are projected to have a moderate growth rate from 2015 through 2030, similar to historical growth, except Jessamine and Scott counties, which are expected to show a higher level of growth of 34% and 15%, respectively. As shown on **Exhibit 3-1**, Jessamine County and Scott Counties are located approximately 10 miles south and north of Central Lexington, respectively. The main service area in Fayette County has a 14% projected growth, which could result in approximately 2 to 4 mgd additional demand during average and max day conditions.

The demand projection ranges for the average day are shown in **Table 3-3**, along with corresponding maximum day projections for 2015 through 2030. The base case projection average day has a maximum day calculated from the 95% confidence factor of the past ratio of maximum day to average day demand. It assumes 12% unaccounted-for water. The "prediction interval" cases shown in the Table utilize a regression curve fitted to a plot of 30 years of historic maximum day vs. average day ratios, and a statistical technique to predict the 95% reliability range above and below the fitted regression curve. This technique proved statistically more appropriate than the base case maximum day calculation using the 95% reliability derived from a normal curve distribution. The hot and dry prediction is shown, using the base case ratio for maximum day to average day. The average summer demand is calculated along with a drought average day demand under hot, dry conditions.

Table 3-3 Central Demand Projections						
	2015			2030		
Demand Scenario	Avg. Day	Max. Day	Drought Avg. Day	Avg. Day	Max. Day	Drought Avg. Day
Normal Weather	41.74	73.33		47.17	82.44	
Demand with Hot/Dry Weather	44.31	77.85	58	50.08	87.35	65

3.6 NORTHERN SYSTEM DEMANDS

In 2010, the Northern System had an average day demand of approximately 1.0 mgd and a maximum day demand of 1.5 mgd. Potential future system demand conditions were based on the trends that were observed by the KAW staff within the Northern System since it was acquired in 2005. It is assumed that the 2025 average day and maximum day demands for the Northern System will increase to 1.3 mgd and 2.0 mgd, respectively. This demand is assumed to continue through 2030. Just like the Central System, it is also assumed that the future hot and dry scenario projections are approximately 6% above the base case.

3.7 SUMMARY OF DEMAND PROJECTIONS

As described above, the average day demand in the Central System are expected to increase from 40.73 mgd in 2010 to 46.19 mgd in 2030, with maximum day demands increasing from 61.36 mgd in 2010 to 82.44 mgd in 2030. Likewise, the average day demand in the Northern System is expected to increase from 1.0 md in 2010 to 1.3 mgd in 2025, with the maximum day demands increasing from 1.5 mgd in 2010 to 2.0 mgd in 2025. It is projected that the average and maximum day demands will remain constant in the Northern System from 2025 to 2030. A summary of historic and projected demands is presented in **Table 3-2** and **Exhibit 3-2**.

3.8 DEMAND MANAGEMENT

The demand projections form the basis for an initial analysis of the adequacy of source of supply, production and distribution facilities. As will be shown in **Section 4**, Kentucky American Water has adequate source of supply, production, and storage facilities through the planning horizon to meet existing and projected demands.

In 2008, Kentucky American Water retained Strand Associates to review its conservation program, to evaluate it against best management practices, and recommend changes for implementation.

The demand model has been used to predict average, maximum, and drought average demand within the planning horizon. Although the predictions are shown through 2030, the projected demands may be reduced or increased in the future through additional trends in water use, conservation programs, pricing, and population. Therefore, the model should be routinely updated and changes to the model considered as necessary. These updates should be compared to the systems current supply and productions facilities to ensure there is adequate supply in the system.

Table 3-1
Population Growth

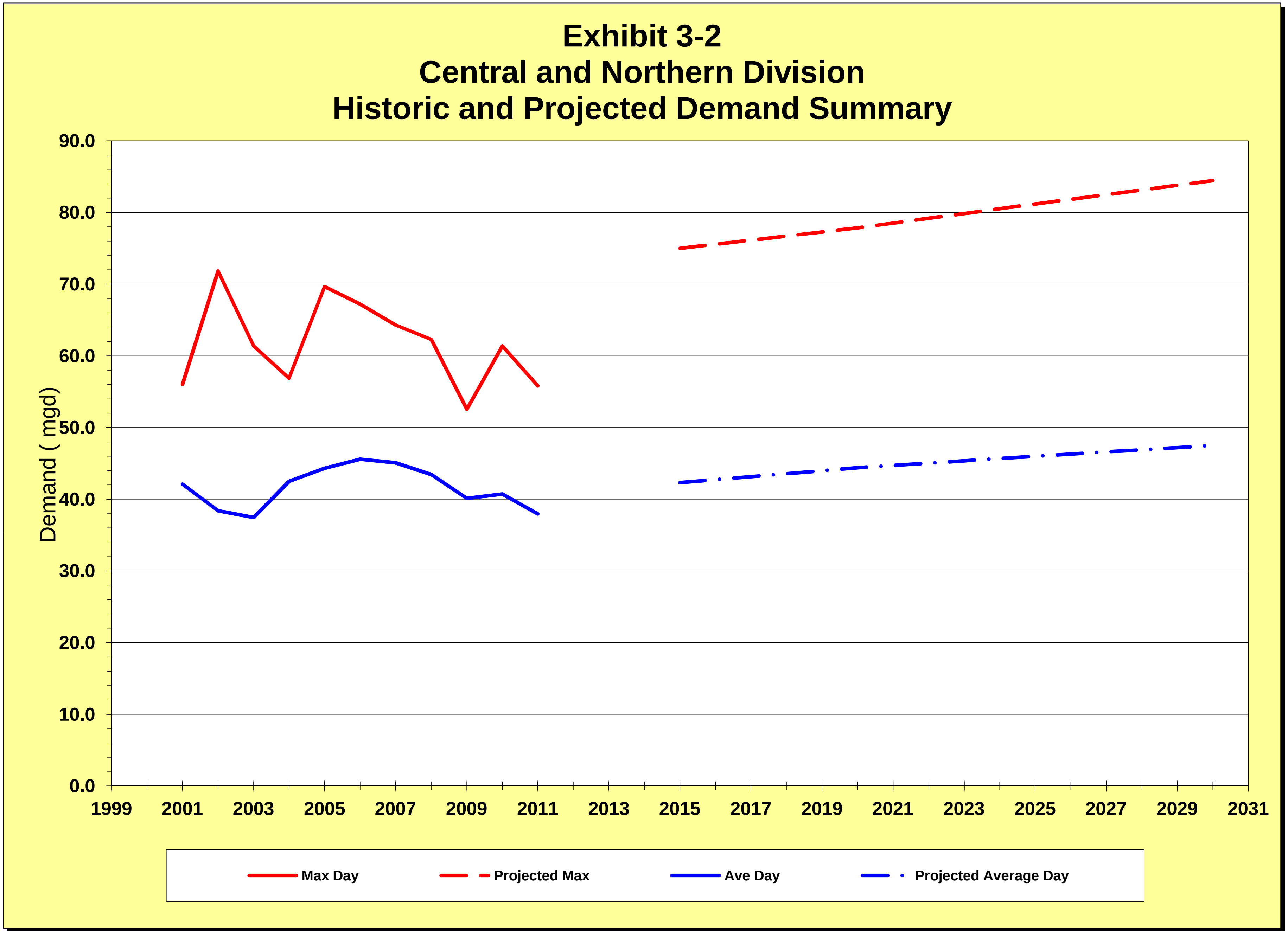
County	Percentage Served by KAW	1940	1950	1960	1970	1980	1990	2000	2005	2010	2015	2020	2025	2030
Bourbon	11%	17932	17752	18178	18476	19405	19545	19360	19721	19906	20258	20586	20854	21039
			-1.00%	2.40%	1.64%	5.03%	0.72%	-0.95%		2.82%		4.39%		3.86%
Clark	10%	17988	18898	21075	24090	28322	29790	33144	34638	36361	38008	39611	41151	42487
			5.06%	11.52%	14.31%	17.57%	5.18%	11.26%		9.71%		14.36%		11.78%
Fayette[1]	100%	78899	100746	131906	174323	204165	229367	260512	271540	285921	299052	312190	326973	341326
			27.69%	30.93%	32.16%	17.12%	12.34%	13.58%		9.75%		14.97%		14.14%
Franklin[2]	NA	23308	25033	29228	32949	41830	43781	47687	48388	49203	49833	50320	50740	51085
			7.40%	16.76%	12.73%	26.95%	4.66%	8.92%		3.18%		3.99%		2.51%
Gallatin	8%	4307	3969	3818	3992	4842	5393	7870	8040	8214	8513	8811	9069	9330
			-7.85%	-3.80%	-4.56%	21.29%	11.39%	45.93%		4.37%		9.59%		9.60%
Grant	1%	9875	9809	8754	9485	13308	15787	22384	24429	26325	28516	30851	33356	35938
			-0.67%	-10.76%	8.35%	40.31%	18.63%	41.79%		17.61%		26.29%		26.03%
Harrison	2%	15124	13736	13704	14158	15166	16127	17983	18196	18750	19178	19590	19958	20267
			-9.18%	-0.23%	3.31%	7.12%	6.34%	11.51%		4.27%		7.66%		5.68%
Jessamine	0%	12174	12458	13625	17430	26146	31436	39041	43175	48615	54202	60051	66227	72347
			2.33%	9.37%	27.93%	50.01%	20.23%	24.19%		24.52%		39.09%		33.48%
Madison[2]	NA	28541	31179	33482	42730	53352	57730	70872	78647	84586	92602	101021	110278	119242
			9.24%	7.39%	27.62%	24.86%	8.21%	22.76%		19.35%		28.45%		28.77%
Owen	56%	10948	9755	8196	7248	8924	9035	10547	11171	11603	12175	12767	13357	13885
			-10.90%	-15.98%	-11.57%	23.12%	1.24%	16.73%		10.01%		14.29%		14.05%
Scott	30%	14314	15141	15376	17948	21813	23389	33061	39293	47249	56112	66411	78759	92613
			5.78%	1.55%	16.73%	21.53%	7.23%	41.35%		42.91%		69.01%		65.05%
Woodford	2%	11847	11212	11913	14434	17778	20254	23208	23931	24790	25440	25992	26405	26685
			-5.36%	6.25%	21.16%	23.17%	13.93%	14.58%		6.82%		8.61%		4.89%

[1] Fayette County Persons Per Household factor was lowered to set percentage of population served by KAW to 100%.
[2] Data not available.

Table 3-2
Historic and Projected Demand Summary

Demand Scenario	Historic				Projected			
	All Time[1]	2000	2005	2010	2015	2020	2025	2030
Normal Weather								
Average Day Demand - Central Division		41.02	44.30	40.73	41.22	43.19	44.68	46.19
Average Day Demand - Northern Division		N/A	N/A	1.00	1.10	1.20	1.30	1.30
Average Day Demand - Total		41.02	44.30	41.73	42.32	44.39	45.98	47.49
Maximum Day Demand - Central Division		66.37	69.65	61.36	73.33	76.03	79.17	82.44
Maximum Day Demand - Northern Division	71.82	N/A	N/A	1.50	1.67	1.83	2.00	2.00
Maximum Day Demand - Total		66.37	69.65	62.86	74.99	77.87	81.17	84.44
Hot, Dry Scenario								
Average Day Demand - Central Division		-	-	-	44.31	46.02	48.01	50.08
Average Day Demand - Northern Division		-	-	-	1.17	1.27	1.38	1.38
Average Day Demand - Total		-	-	-	45.48	47.29	49.39	51.46
Maximum Day Demand - Central Division		-	-	-	77.85	80.67	83.94	87.35
Maximum Day Demand - Northern Division		-	-	-	1.77	1.94	2.12	2.12
Maximum Day Demand - Total		-	-	-	79.62	82.61	86.06	89.47

(1) All time high Aug 5, 2002.



Note - Historic Demand does not include the Northern Division. Northern Division only included in future projections.

SECTION 4
SOURCE OF SUPPLY

Raw water for the Kentucky American Central Division system can be obtained from three sources: the Kentucky River, Jacobson Reservoir on East Hickman Creek and Lake Ellerslie on West Hickman Creek. The Kentucky River is the predominant supply of raw water for the Kentucky American system. The Kentucky River is utilized at Pool 9 and at Pool 3. About 80 percent of the daily consumption is obtained from the river.

Raw water for the Kentucky American Northern Division system can be obtained from two sources: Severn Creek near the confluence of the Kentucky River in Pool 2, and Lower Thomas Lake, located at the Owenton WTP.

4.1 INTRODUCTION

The need for additional raw water supply in the Central Division was recognized as early as 1986 in the CPS. In 1988, a dry weather period produced very low flows in the river during the early summer months which dramatically emphasized the need to supplement existing sources of supply. However, the Pool 9 level had fallen below the crest of the dam for eleven days. Leakage through the locks and dams was exposed and the need for how to best address the raw water source of supply became a community issue for two decades.

Minimum passing flows were introduced into the Commonwealth of Kentucky's water permitting policies which control allowable withdrawals from the river in dry weather periods. These passing flows are meant to help minimize the impact on aquatic life while providing available flow for any downstream users as well.

In 2010, Kentucky American addressed its decades old deficit of raw water supply through a second intake on the Kentucky River at Pool 3. This project was the solution proposed from a regional effort to determine the best source for raw water and treatment capacity. Over 75 alternatives were reviewed and the PSC approved a Certificate of Convenience and Necessity application for the project after an exhaustive review that included three separate cases. The approved application was appealed to the Kentucky Supreme Court, although the approval was upheld at every judicial challenge.

The new raw water intake and treatment plant named Kentucky River Station II at Hardin's Landing, was sited in Pool 3 of the Kentucky River to take advantage of the fact that there are no other water withdrawals downstream from that point to the confluence of the Ohio River. If Kentucky American determines in future planning that additional source of water supply is needed, the plant is a mere 19 miles from the Ohio River. However, at this time any need for additional raw water supplies is nearly thirty years into the future.

The raw water supply for the Northern Division has continued to be a problem of water quality since the purchase of the system in 2005. The Severn Creek becomes nearly stagnant during low flow periods, and Lower Thomas Lake, while adequate in volume has significant algae growth which becomes disinfection by-product precursors.

4.2 KENTUCKY RIVER

The primary source of supply for the Kentucky American Water Company is the Kentucky River at Pool 9. The river originates in the southeastern end of Kentucky and traverses a northwesterly path of over 250 miles across the state to Carrollton where it empties into the Ohio River. Water levels in the Kentucky River are established by a series of fourteen dams. Locks were originally provided at each dam for commercial and recreational navigation although a lack of commercial use prompted the Corps of Engineers to turn them over to the Commonwealth of Kentucky, and many of the locks have been permanently closed to maintain the pools. Flows in the river are sustained to some extent by two reservoirs located in the headwaters of the river, Buckhorn Reservoir and Carr Fork Reservoir, which are both operated by the Corps of Engineers. The primary purposes of the reservoirs are flood abatement, water quality and recreation. Minimum release requirements are established for each Reservoir to sustain aquatic life below the dams.

The Kentucky River Authority ("KRA") has initiated efforts to shore up the lock and dams on the Kentucky River. First, the KRA installed release valves in Pools 10-14 upstream to allow water to be transferred from the upstream Pools 12 and 13 that had no users to downstream pools with multiple withdrawers. Then the KRA developed a plan and received funding to replace or repair the dams, beginning with Dams 9 and 3. These two dams have been completed, and focus has turned to Dam 8. Leakage through the dams is significant and could jeopardize maintenance of pool levels in a major drought.

Kentucky American has taken water from the river each year since 1931 following the 1930 drought when the first intake was developed in the river pool formed by Lock and Dam 9. Water from the river at Pool 9 can be utilized at the Kentucky River Station treatment plant on the bluff above the river, or pumped to Jacobson Reservoir or directly to the Richmond Road Station for purification. Water from the river at Pool 3 where the KRS-2 plant was completed in 2010, can only be utilized at that plant.

In mid-1992 the intake pumps were replaced with six new 12.4 mgd pumps designed to deliver water directly from the intake to the Kentucky River Station at the top of the bluff, eliminating a second lift set of pumps at the intake. In 2007 these pumps were replaced. The intake pumps supply 40 mgd to the Kentucky River Station. They also supply up to an additional 22 mgd to a transfer pump station which supplies water to Jacobson Reservoir or directly to the Richmond Road Station. The intake pumps were replaced as part of a reliability project. The combination of water pumped from the river and pumped from Jacobson Reservoir will supply up to 26 mgd to Richmond Road Station.

In 1930, during the drought of record, river flows were extremely low. The average daily flow at the Lock 10 gauging station during the four driest months was 25 mgd. The lowest single day flow was estimated to be 6 mgd. Present day river flows should be somewhat higher in a repeat of the 1930 drought due to augmentation from the Buckhorn and Carr Fork Reservoirs which began operations in 1960 and 1976, respectively. This was demonstrated in 1999, which was a prolonged, extreme drought. In 1999, raw water supplies were further augmented by the transfer of water by the KRA from upstream Pools 12 and 13.

The total available safe yield of the Kentucky River at the Water Company's intake in the Lock 9 pool is the daily flow into the pool during the drought of record plus the daily draft from storage in Lock Pool 9, less any evaporation, outflow and required releases. An analysis of Pool 9 has established the safe yield at 35 mgd. Additional analysis of Pool 2 (downstream of Pool 3) determined the safe yield to be in excess of 30 mgd and as much as 78 mgd.

Dry weather was experienced in the summer of 1988 which emphasized the need for additional water resources. During that drought, and subsequent droughts, the customer response to the Company's requests to reduce consumption was effective in reducing demand. Thus some moderate demand reduction was incorporated into the planning for raw water supplies. In 2007 application for a Certificate of Convenience and Necessity, KAW established a raw water deficit of 28 mgd but was proposing only an additional 20 mgd of raw water and treatment capacity assuming that some moderate demand restrictions would be necessary during a drought of record.

4.3 JACOBSON RESERVOIR

Jacobson Reservoir consists of an impounding dam constructed in 1914 on East Hickman Creek with an estimated gross storage capacity of 619 MG.

The dam is an earth fill structure with a concrete core wall and spillway. The crest of the spillway is at elevation 967.3 feet. Depth-capacity surveys were conducted in 1964, 1977 and 1991 with resulting gross capacities determined at 818 MG, 745 MG and 619 MG respectively. The siltation rates for the interim periods between capacity measurements were 0.25 percent, 0.60 percent and 1.03 percent. The higher siltation rate from 1977 to 1990 reflects construction activities on the watershed. While the sedimentation rate in Jacobson Reservoir will slow relative to recent rates observed during watershed development, the capacity loss due to siltation will likely continue at a higher rate than it would have if the watershed had remained undeveloped grasslands or pasture. The current estimated usable storage capacity is 550 MG allowing for unusable storage in inaccessible pockets, however, estimates from the 1991 study projected that the reservoir capacity could be reduced to 500 MG due to siltation by this time.

The dam at Jacobson Reservoir has been inspected in accordance with the National Dam Inspection Act (Public Law 92-367) administered by the Corps of Engineers. Due to the potential for downstream damage in the event of failure, the dam is classified as having high hazard potential under guidelines for the national program on safety inspection of dams. Modifications have been made to the dam to prevent overtopping by the Probable Maximum Flood. A limited service emergency spillway channel was constructed at the east end of the dam. The earthen embankment was raised 4 feet and the upstream face of the new embankment was lined with rip rap. The changes were made to bring the dam in compliance with existing State and Federal regulations. In addition, the dam is inspected annually by Kentucky American personnel. The program is in compliance with State regulations.

The primary function of Jacobson Reservoir is to supply water to the Richmond Road Station water treatment plant. Rain water runoff from the East Hickman Creek watershed is stored in the reservoir. A pumping station at the base of the dam delivers raw water from Jacobson Reservoir through three transmission mains to Richmond Road Station which is adjacent to Lake Ellerslie. The transmission mains are 16 inch, 20-inch and 30-inch in size.

During peak demand periods Jacobson Reservoir supplies rates up to 26 mgd to Richmond Road Station. When rainfall is insufficient, Jacobson Reservoir can be supplied or supplemented with raw water from the Kentucky River. Transfer pumps relay raw water from the KRS-1 intake pumps through a 30-inch diameter pipeline to the reservoir.

During normal weather patterns Jacobson Reservoir is operated to capture and use rain water runoff. Over the period from 1979 to 1990, Richmond Road Station has pumped an average of 6.70 mgd on an annual basis. An average of 6.53 mgd has been transferred from the Kentucky River to Jacobson Reservoir over a period of 114 days during high demand periods. The remainder has been supplied from Jacobson Reservoir. The supplementary supply available from the river via the transfer pumps and the 30-inch pipeline allows the reservoir to be used as off stream storage to augment river flows during low flow periods.

Prior to the construction of KRS-2, the Kentucky American operating procedures changed when dry weather indicated the possible start of a drought period. When the water level in Jacobson Reservoir dropped, the transfer pumps at the Kentucky River Station were turned on to keep Jacobson Reservoir full as long as possible, but with first priority given to satisfy the raw water requirements of the Kentucky River Station and allocation permit requirements. This procedure ensures that the reservoir is maintained at the highest possible level going into a drought period. With the completion of KRS-2 in October 2010, KAW is still making adjustments to the operating procedures, recognizing the efficiency of continuing to use Jacobson Reservoir as an off stream storage facility.

If the total usable 550 MG capacity of the reservoir was isolated on its own watershed, with no pumpage added from the river, the estimated safe yield of the creek/reservoir supply during the drought of record would be about 1.3 mgd. Following the drought of 1999, KAW determined that its safe yield of Jacobson Reservoir is more than likely 0 mgd. While this does not eliminate it as a valuable operating asset, it is not appropriate to include in long range source of supply planning reliability.

In recent years, use of Jacobson Reservoir has been limited due to water quality concerns. The heavy algal growth has led to a high organic content in the water which is at times difficult to treat without taste and odor complaints. KAW has treated the reservoir to reduce the algal growth; however, recommendations to assess maximum operating efficiency and water quality and quantity of the Jacobson Reservoir are provided in **Section 4.10**.

4.4 LAKE ELLERSLIE

Lake Ellerslie was constructed on West Hickman Creek in 1885. The drainage area of the reservoir is 2.3 square miles and the estimated storage capacity is about 88.7 MG. Low lift pumps with a capacity of 10 mgd are installed in the adjacent Richmond Road Station and pump raw water to the plant's settling basin.

The watershed is small and fully developed with commercial and residential buildings. During dry weather the safe yield is negligible. The reservoir is kept near spillway level and the stored water is reserved as a supplemental, emergency supply. The dam at Lake Ellerslie is partly submerged on the downstream side by an adjacent reservoir. The Lake Ellerslie dam will be scheduled for inspection to determine dam stability and safety if the downstream dam were to fail. KAW completed a project in the 1992 CPS to inspect and evaluate the Lake Ellerslie dam.

4.5 SEVERN CREEK

Severn Creek flows into the Kentucky River at Pool 2, and has a very low flow during dry periods, becoming a backwash of the Kentucky River. Because of this, there can be significant organic and algal growth which can lead to formation of disinfection by products (DBP) and taste and odor issues. Prior to KAW’s purchase of the Northern Division treatment plant in 2005, the City of Owenton entered into a Memorandum of Agreement with the Kentucky DOW to construct a new intake on the Kentucky River at Pool 3 and discontinue use of the Severn Creek intake. In 2005, Kentucky American began operations of the Owenton treatment plant, and through treatment modifications was able to minimize the DBP formation. Thus improvements were made to the Severn Creek intake, but the Kentucky River Pool 3 intake was not constructed.

In 2012, Kentucky American determined that significant improvements were necessary at the Owenton treatment plant. Due to the large capital expenditures required, a review of the alternatives was undertaken. KAW concluded that the best alternative was to connect the Northern Division system to the KRS-2 treatment plant and eliminate the Owenton treatment plant and raw water intake.

4.6 LOWER THOMAS LAKE

The Lower Thomas Lake is adjacent to the Owenton treatment plant. KAW has the ability to transfer water from Severn Creek to the lake, or withdraw directly from the lake. Because of the high algal content, KAW does not significantly utilize the lake. The dam and lake are still owned by the City of Owenton. The earthen dam has very visible deterioration including erosion along the spillway and sinkholes on the face of the dam. KAW has limited technical information on the dam and lake. Following the connection of the Northern Division system to KRS-2, the use of this lake by KAW will be eliminated.

4.7 GROUND WATER

Ground water is not used as a source of supply by the Kentucky American Water Company. The Metropolitan Lexington Urban Study - Water Resources Analysis by the Corps of Engineers, October, 1978, summarizes the ground water situation. Wells in the area generally yield less than 200 gpm even in the more favorable valley locations. The bedrock is limestone which yields hard water when it yields any water at all. Nearly all producing wells in bedrock are less than 100 feet deep because saline or sulfurous water may occur at about 200 feet. Occasional large springs appear in the area such as the Royal Spring used as a source of supply by Georgetown. The Corps of Engineers concluded that only surface water should be considered when developing supplemental water supplies for the Lexington region.

A more recent report entitled "A Reconnaissance of the Ground Water Resources of the Kentucky River Basin", dated January 29, 1991, was prepared by the Kentucky Geological Survey for the Kentucky River Basin Steering Committee. The 32 member Steering Committee was formed in 1989 with the stated goal of developing long range source of supply plans for the

Kentucky River Basin. The Geological Survey report on ground water compiled existing data and reference material to evaluate the potential for new ground water developments to meet future regional water demands. The Steering Committee's Phase 2 Report Entitled "Development of a Long Range Water Supply Plan" summarizes the regional ground water potential: "The potential development of ground water resources to provide a sustainable safe yield that would offset a reasonable portion of the projected water supply deficit is not supported. Although there is the potential for development of individual wells and springs that might serve populations of a few thousand, communities that might benefit thereby are currently served by surface water supplies. Surface water supplies that are developed to provide a safe yield on a regional basis could be developed to serve these communities at a minimal additional cost using the existing water supply infrastructure. Therefore, a long range plan that develops a safe yield from the Kentucky River or another surface water source would also benefit communities that might be served by wells. It appears, then, that in the context of a long range plan for water supply for the mainstream Kentucky River, ground water resource development is only appropriate where a community cannot be economically served by a facility that develops safe yield for other communities as well".

4.8 WATER ALLOCATION PERMITS

The Commonwealth of Kentucky Division of Water (DOW) administers permitted withdrawals from surface water supplies. DOW requirements for Permit No. 0200 only, as revised on September 17, 1999, call for passing flows equal to the minimum average seven (7) day flow with a one in ten year frequency (7Q10). The period used to establish the 7Q10 was after Buckhorn Reservoir began operating in 1960. The 7Q10 at Pool 9 has been calculated as 120 cfs. Permit Nos. 200 and 1572 are not presently affected by a passing flow requirement. All permits are expected to be subject to minimum flow requirements when each permit is brought up for renewal or revision in the future if downstream users are impacted. Under this recently implemented policy, it would be necessary for Kentucky American to enact strict demand management steps to comply with Permit No. 0200 when the river flow is below the 7Q10 for more than four consecutive days, but based on discussions with DOW officials the Water Company would not be required to shut down its water supply intake.

The 16 mgd monthly allocation from Jacobson Reservoir under Permit No. 201 was obtained to cover peak daily withdrawals up to 25 mgd which are needed to fully supply the Richmond Road

Station. To date this permit has been adequate to cover average withdrawals during the maximum month.

The Kentucky American Water Company does not need a diversion permit for Lake Ellerslie. Since this supply is used only during emergency conditions, it is exempt from state water permitting requirements. A letter from DOW to this effect is on file at the Water Company.

4.9 EVALUATION OF EXISTING SOURCES OF SUPPLY

The Kentucky River is the primary raw water source for Kentucky American in the Central Division, and with the connection of the Northern Division to KRS-2, it will become the primary raw water source for all of Kentucky American Water in 2013. Lake Ellerslie has no significant dry weather yield. Jacobson Reservoir contributes during dry weather but is considered to have no safe yield during a drought of 120 day duration or longer. Severn Creek becomes a backwater of the Kentucky River during low flows, and Lower Thomas Lake has no significant drought yield from anecdotal history from the Owenton citizens. The balance of the supply must come from the river.

The 1986 Least Cost/Comprehensive Planning Study assigned a safe yield of less than 29 mgd to the river at the Lock 9 pool based on the Corps of Engineers evaluation of the regional water supplies. Subsequent studies and the ability to release water from upstream pools increased that amount to 35 mgd. As part of the development of the KRS-2 plans in 2006, the safe yield of Pool 2 was determined by Gannett Fleming to be in excess of 30 mgd and as much as 78 mgd.

In 1989, data was not available to assess the true effect of the low flows on aquatic life in the Kentucky River. The Division of Water considered it necessary to impose passing flow criteria based rigidly on the 7Q10 flow, to protect aquatic life and provide sufficient flow for downstream users. This passing flow criteria would have severe implications on Kentucky American's ability to meet customer demands and KAW undertook an Aquatic Study to better define the impact of withdrawals at low flow levels. The Aquatic Study was conducted in 1990-1991 by Environmental Science and Engineering, Inc. (ESE) and examined the Kentucky River from lock and Dam 4 to Dam 10. ESE investigated the water quality, water quantity and biota of the Kentucky River to determine how withdrawals during periods of low river flow (less than 7Q10) would affect water quantity, quality, aquatic life, downstream users, and dischargers and recreational users. The Aquatic Study concluded that water quality under modeled design flows from 50 to 5000 cfs; (32.2 to 323 mgd) would not vary significantly, and would not have a negative impact on the aquatic life even during extended durations. In addition, the study supported that use of the water from the storage in river pools would not impact the aquatic life, if the river flushing flows greater than 800 cfs (517 mgd) occurred at a frequency greater than once every 30 days from June through October.

Following the study, the Kentucky DOW revised KAW's permit and modified the policies regarding other permits. KAW was required to confirm the modeled water quality during low flow events, and once that occurred the permit was further modified. Because of this effort, the Pool 3 permit did not require passing flow amounts since there are no other downstream users in the Kentucky River.

4.9.1 Safe Yield Analysis of Existing Supplies

Kentucky American retained Harza in March, 1992 to utilize computer modeling to accurately determine the safe yield specifically for Kentucky American's source of supply facilities, including Jacobson Reservoir.

The 1930 drought of record was selected for the determination of the safe yield. Under the most probable set of assumptions regarding regional demands, lock leakage, the effect of Buckhorn and Carr Fork Reservoirs, allowance for passing flow and several other factors, the safe yield of the Kentucky American supplies during the drought was calculated to be 35 mgd.

Harza found that the computed safe yield is sensitive to a number of factors including:

1. The selection of the drought and the estimation of stream flows for the drought;
2. The use of storage in the reservoirs: Pool 9, Jacobson Reservoir, Buckhorn Reservoir and Carr Fork Reservoir;
3. Minimum instream flow requirements for the Kentucky River;
4. Withdrawal and discharge of water by municipal, industrial, commercial and irrigation users; and
5. Estimates of leakage through the locks and dams.

Sensitivity studies were undertaken to evaluate the effect on the safe yield of variations in these factors. Variations in lock leakage were found to have a significant impact on Kentucky

American's safe yield. For instance, the leakage through and around Dam 9 was estimated at approximately 50 cfs, and this assumption was included in the base case analysis. If the assumed leakage is increased to 75 cfs, then Kentucky American's safe yield decreases to 26 mgd rather than 35 mgd. On the other hand, if the assumed leakage is 25 mgd, then the safe yield increases to 49 mgd. Although the Dam 9 was replaced, the karst topography in the area of the dam has been suspected of being a source of leakage around the dam for years. The sensitivity analysis highlights the critical importance of having the Kentucky River locks and dams repaired and maintained to sustain the pools. The replacement of Dam 9 in 2010 and Dam 3 in 2011 were critical in sustaining the safe yields, however, it must be noted that no additional storage was created so the safe yield available has not been increased.

4.9.2 Adequacy of Raw Water Supply

The water supply available to KAW is highly dependent upon the time of year and whether or not the region is under a drought condition. Under drought conditions, the available water to KAW also varies, depending upon the level of the river, as shown in **Table 4-1**.

Table 4-1
Kentucky River Station 1
KRS1 (Pool 9) Kentucky River Withdraw Permit

Parameter	River Flow Rate (1)	Allowable Withdraw
River Level	> 140 cfs	60 mgd
River Level	139.99 - 120.00 cfs	58 mgd
River Level	119.99 - 90.00 cfs	54 mgd
River Level	89.99 - 60.00 cfs	50 mgd
River Level	59.99 - 30.00 cfs	48 mgd
River Level	29.99 - 0.00 cfs	45 mgd
Drought Phase 2	N.A.	45 mgd
Drought Phase 3	N.A.	42 mgd
Drought Phase 4	N.A.	40 mgd
Drought Phase 5	N.A.	35 mgd
Drought Phase 6	N.A.	30 mgd

(1) - River flows are based on a USGS gauging station for flows entering into the KAWC pool at Dam 10

At this time, the projected raw water supply allocation available to Kentucky American ranges from an extreme drought low of 51.8 mgd to a winter high of 84.1 mgd, as shown in **Table 4-2** below.

Table 4-2 Allocations from Surface Water Supplies								
Source (1)	**Date Last Revised**	**Permit No.**	**Allocation (mgd)**	**Anticipated Withdrawal (mgd)**	**Passing Flow Reqm't**	**Summer (mgd)**	**Winter (mgd)**	**Drought (mgd) (2)**
Kentucky River at Pool 3	1/10/07	1572	6 - 20(3)	6 - 24	None	20	6	20
Kentucky River at Pool 9	9/17/99	200	45.0	45.0	Yes	45	63	45-30
Jacobson Reservoir	5/26/89	201	16.0	16.0	None	16	16	16
Lower Thomas Lake	8/29/06	0874	0.80 - 0.90(4)	0.80 - 0.90	None	0.80	0.9	0
Severn Creek	1/27/12	0863	1.1 - 1.2(5)	1.1 - 1.2	none	1.2	1.1	0
Current Total Supplies						82.8	87.1	65-50

(1) Not included in this table is Lake Ellerslie, which has poor water quality but is available as an emergency supply.
(2) Drought condition assumes Phase 6 worst case scenario occurring between June and August.
(3) Jan - May = 6mgd; June - Aug = 20 mgd; Sep - Dec = 6 mgd
(4) Jan - Apr = 0.80 mgd, May - Jun = 0.85 mgd; Jul - Aug = 0.090 mgd
(5) Jan - May = 1.1 mgd, Jun - Aug = 1.2 mgd

Comparison of supplies with demands presented in **Section 3** indicates that under most scenarios, supplies are adequate to meet current and projected future demands through the planning period. Only under severe droughts would limitations on allocation pose challenges in meeting the projected drought average day demands if elevated demands are left unabated. KYAW has in place a comprehensive drought management plan that includes demand management measures aimed at mitigating such circumstances and reducing demands to effectively fall within allocation limits. This demand management approach in conjunction KYAW's flexibility in supply/treatment options is considered adequate and cost-effective in meeting current and projected demands for the service area. Therefore, no recommendations for additional supply or treatment capacity are needed at this time.

It is recommended that Kentucky American continue to work closely to support the efforts of the Kentucky River Authority to replace the older dams on the river, and maintain the new structures. The Kentucky River Authority also maintains flow gauges and is responsible for transferring water from upstream pools during extremely low flow conditions.

SECTION 5
PRODUCTION

5.1 GENERAL

This section describes the existing production facilities and needed improvements at Kentucky-American Water. KAW's production facilities consist of three primary treatment plants and one recently acquired plant - the Kentucky River Station 1 (KRS-1), the Richmond Road Station (RRS), Kentucky River Station 2 (KRS-2) and the recently acquired Owenton facility. The Commonwealth of Kentucky recognizes the “rated” capacities of the three primary plants as equal to the “reliable” capacity. Thus, the current total rated capacity of the KAWC production facilities is 85 mgd with KRS-1, RRS and KRS-2 rated at 40 mgd, 25 mgd, and 20 mgd, respectively. KRS-1 has been granted a temporary re-rating to 45 mgd for summer months. The source of supply for KRS-1 and RRS is Kentucky River Pool 9, located at the KRS-1 site. The source of supply for KRS-2 is Kentucky River Pool 3, located at the KRS-2 site. Based on the overall plant capacities and the demand projections presented in **Section 3**, there does not appear to be a production deficit through the projected demand evaluation period of 2030.

It should be noted that the Department of Water (DOW) does not strictly mandate a plant’s maximum permissible production, but rather limits the maximum filter loading rate (5.0 gpm/sf for KRS-1, KRS-2 and RRS) to which a plant may operate as long as water quality requirements are met. The DOW, however, does limit the source water allocation allowance as discussed in **Section 4**.

Table 5-1 shows the KRS-1 Kentucky River Withdrawal Permit. The State limits the withdrawal from Pool 9 (i.e.; KRS-1) to maximum 60.0 mgd during the months of November thru April and 63.0 mgd during the months May thru October at Pool 9. KAW can go over this amount by 15% if the river level is greater than 140 cfs over a 30 day running average period. DOW does not want KAW to exceed 60/63 mgd, but is providing leeway over a 30 day period before the permit is considered to be in violation or before a permit renewal is needed. Thus as long as the river is at a sufficient flow level, KAWC can withdraw a maximum of a 30-day average of 72.4 mgd from the Pool 9 intake. (63 x 1.15 =72.4). If the river level drops below 140 cfs for four consecutive days during the 30-day period, then the 15% allowance is no longer valid. This approach is consistent with all water withdrawal permits in Kentucky.

There are also limitations if the river is flowing at low levels, and the 60/63 mgd capacities may be reduced. A reducing factor is applied based on decreased river flows as follows:

Table 5-1
Kentucky River Station 1
KRS1 (Pool 9) Kentucky River Withdraw Permit

Parameter	River Flow Rate (1)	Allowable Withdraw
River Level	> 140 cfs	60 mgd
River Level	139.99 - 120.00 cfs	58 mgd
River Level	119.99 - 90.00 cfs	54 mgd
River Level	89.99 - 60.00 cfs	50 mgd
River Level	59.99 - 30.00 cfs	48 mgd
River Level	29.99 - 0.00 cfs	45 mgd
Drought Phase 2	N.A.	45 mgd
Drought Phase 3	N.A.	42 mgd
Drought Phase 4	N.A.	40 mgd
Drought Phase 5	N.A.	35 mgd
Drought Phase 6	N.A.	30 mgd

(1) - River flows are based on a USGS gauging station for flows entering into the KAWC pool at Dam 10.

In September 2010, KAW placed in service a new 20 mgd treatment plant at Pool 3 of the Kentucky River, referred to as KRS-2. This plant provides additional treatment capacity and does not have restricted withdrawals based on river flows, however has varying withdrawal rates depending on the season. The State limits the withdrawal from Pool 3 (i.e.; KRS-2) to maximum 6.0 mgd during the months of November through April and 20.0 mgd during the months May thru October at Pool 9. Therefore KAW anticipates being able to shift water production during low river flows at Pool 9 as needed.

Descriptions of the Existing Facilities are provided in sub-**section 5.2.1** for KRS-1, **5.2.2** for RRS and **5.2.3** for KRS-2. These descriptions include an evaluation of the adequacy of each process and associated plant facilities for both the current rated capacity of each plant and the higher production rates discussed above, including the reliability of each process. Where applicable, Ten State Standards and/or AWWA/ASCE design guidelines are referenced and compared to the current and proposed plant ratings. This investigation also encompasses a review of historical raw water quality conditions in order to confirm that more desirable conditions would occur during the period of the year from June 1 through September 30 (June 1

to August 31 for KRS-2 based on withdrawal limits) which is the time during which a plant production capacity increase is likely to be needed to meet peak demands.
Additionally, plant facilities will be evaluated in the context of the Stage 2 Disinfectants/Disinfection By-Product (D/DBP) Rule. Like the Stage I D/DBP Rule, the Stage 2 D/DBP Rule requires TOC reduction based on raw water TOC concentrations and alkalinity. Also, under the Stage 2 D/DBP Rule, the plants will be required to meet a TTHM and HAA5 MCL of 80 ppb and 60 ppb, respectively, based on a Locational Running Annual Average (LRAA).

Another regulation that could impact KAWC is the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) which sets requirements for Crypto. The first round of sampling under the LT2ESWTR was completed in 2008. Source water *Cryptosporidium* concentrations were below 0.075 oocysts/L in the KY River. Therefore, no additional treatment (removal or inactivation) was read at the KRS-1 or RRS treatment plants. The second round of source water monitoring begins April 2015, for a total of 24 months. If the results are greater than 0.075 oocysts/L, additional treatment or for Crypto plant modifications will be required.

5.2 EXISTING FACILITIES

5.2.1 Kentucky River Station

The Kentucky River Station was originally constructed in 1958 and is located approximately ten (10) miles southeast of the City of Lexington at the top of the Kentucky River bluff in Lock 9 Pool at river mile 167.45. In 1980, purification units No. 9 and 10 were installed, increasing the plant's rated reliable capacity to the current year-round 40 mgd and allowing it to reach a capacity of 45 mgd during the summer months. The last major plant upgrades occurred in the early/mid 1990’s, with the addition of a new chemical building. With the exception of chlorine and ammonia, all chemicals are stored under one roof. In 2007 all six of the raw water pumps were replaced. A second distribution electrical substation was also added to the plant in 2007 for the high service pumps.

The station is a conventional surface water treatment plant utilizing Aldrich purification units, which consist of upflow flocculation discharging to perimeter multi-media filters. It derives its total source of supply from the Kentucky River. Facilities to transfer raw water to the Richmond Road Station and/or Jacobson Reservoir are also located and operated at the Kentucky River Station.

5.2.1.1 Raw Water Pumping Facilities

Raw water is withdrawn from the Kentucky River by means of six (6) vertical turbine pumps, constant speed, intake pumps located approximately 400 feet below the river bluff. The raw water from the river enters a dual compartment intake suction well by first passing through a single bar rack and parallel traveling screens. This suction well was modified in 1992 with the addition of baffle walls and suction cages which offset adverse pump suction conditions caused by high approach velocities or low river levels. Each half of the suction well can be taken out of service for maintenance as needed. The intake pumps are in an enclosed lightweight structure which shields them from the elements. Access to the intake is via incline car, barge, helicopter, or stairs. The incline car was constructed over 40 years ago, and has reached the end of its expected life. The capacity of the existing incline car is limited to 1,500 lbs which restricts use to only personnel and small tools. Replacement or removal of pumps and motors is accomplished via barge and/or helicopter access at substantial expense.

The discharge piping for each intake pump is equipped with a common surge control valve and an ultrasonic flow meter housed in a below ground valve vault. The vault also houses relief valves that will respond to hydraulic transient events to maintain acceptable pressures in the event of a power failure. The raw water is lifted approximately 400 feet to the top of the river bluff through parallel 48-inch, 36-inch, and 20-inch mains. At this point, the raw water is delivered to both KRS-1 and to a transfer pump station which supplies river water to RRS. The transfer pump station is the only means of getting river water to the RRS as the raw water pumps do not have sufficient head capability to reach the RRS alone. The flow of raw water into KRS-1 is controlled and measured by a raw water venturi and rate of flow controller located at the top of the bluff at the Rapid Mix facility. Water that is diverted to RRS via the transfer pumps is delivered via a 30-inch main. A Site Plan (**Exhibit 5-1**) is included at the end of this section, as well as a process schematic (**Exhibit 5-2**) along with a general hydraulic pumping profile schematic for the three treatment facilities (**Exhibit 5-3**) and a residuals waste stream schematic (**Exhibit 5-4**).

The raw water pumps were originally designed to pump in series to a high lift, secondary, station formerly located adjacent to the raw water pumps at the foot of the bluff. The secondary station was removed in 1992 and the raw water pumps were upgraded to a higher head. The replacement raw water pumps were designed to pump over the bluff, where the raw water either flows to the transfer station (where it is then pumped to either the Jacobson reservoir or RRS), or to the KRS-1 Rapid Mix. Shortly after the intake pumps were installed in 1992, the pump capacities began to drop off significantly. Further investigation revealed that a harmonics problem was causing excessive vibration and pump wear. Modification to the pumps and the structure which houses them were made to correct the harmonics problem. This slowed but did not eliminate premature wear of the pumps. KAWC and the pump manufacturer at the time identified the cause for pumps' continued wear to be attributed to grit and abrasive fines in the river.

In 2007, the pumps were replaced again with six stage, 10,000 gpm, 1,250 hp vertical pumps designed at 410' TDH. A summary of the pumps is shown in **Table 5-2**.

Table 5-2
Kentucky River Station 1
Raw Water Intake Pumping (constant speed)

Raw Water Pump	Driver	Location	Individual Rated Field Capacity (mgd)	Actual Peak Capacity (mgd)*	Motor Size (HP)
Intake Pump No. 1	Electric	Kentucky River	15	15	1,250
Intake Pump No. 2	Electric	Kentucky River	15	15	1,250
Intake Pump No. 3	Electric	Kentucky River	15	15	1,250
Intake Pump No. 4	Electric	Kentucky River	15	15	1,250
Intake Pump No. 5	Electric	Kentucky River	15	15	1,250
Intake Pump No. 6	Electric	Kentucky River	15	15	1,250
			Total Actual Capacity	90	
			Reliable Capacity	75	

* Based on discussions with the plant operator, three pumps will pump up to 45 mgd, indicating a shallow system head curve and optimal use of each of the pumps when operating in parallel. Actual parallel flow capacity would require a field test, however based on discussions with field personnel. It appears that an estimated reliable capacity of 75 mgd is likely close to the actual value.

The raw water pumps have adequate reliable capacity, however continue to experience performance issues. Because the pumps are only four years old, KAW believes the current issues are maintenance related, and KAW is currently addressing the issue by meeting and consulting with the pump vendor. Based on the current actual capacities, total and reliable raw water pumping capacity exists to supply both KRS-1 and RRS.

5.2.1.2 Transfer Pumps

There are two transfer pumps that receive water from the raw water pumps. Raw water flow is either fully or partially diverted from KRS-1 to RRS at the top of the bluff. Each transfer pump is rated at 18.1 mgd, according to operations staff, with a combined capacity equal to approximately 24 mgd. The current operational strategy is for one transfer pump to match one RW pump at RRS, and only one transfer pump is run at a time and is typically throttled. According to KAW plant personnel, the RRS typically only requires 17 – 18 mgd, essentially rendering the second pump as standby. Under the rare condition where the RRS demand has exceeded 18 mgd in recent years (under this condition, 20 mgd has been the maximum demand on the transfer pumps), RRS will draw water from the Jacobson reservoir. With KRS-2 now on line, there is additional redundancy for RRS.

Pumping limitations and issues are discussed in more detail from an energy efficiency perspective in **Section 7**.

5.2.1.3 Raw Water Quality and Chemical Pretreatment

The plant's entire source of supply is derived from the Kentucky River. The quality of the raw water entering KRS-1 is generally good, however TOC concentrations are relatively high. **Table 5-3** lists raw water characteristics at KRS-1 for the 5-year period ending December 2010. In addition to the overall daily values for the entire year, the table also includes a breakdown of raw water quality conditions for the periods of each year between June 1 and September 30 as well as October 1 and May 30. This information is provided to confirm that more desirable conditions would occur during the June 1 to September 30 period during which a plant production capacity increase will likely be needed to meet peak demands.

Table 5-3 Kentucky River Station 1 Raw Water Quality (January 2006 - December 2010)										
Raw Water Parameter	**Units**	**Daily Range**			**Jun 1 to Sep 30 only**			**Oct 1 to May 30 only**		
		Max	**Avg**	**Min**	**Max**	**Avg**	**Min**	**Max**	**Avg**	**Min**
Pumpage (mgd)	mgd	46.4	28.7	14.1	46.4	34.3	19.6	45.3	25.9	14.1
Turbidity	NTU	1,053.0	35.2	1.0	323.0	16.7	2.0	1,053.0	44.24	1.0
pH	Units	8.2	7.7	7.2	8.2	7.7	7.3	8.1	7.7	7.2
Total Alkalinity	mg/L	160	78.8	15	150	89.6	15	160	73.5	25
TOC (1)	mg/L	7.04	2.59	1.19	6.02	2.91	1.84	7.04	2.43	1.19
SUVA (1)	L/mg-m	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Iron	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Manganese	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Total Hardness	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Calcium Hardness	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Total Dissolved Solids	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Fluoride	mg/L	0.38	0.13	0.01	0.38	0.14	0.08	0.26	0.13	0.01
Sodium	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Bromide	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Chloride	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Color	units	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Odor	TON	4	1	1	2	1	1	4	1	1
Fecal Coliform	/100 mL	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Cryptosporidium(2)	Oocysts/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a

NOTE: n/a means data was not available

(1) TOC data is collected twice per month.

(2) Based on the results between July 1997-December 1998, the *Cryptosporidium* concentrations were less than 0.075 oocysts/L requiring no additional treatment (removal or inactivation) at the RRS treatment plant. Additional sampling conducted in 2006 were below the trigger of 0.075 oocysts/L, exempting KAW from a Bin 2 Classification. However, it should be noted that the need for additional *Cryptosporidium* treatment will be based on future sampling results.

All pretreatment chemicals at KRS-1 are fed within the two stage mechanical rapid mix tank located at the top of the bluff. These chemicals include chlorine for pre-disinfection, caustic soda for pre-pH control, polyaluminum chloride (PACl) and FeCl as the primary coagulants, cationic polymer as a secondary coagulant, and a non-ionic polymer to improve floc formation and settling. Ferric chloride is only used during high natural organic events in the raw water. Ferric chloride provides better organics removal, however PACl provides better turbidity removal, generates less residual sludge, and is less expensive than ferric. Ferric chloride also lowers the pH of the water, that of caustic. Addition is needed to increase pH to prevent corrosion in the distribution system.

There is a cationic polymer chemical feed line extending down the bluff to the bar rack for zebra mussel control, however that is not currently being utilized and is thought to be broken in several places. The line was installed 15 to 20 years ago. (There are currently no mussels at KRS-1, but they have been identified at KRS-2).

The plant also has a bag feed facility to feed powdered activated carbon (PAC) for the adsorption of organic contaminants in the event of a chemical spill in the river. Plant personnel have indicated that, while the need for PAC is infrequent, (less than once per year), it is an excessively labor intensive process. The process consists of lifting 50-lb bags. **Table 5-4** shows the pretreatment chemical usages experienced at KRS-1 for the 5-year period ending December 2010.

<u>Disinfection</u> - Chlorine gas (from one ton containers and evaporators) is currently fed in pretreatment. Chlorine is also fed as intermediate treatment at the filters around the ring which is submerged beneath the internal perimeter of the hydrotreaters. There is no post chlorine at clearwell because there is enough residual from the intermediate chlorine to bind with ammonia for chloramines.

Although potassium permanganate is stored on site in dry form, it is not currently being used. Dry potassium permanganate is not used. It is stored in case there is a spill or an emergency. If it were to be used, liquid form is preferred and it should be fed at the raw water pumps for more contact time. KAW would need another feed line for this, as well as a new bulk storage tank, rather than 55 gal drums.

Plant personnel noted that because ortho phosphate is fed directly to the clearwell, there is a tendency for the pumps to get coated, compromising pump efficiency and requiring additional labor to pull the pumps periodically for cleaning.

Table 5-4 Kentucky River Station 1 Pretreatment Chemical Usage (January 2006 - December 2010)					
Pretreatment Chemical	**Form [(1)]**	**Min Dosage (mg/L)**	**Avg Dosage (mg/L)**	**Max Dosage (mg/L)**	**No. of Days Used**
Cationic Polymer (Coagulant Aid) "CedarFloc 524"	Product	0.35	2.45	9.70	1,814
Non-ionic Polymer (Filter) "CedarFloc 550"	Product	0.11	0.28	1.00	1,452
Potassium Permanganate	Dry	0.28	0.79	1.98	96
Chlorine [(2)]	Gas	1.14	6.49	11.17	1,815
Caustic Soda [(3)]	Dry	0.13	11.82	46.93	745
Quick Lime	Dry	-	-	-	0
Polyaluminum Chloride "DelPac2020"	Dry	0.44	28.25	182.21	1,716
Ferric Chloride[(4)]	Dry	0.19	47.11	154.44	405
Powdered Activated Carbon	Dry	-	-	-	0

(1) The form of the chemical indicated in the table is not necessarily the form in which the chemical is fed; rather, it is the form in which the dosage numbers are based.
(2) Chlorine is also fed at the filters.
(3) Caustic is fed in the clearwell.
(4) Ferric chloride is used only during high organic events.
(5) Trade names may vary from year to year.

All of the plant's pretreatment chemicals, with the exception of chlorine, are housed in a Chemical Building which was constructed in 1996. The chlorine storage and feed facilities, which were upgraded in 1999 to comply with industry and internal safety standards, are located in the plant's main control building (i.e.; the high service pump building). These facilities include a dry scrubber to neutralize a single one-ton chlorine gas container in the event of an accidental leak. KAW started utilizing ferric again as part of the treatment process, and the demand for ferric compromises adequate PACl storage.

Two tables are presented below which compare the actual size or capacity of chemical storage and feed components with that which would be required when sizing these components using actual chemical dosages experienced over the five year period from 2006-2010. **Table 5-5** compares actual storage volumes to a range of calculated volumes based on [max day/avg dose] and [avg day/max dose] using 52.4 mgd and 28.7 mgd (28.7 mgd is the "average of the average” production rate over the five year period) as the peak (maximum plant capacity) and average day (average production between 2006 and 2010) production rates, respectively. **Table 5-6** compares total and reliable (largest feed unit out of service) feed equipment capabilities to actual feed rates, and also to a worst case feed rate based on a 52.4 mgd production (peak) rate and the maximum historic dosage experienced between 2006-2010. Neither table includes quick lime since it was not applied during the five-year period from 2006-2010. **Table 5-5** does not include powdered activated carbon since this chemical is only for emergency purposes and would be brought on site as needed.

Table 5-5
Kentucky River Station 1
Pretreatment Chemical Storage

Pretreatment Chemical	Type of Storage	Actual Bulk Storage	Bulk Storage Req'd for 31 days		Actual Day Storage	Day Storage Required
			Avg Day/ Max Dose	Max Day/ Avg Dose		
Cationic Polymer	Bulk tank	8,500 gal	7,846 gal	3,618 gal	100 gal	316 gal
Potassium Permanganate	55 lb. drum	6 drums	285 drums	195 drums	N/A	N/A
Non-ionic Polymer (Coagulant Aid)	55 gal. drum	8 drums (440 gal)	862 gal	413 gal	N/A	N/A
Chlorine [(1)]	Ton cylinder	14 cylinders	22 cylinders	22[(1)] cylinders	N/A	N/A
Caustic Soda[(1)]	Bulk tank	12,000 gal	12,468 gal	9,517 gal	300 gal	1093 gal
Polyaluminum Chloride	Bulk tank	Each @ 16,450 gal Total: 32,900 gal	141,000 gal	37,614 gal	2 -1,000 gal	5,358 gal
Ferric Chloride	Bulk tank	16,450 gal	120,000 gal	89,608 gal	1,450 gal	4,015 gal

(1) Post September 11, 2001 safety requirements include 15 days storage for chlorine.

When analyzing **Table 5-5**, it should be noted that not all of the chemicals are utilized daily which would reduce the amount of storage required over a 31-day period. For example, caustic soda was only fed on 745 days during the 5-year period from 2006-2010. Refer to the last column in **Table 5-4** to see the frequency of chemical usage.

The existing bulk storage for PACl and ferric are of concern. KAWC began scaling back on ferric chloride use and using more PACl at KRS-1 because PACl is significantly less expensive, results in less sludge generation, is equivalent at removing turbidity, and is only slightly less effective at removing TOC than ferric. As such the ferric bulk storage and feed system is the same as the PACl system, but ferric is used significantly less than PACl. Examination of the bulk storage volume requirements in **Table 5-5** reveals that PACl storage cannot provide 31 days of storage for either an average plant flow at a maximum dosage, where only three (3) days are provided, or a maximum plant flow at an average dosage where sixteen (16) days are provided. With respect to the maximum dosage (182 mg/L) at average flow guideline, it should be noted that such a high dose is very infrequent. Over the five-year period from 2006-2010, the PACl dosage exceeded 100 mg/L on only twenty two (22) days, and was less than 50 mg/L for more than 85% of the time. Assuming a peak flow of 52.4 mgd and an average flow of 28.7 mgd, and a dose of 50 mg/L, the current bulk storage would provide approximately 7 and 13 days of storage, respectively. Over the five year period from 2006 to 2010, the max flow was 44 mgd. Using this as the max flow, instead of 52.4, and using a dose of 50 mg/l, a similar analysis indicates the current bulk storage will provide 9 and 12 days of storage for max and average flows.

Utilizing historical data as discussed above, rather than the max dose, bulk PACl storage is still at risk even though PACl delivery in the vicinity of the plant is reliable. Capital improvements are recommended to increase PACl storage, or evaluate an alternate coagulant.

The existing chlorine storage room can only accommodate fourteen (14) cylinders (four on the scale and ten on the floor), which is less than the forty-four (44) cylinders required for 31 days of storage. As a result, it is necessary to receive more frequent shipments, posing a vulnerability to KAW if that reliability were compromised. Receiving frequent shipments has not been a concern since the chorine vendor has been historically reliable and improvements are not recommended at this time. See RRS for sentence on chlorine storage.

Table 5-6 Kentucky River Station 1 Pretreatment Chemical Feed				
Pretreatment Chemical	**Feed Units**	**Total Feed Capacity**	**Reliable Feed Capacity**	**Actual Feed Range(1) (min – max)**
Cationic Polymer	gal/hr	12.0	6.0	0.1 – 10.3
Potassium Permanganate (3)	ft^3/hr	4.7	0.0	0 - 0.9 @ 0.48 ppm
Non-ionic Polymer (Coagulant Aid)	gal/hr	1.0	0.0	0.1 – 0.4
Chlorine (feeders) (2) Chlorine (evaporators) (2)	lbs/day	12,000 16,000	9,000 8,000	1 – 2,895
Caustic Soda **	gal/hr	84.0	56.0	0.1 – 45.4
Polyaluminum Chloride	gal/hr	440	220	0.1 – 159.,2
PAC (3)	ft^3/hr	69.4	0.0	0.1 – 102.6
Ferric Chloride	Gal/hr	11 - 310	0	0.1 – 164.6

(1)Based on actual dosage minimums and maximums for the five-year period 2006-2010.
(2)Pre and post.
(3)Fed only as needed.

5.2.1.4 Rapid Mixing

Raw water from the intake pump station passes through the plant's influent venturi meter and flow control valve and is chemically dosed at the two stage rapid mix tank where coagulation occurs. The Rapid Mix tanks are 11' x 11' x 15' deep. **Table 5-7** presented below evaluates the process adequacy of the rapid mixing processes at KRS-1. The primary focus of the evaluation is to determine if any design parameters might be compromised at a 52.4 mgd production rate by comparing AWWA/ASCE design guidelines and Ten State Standards with the operation of the plant at current and maximum production rates of 40 mgd (rated plant capacity), 50 mgd (rated filter capacity of 5 gpm/sf), and 52.4 mgd (peak plant capacity).

Table 5-7 Kentucky River Station 1 Rapid Mixing						
Production Rate(1)	**Mean Velocity Gradient, G**			**Detention Time, t**		
	Actual (sec^{-1})	**AWWA/ASCE** (sec^{-1})	**Ten State** (sec^{-1})	**Actual** (sec)	**AWWA/ASCE** (sec)	**Ten State** (sec)
40 mgd	953	600 - 1000	n/a	29	10 - 60	< 30
50 mgd	953			23		
52.4 mgd	953			22		

(1) Maximum Pumping from KRS-1 from 2006-2010 was 46.40 mgd in 2006

G values indicated above are at average water temperature (66 deg F). G values are not affected by plant production rates, but are shown here to confirm adequacy of the process. Detention times are adequate, and an increase in the plant production to 52.4 mgd will not compromise the rapid mixing process. No modifications are recommended.

5.2.1.5 Sedimentation and Filtration

Coagulated water leaving the rapid mix tank travels through a looped piping system to ten (10) Aldrich/Dorr Oliver purification units where sedimentation and filtration occur. Five (5) valve houses serve pairs of purification units and house instrumentation and sampling points in addition to the control valves. Each unit is 69'-8" in diameter, 17'-9" deep (overall outside dimensions) with a 60'-0" diameter, 16'-3" deep center sedimentation compartment. Coagulated water enters through the bottom center column of each tank where it is dispersed into the main settling chamber for sedimentation. Sludge removal occurs in the sludge center ring via rakes along the slope of the floor of the tank.

In each purification unit, settled water overflows a circular weir into the rapid sand filter compartment of each unit. All of the units have adjustable V-notch overflow weirs. Each filter compartment has a surface area of 728 square feet (sf) and a mixed filter media gradation consisting of 11-inches of assorted size silica gravel, 6 1/2-inches of 0.35 mm garnet, 7-inches of sand, and 17 1/2-inches of anthracite. Two sets of spray rings are installed on the periphery of the filtration compartment for feeding a filter aid. The spray rings are also used for feeding intermediate chlorine to allow for better formation of chloramines. Each unit is equipped with an outlet rate of flow control valve which modulates according to the filter water level. Both turbidity and particle counts are monitored continuously for each filter unit. In June 2002, SCADA monitoring, alarm, and control was implemented to immediately shutdown any filter exceeding 1.0 NTU. This function has been successful in preventing KRS-1 from exceeding the filter composite 1.0 NTU limit. Filter to waste capability is available on each filter via gravity drain to the backwash water holding tank, however, the filter-to-waste step is not currently employed.

The Aldrich units formerly maintained a sludge blanket, however, automated sludge blanket maintenance had been difficult due to an inability to effectively control the velocity range through the sedimentation compartment. That is, constantly varying flow rates made stabilization of the sludge blanket difficult. Ultimately, sludge was spilling into the filters and causing filter upsets, so the sludge blanket process was discontinued and replaced with the current upflow sedimentation process.

Tables 5-8 and **5-9** presented below evaluate the process adequacy of the sedimentation and filtration processes at KRS-1. Over the five year period from 2006-2010, the settled water turbidity averaged 4.9 NTU. The TOC removal during that time complied with the required TOC removal requirements as mandated in the Stage I D/DBP and future Stage 2 D/DBP. The filter effluent turbidities over the five year period from 2006-2010 have averaged 0.03 NTU with a maximum value of 0.15 NTU which is well below the Interim Enhanced Surface Water Treatment Rule (IESWTR) MCL of 0.3 NTU in 95% of samples and the not-to-exceed MCL of 1.0 NTU, respectively. Under IESWTR a filter assessment would be required for any individual filter that exceeds 1.0 NTU and a Comprehensive Performance Evaluation by the State would be required if turbidity levels for an individual filter exceed 2.0 NTU for two consecutive 15 minute intervals. The primary focus of the evaluation in these tables is to determine if any design parameters might be compromised at a 52.4 mgd production rate by comparing AWWA/ASCE design guidelines and Ten State Standards with the operation of the plant at the current and maximum production rates (40, 50 and 52.4 mgd).

Table 5-8
Kentucky River Station 1
Sedimentation
Sedimentation Evaluation with all ten (10) sed basins operating

Production Rate[1]	**Flow Through Velocity, V**			**Detention Time, t**		
	Actual (ft/min)	**AWWA/ASCE** (ft/min[1])	**Ten State** (ft/min[1])	**Actual** (hr)	**AWWA/ASCE** (hr)	**Ten State** (hr)
40 mgd	N/A[2]	N/A	N/A	2.06	> 0.5	4.0
50 mgd	N/A			1.65		
52.4 mgd	N/A			1.57		
Production Rate	**Surface Loading Rate** [3]			**Weir Overflow Rate**		
	Actual (gpm/sf)	**AWWA/ASCE** (gpm/sf)	**Ten State** (gpm/sf)	**Actual** (gpm/ft)	**AWWA/ASCE** (gpm/ft)	**Ten State** (gpm/ft)
40 mgd	0.98	0.5	--	14.7	20.0	10.0
50 mgd	1.23			18.4		
52.4 mgd	1.29			19.3		

(1) Maximum Pumping from KRS-1 from 2006-2010 was 46.40 mgd in 2006.
(2) N/A for upflow clarifiers.
(3) Surface Loading Rate is for straight sedimentation, which is currently the mode of operation in the Aldrich units due to the absence of a sludge blanket.

The surface loading rate recommendation for a typical clarifier, with no sludge blanket, is recommended not to exceed 0.5 gpm/sf, while 1 gpm/sf is a typical design for a solids contact (sludge blanket) clarifier. In many cases at KRS-1, this value is exceeded, and it is possible that high settled water turbidities are a consequence of the high surface loading rate. It is recommended that KAW evaluate alternative(s) to decrease settled water turbidities at high flow rates. Modification recommendations are discussed in the residuals section.

Table 5-9
Kentucky River Station 1
Filtration

Production Rate[(1)]	**Surface Loading Rate**			**Surface Loading Rate (1 filter out)**		
	Actual (gpm/sf)	**AWWA/ASCE** (gpm/sf)	**Ten State** (gpm/sf)	**Actual** (gpm/sf)	**AWWA/ASCE** (gpm/sf)	**Ten State** (gpm/sf)
40 mgd	3.81	2.0 - 7.0	n/a	4.24	2.0 - 7.0	n/a
50 mgd	4.77			5.30		
52.4 mgd	5.00 [2]			5.55		
47.2 mgd	4.50			5.00		

(1) Maximum Pumping from KRS-1 from 2006-2010 was 46.40 mgd in 2006
(2) DOW allows up to 5 gpm/sf as long as water quality is compliant.

The DOW has indicated that filtration rates of 5.0 gpm/sf should not be exceeded at any time. At a plant production rate above 47.2 mgd, this limitation would be exceeded when one (1) filter is out of service for backwashing. Thus the plant operators will need to ensure that the plant influent rate of flow does not exceed 47.2 mgd with one filter out.

Washwater Supply Tanks - Two (2) wash water supply tanks provide the necessary head and volume for adequate backwashing of the filters. The total volume of water used during a backwash is approximately 70,000 gallons. Wash Water Tank No. 1 is 28 feet in diameter, 45 feet high with a total volume of approximately 207,000 gallons. Wash Water Tank No. 2 is 36 feet in diameter, 45 feet high, with an approximate volume of 343,000 gallons. The tanks are refilled by two (2) pumping units located in the main control building and taking suction from clearwell storage. Each of the refill pumps has a rated capacity of 1,000 gpm. All of the filters have surface wash facilities, however surface wash is not currently employed. The wash water supply facilities are adequate to perform multiple backwashes of each filter over a 24-hour period. Such wash capability will not be exceeded at a production rate of 52.4 mgd and, therefore, no modifications are recommended.

5.2.1.6 Clearwell and Finished Water Pumping Facilities

Clearwells – Filtered water from the purification units is directed to two (2) concrete clearwells, operated in series, both of which are located beneath the main control building. Clearwell No. 1 has a total capacity of 485,000 gallons and has a baffled inlet and outlet. Clearwell No. 2 has a total capacity of 490,000 gallons and has only outlet baffling. The two clearwells are interconnected by several sluice gates. Neither clearwell has interior baffling, and neither is currently utilized for disinfection credit as ammonia is fed prior to both clearwells.

In addition to the two below ground clearwells, an unbaffled, above ground, steel clearwell storage tank also exists at the plant. The tank is 110 feet in diameter, 29 feet to overflow, with a total capacity of 2,000,000 gallons bringing the total clearwell storage capacity to approximately 3,000,000 gallons. Finished water is transferred to the above-ground clearwell from Clearwell No. 1 through a 24-inch main by means of a 5 mgd vertical turbine pump. The above ground clearwell tank is not designed to operate in series with the other two clearwells. The stored water from the above ground tank flows back into the concrete clearwells through a rate-of-flow controller as additional finished water is needed to meet system demands. Operators have the ability to set the rate-of-flow controller to a desired supply via the SCADA system. The primary use of the above-ground clearwell is to equalize production flows so that the plant doesn't need to be brought on line if RRS has peak demands, and chemical feed rates are not continually fluctuating. In typical operation the above-ground clearwell is turned over daily to equalize production flows.

In subsequent **Section 5.2.1.7, Table 5-15**, it can be seen that the disinfection credit achieved at the plant is marginal under high flow, cold water conditions and not achievable under worst case conditions.

Distribution Pumps - Finished water is delivered to the distribution system through two (2) 30-inch mains by means of six (6) pumps which take suction from the two concrete clearwells. Pumps No. 12 and 13 are split case horizontal pumps while the four remaining pumps are vertical turbine style. Each of the 30-inch mains is equipped with a venturi meter to indicate and record the flows generated from distribution pumps 10, 11, 12, and 13. A separate venturi meter monitors the flows from distribution pumps 14 and 15 which connect downstream of the other meters.

Although surge protection is provided for each pump by means of surge control ball valves on the discharge of each pump, these valves are not configured to provide fast controlled closure upon realization of a power failure. Transient events and main breaks have occurred in the past, particularly during pump testing operations and in power failure occasions. A study performed in 1999 recommended that eight (8) air/vacuum valves be installed on the 30-inch mains to alleviate this problem. However, additional investigation has revealed that the cause of the transient events during the pump testing was due to open fire hydrants in the distribution system. A summary of the distributive pumping capabilities at KRS-1 is provided in **Table 5-10**.

Table 5-10
Kentucky River Station 1
Distributive Pumping

Distributive Pump	Driver	Individual Rated Capacity (mgd)	Worst Case Peak Capacity (mgd)	Best Case Peak Capacity (mgd)	Driver Size (HP)
Distributive Unit No. 10	Electric	8.0 @ 380’	6.4 @ 426’	7.8 @ 389’	700
Distributive Unit No. 11	Electric	8.0 @ 381’	6.6 @ 426’	7.8 @ 389’	600
Distributive Unit No. 12*	Electric	8.5 @ 380’	5.8 @ 426’	7.6 @ 389’	700
Distributive Unit No. 13*	Electric	10.0 @ 380’	7.0 @ 426’	9.4 @ 389’	800
Distributive Unit No. 14	Electric	10.0 @ 380’	7.8 @ 426’	9.6 @ 389’	800
Distributive Unit No. 15	Electric Diesel	10.0 @ 380’	8.6 @ 426’	9.6 @ 389’	900 765
		Total Peak Capacity	42.2 @ 426’	51.8 @ 389’	
		Reliable Capacity	33.6 @ 426’	42.2 @ 389’	

*Split case horizontal pumps. The remaining pumps are VTP.

The worst case and best case peak capacities in the table above reflect the two extremes of the operating conditions that will exist in the distribution system on a single day. The target capacity capable of leaving the plant is 51.4 mgd which reflects a plant capacity of 52.4 mgd minus in-plant wash water needs of approximately 1.0 mgd. Based on computer modeling, the worst case scenario reflects a min hour system demand of 37.6 mgd when all the distribution storage tanks are filling and system head is high. The discharge pressure from the plant in this scenario would be 426 feet. The best case scenario reflects a peak hour system demand of 107 mgd with the Hume Road, Mercer Road, and Parker’s Mill pump stations operating. The discharge pressure from the plant in this scenario would be 389 feet. The respective flow rates were then determined from pump curves which were developed from field tests. It can be seen in the table above that the total capacity of the distributive pumping units at KRS-1 is marginally adequate to generate 51.2 mgd. However, from a reliability standpoint, the pumping capacity is inadequate. The reliable pumping capacity under the worst case scenario is only 33.6 mgd. The average of the best and worst case scenarios is 37.9 mgd.

5.2.1.7 Chemical Post Treatment and Finished Water Quality

Post treatment chemicals include chlorine and ammonia for disinfection, caustic soda for pH adjustment (if needed), hydrofluosilicic acid for fluoridation, and a corrosion inhibitor for corrosion control. Chlorine is fed on top of the filters and all other post treatment chemicals are fed at a vault located before the clearwell inlet.

Chlorine residual is typically about 0.5 ppm in the water leaving the clarifier. Based on discussions with plant personnel, intermediate chlorine injection at the filters from rings in the hydrotreater increase the residual to approximately 3.3 to 3.8 ppm following filtration. Upon chloramine conversion, the chloramine concentration is 3 to 4 ppm leaving the plant. Ammonia is fed in the two filter effluent lines leaving the hydrotreaters.

Table 5-11 shows the post treatment chemical usages experienced at KRS-1 for the 5-year period ending December 2010. The chemical usage for chlorine and caustic soda were previously shown in the table for pretreatment chemical usage.

Table 5-11
Kentucky River Station 1
Post Treatment Chemical Usage (January 2000 - December 2002)

Post Treatment Chemical	Form *	Min Dosage (mg/L)	Avg Dosage (mg/L)	Max Dosage (mg/L)	No. of Days Used
Anhydrous Ammonia	Gas	0.41	1.28	2.00	1,719
Hydrofluosilicic Acid	Product	1.15	4.94	9.75	1,814
Corrosion Inhibitor	Product	0.72	3.56	18.33	1,815

* The form of the chemical indicated in the table is not necessarily the form in which the chemical is fed; rather, it is the form in which the dosage numbers are based.

The facilities for both the chlorine and ammonia feed systems are located in the Main Control Building. The caustic soda, hydrofluosilicic acid, and corrosion inhibitor storage and feed systems are located in the Chemical Building which was constructed in 1996. Two tables are presented below, similar to those which were presented for pretreatment chemicals, which compare the actual size or capacity of chemical storage and feed components with that which would be required when sizing these components using actual chemical dosages experienced over the study period. **Table 5-12** compares actual storage volumes to a range of calculated volumes based on [max day/avg dose] and [avg day/max dose] using 28.7 mgd and 52.4 mgd as the average day and peak production rates, respectively. **Table 5-13** compares total and reliable (largest feed unit out of service) feed equipment capabilities to actual feed rates, and also to a projected feed rate based on a 52.4 mgd production rate and the maximum dosage experienced during the period of the year from June 1 to September 30 when peak demands are most likely to occur.

Table 5-12
Kentucky River Station 1
Post Treatment Chemical Storage

Post Treatment Chemical	Type of Storage	Actual Bulk Storage	Bulk Req'd for 31 days		Actual Day Storage	Day Storage Required
			Avg. Day/ Max Dose	Max Day/ Avg. Dose		
Anhydrous Ammonia	(2) Bulk cylinders	13,130 gal	3,100 gal	3,468 gal	N/A	N/A
Hydrofluosilicic Acid	Bulk tank	8,500 gal	8,300 gal	6,600 gal	300 gal	227 gal
Corrosion Inhibitor	Bulk tank	6,400 gal	11,900 gal	3,961 gal	200 gal	450 gal

The analysis in **Table 5-12** indicates that the storage facilities at KRS-1 are adequate. However, the anhydrous ammonia tanks (two 6,565 gallon tanks) are located outdoors. An evaluation was conducted by AWWSC in December 1999 to evaluate the practicality of converting the anhydrous ammonia system at KRS-1 to a safer aqueous (aqua) ammonia system. The evaluation of both alternatives considered scrubber requirements, system and scrubber maintenance costs, capital and operating costs, implementation costs (structural…) and overall feasibility. A scrubber would need to adequately neutralize the volume of a 3,000 gallon tank. The tanks would also need to be enclosed in a building in order to contain the leak and allow for scrubbing. Additionally, KYAW would need to make arrangements for discharge of the waste from the scrubber, if needed. A more practical and safer solution would be to convert to an aqua ammonia storage and feed system. However, based on costs, it is not recommended to convert to aqua ammonia at this time. If major improvements are needed for the anhydrous ammonia storage and feed system, it would be recommended to convert to aqua ammonia at that time.

Table 5-13 Kentucky River Station 1 Post Treatment Chemical Feed				
Post Treatment Chemical	**Feed Units**	**Total Feed Capacity**	**Reliable Feed Capacity**	**Actual Feed Range (1) (min – max)**
Anhydrous Ammonia	lbs/day	1,495	1,020	2 - 456
Hydrofluosilicic Acid	gal/hr	22.0	11.0	0.1 – 13.2
Corrosion Inhibitor	gal/hr	40.0	16.0	0.1 – 14.8

(1) Based on actual dosage minimums and maximums for the five year period 2006 – 2010.

Generally, post-treatment chemical feed capabilities at KRS-1 are adequate.

Finished water quality from KRS-1 is good, and all current Federal and State regulations are met. KRS-1 has also met EPA's "Partnership for Safe Water" filtered water turbidity goals for 2012 while treating a flow range of 17.6 to 52.4 mgd. KRS-1 has been able to meet the EPA Partnership for 14 consecutive years. EPA Partnership compliance is voluntary and focuses on pathogen reduction. **Table 5-14** lists finished water characteristics at KRS-1 for the study period.

Table 5-14 Kentucky River Station 1 Finished Water Quality (January 2000 - December 2002)										
Finished Water Parameter	**Units**	**Daily Range**			**Jun 1 to Sep 30 only**			**Oct 1 to May 31 only**		
		Max	**Avg**	**Min**	**Max**	**Avg**	**Min**	**Max**	**Avg**	**Min**
Turbidity	NTU	0.15	0.03	0.01	0.13	0.04	0.01	0.15	0.03	0.02
pH	units	7.8	7.2	6.7	7.8	7.2	6.7	7.6	7.2	6.8
Total Alkalinity	mg/L	140	67.8	8	110	77.1	35	160	73.5	25
TOC	mg/L	3.05	1.72	0.72	3.05	2.02	1.51	2.75	1.57	0.72
Iron	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Manganese	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Total Hardness	mg/L	450	181.0	52	362	209.3	70	450	167.2	52
Calcium Hardness	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Total Dissolved Solids	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Chlorine Residual (total)	mg/L	4.5	3.6	2.7	4.4	3.6	2.7	4.5	3.6	2.7
Fluoride Residual	mg/L	1.41	1.09	0.38	1.41	1.07	0.38	1.34	1.09	0.82
Sodium	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Bromide	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Chloride	mg/L	80.0	24.6	3.5	80.0	28.2	3.5	69.0	22.9	4.5
Orthophosphate	mg/L	2.27	1.32	0.93	2.18	1.31	0.93	2.27	1.32	0.97
Zinc	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Aluminum	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Odor	TON	1.8	1	1	1	1	1	1.8	1	1
Ammonia	mg/L	0.46	0.08	0.01	0.27	0.05	0.01	0.46	0.09	0.01
Total Coliform	/100 mL	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a

NOTE: n/a means data was not available

Disinfection credit at KRS-1 is achieved in pretreatment only. Pre-chlorine is fed at the head of plant and intermediate chlorine is fed on top of the filters. Ammonia is fed prior to the clearwells and no practical disinfection credit is achieved beyond this point. The actual log inactivation is calculated daily based on a tracer study that was performed by Kentucky American personnel in 1996 as per Kentucky DOW guidelines. The tracer study resulted in a T10 value for the pre-chlorination (to the top of filters) and a T10 value for the intermediate chlorination (through the filters). T10 values were created at flows of 25 mgd or less, 26 to 39 mgd (at whole values), and 40 mgd or more (16 values for pre and 16 values for intermediate). The log inactivation is calculated by multiplying the T10 value (based on a various flows through the treatment plant) by the chlorine residual, and divided by the E-table factor that is obtained from CT tables for one log inactivation of *Giardia cysts* by Free Chlorine at various pHs and temperature ranges.

The Commonwealth of Kentucky requires a *Giardia* log inactivation of 1.0. **Table 5-15** below compares the adequacy of the actual disinfection to the required disinfection (in terms of log inactivation) over the five-year period from 2006-2010. The table shows the effectiveness of the plant in meeting disinfection requirements for summer (June 1 to Sept 31) and winter (Oct 1 to May 31) historic flows and the maximum production allowance granted by DOW of 52.4 mgd which is dictated by the filter loading limit of 5.0 gpm/sf.

Table 5-15
Kentucky River Station 1
Disinfection (January 2006 – December 2010)

Period	**Actual Log Inactivation @ Historic flows**			**Required Log Inactivation (DOW)**	**Log Inactivation at Maximum flow rate**			
Jun 1 – Sept 30	*Best Case*	*Typ. Case*	*Worse Case*		**Jun 1 – Sept 30**	*Best Case*	*Typ. Case*	*Worse Case*
Max Flow 46.40 mgd	10.96	2.05	**0.32**	1.0	Max Production 52.4 mgd[1]	10.96	2.05	**0.32**
Avg Flow 34.27 mgd	12.54	2.35	**0.36**					
Min Flow 19.61 mgd	15.14	2.83	**0.44**					
Oct 1 - May 31					**Oct 1 - May 31**			
Max Flow 45.26 mgd	7.96	1.30	**0.23**		Max Production 52.4 mgd[1]	7.96	1.30	**0.23**
Avg Flow 25.87 mgd	12.83	2.09	**0.36**					
Min Flow 14.14 mgd	15.05	2.46	**0.43**					

Worst Case: calculated using the following historic records: highest settled water pH, the lowest free chlorine residual for each treatment step, and the lowest temperature recorded for the season identified.
Best Case: calculated using the following historic records: the lowest settled water pH, the highest free chlorine residual for each treatment step, and the highest temperature recorded for the season identified.
Typ. Case: calculated using the following historic records: the average settled water pH, the average free chlorine residual for each treatment step, and the average temperature recorded for the season identified.
(1) Flows above 47.2 mgd consider all 10 filters on-line so a filter loading of 5.0 gpm/sf is not exceeded. All other flows are analyzed with one filter out of service.
Note: Based on the results between July 1997-December 1998, the *Cryptosporidium* concentrations were less than 0.075 oocysts/L requiring no additional treatment (removal or inactivation) at the KRS-1 treatment plant. Additional sampling 2006 were below the trigger of 0.075 oocysts/L, exempting KAW from a Bin 2 Classification. However, it should be noted that the need for *additional Cryptosporidium* treatment will be based on future sampling results.

It can be seen in the table that the disinfection requirements at KRS-1 are comfortably met under typical conditions. At all the historic flow capacities in **Table 5-15** during the worst conditions, the Actual Log Inactivation is less than the required 1.0 log inactivation. KAW should be advised that if operating at a flow greater than 40 mgd under the worse pH and temperature, a minimum chlorine residual greater than 1.55 mg/L should be maintained through both the pre and intermediate process. As the conditions get more typical (i.e. temperature increases and pH decreases) and/or the flow decreases, the minimum chlorine residual required will decrease. KAW should monitor both pre and intermediate chlorine to ensure one log inactivation is provided under all flows and all conditions while maintaining an adequate residual in the distribution system.

KAW has had discussions exploring the option of eliminating (or utilizing at a small feed rate) pre-chlorination, and rely on intermediate chlorine along with a chlorine injection point prior to entering the clearwell (which would need to be installed). In addition, the ammonia injection point would be relocated after the distribution pumps, prior to entering the distribution system. KAW staff has had conversation with the DOW on the definition of worst case scenarios for providing adequate log inactivation. Although the maximum pumping for KRS-1 is 52.4 mgd, KAW does not operate at that capacity during the winter months when the water is the coldest and requires more CT to achieve adequate log inactivation. Since KRS-2 has been online, KRS-1 has operated at a maximum capacity of 23.8 MGD from Oct 1 to May 31 (classified the winter months). The maximum capacity of 52.4 MGD would still be utilized from June 1 to September 30.

Assuming the T10 value for the intermediate chlorine can still be utilized, a baffling factor of 0.3, and half the total capacity for the clearwell (~500,000 gallons) per American Water Guidelines, the table below shows the maximum winter capacity (at a chlorine residual of 3.6 ppm, pH of 8.1, and temperature of 1.8 ^{0}C) and maximum summer capacity (at a chlorine residual of 3.6 ppm, pH of 8.2, and temperature of 18.6 ^{0}C). Although 23.8 MGD is the maximum historic winter capacity of KRS-1 since KRS-2 has been online, 25 MGD will be used in the winter calculations since the 1996 tracer study has a T10 value of 33 for flows of 25 MGD or less.

Winter Log Inactivation (flow of 25 MGD):

-Intermediate Injection Point:

Temperature (^oC)	T10	Chlorine Residual (ppm)	E-Table Factor	Log Inactivation
1.8	33	3.6	132	**0.9**

-Before Clearwell Injection Point:

Clearwell Capacity (gallons)	Baffling Factor	Contact Time (min)	Chlorine Residual (ppm)	CT Value	CT Required for 3-Log	Log Inactivation
500,000	0.3	8.64	3.6	31.1	329	**0.28**

Total Winter Log inactivation = 0.9 + 0.28 = **1.18 > 1.0 log required**

Summer Log Inactivation (flow of 52.4 MGD):

-Intermediate Injection Point:

Temperature (^oC)	T10	Chlorine Residual (ppm)	E-Table Factor	Log Inactivation
18.6	9.4	3.6	49	**0.69**

-Before Clearwell Injection Point:

Clearwell Capacity (gallons)	Baffling Factor	Contact Time (min)	Chlorine Residual (ppm)	CT Value	CT Required for 3-Log	Log Inactivation
500,000	0.3	4.12	3.6	14.8	118	**0.38**

Total Summer Log inactivation = 0.69 + 0.38 = **1.07 > 1.0 log required**

If the capacity during the winter time increases above 25 MGD, the minimum required log inactivation ratio of 1 will not be achieved. If baffles are added to the clearwell and a 0.7 baffling factor is approved by DOW, the winter capacity can increase to 30 MGD, achieving the log inactivation of 1. Without any improvements to the system and under the worse conditions, the maximum winter (October 1 – May 31) and maximum summer (June 1 – September 30) capacities at a chlorine residual of 3.6 ppm are 25 MGD and 52.4 MGD, respectively. It should be noted that if the chlorine residual falls below 3.6 ppm, the flow through the plant must decrease to achieve the required log inactivation.

Table 5-16 below shows the TOC removal ratio for 2006 to 2010.

Table 5-16
Kentucky River Station 1
Precursor Removal

Year	Finished Water Parameter		Qtr 1	Qtr 2	Qtr 3	Qtr 4	MCL
2006	TOC Removal Ratio 2006		1.41	1.39	1.37	1.21	> 1.0
2007	TOC Removal Ratio 2007		1.28	1.29	1.28	1.30	> 1.0
2008	TOC Removal Ratio 2008		1.18	1.15	1.15	1.18	> 1.0
2009	TOC Removal Ratio 2009		1.17	1.21	1.20	1.14	> 1.0
2010	TOC Removal Ratio 2010		1.13	1.15	1.09	1.27	> 1.0

Based on the new Stage 2 D/DBP Rule, the TTHM and HAA5 concentrations are based on Locational Rolling Annual Average (LRAA) instead of a Rolling Annual Average (RAA) per area served by a treatment plant. An internal distribution system evaluation (IDSE) model was conducted identifying sample sites based on population size. KAW is required to monitor at twelve (12) different locations (5 with the highest TTHM concentrations, 4 with the highest HAA5 concentrations, and 3 existing Stage 1 locations). Stage 2 monitoring began on April 1, 2012 for KAW. Exhibit 5-1 and 5-2 show the TTHM LRAA concentrations and the HAA5 LRAA concentrations (utilizing historic quarterly sampling) for the Stage 2 sites for KAW compared to the MCLs of 80 ppb for TTHM and 60 ppb for HAA5.

Exhibit 5-1
KAW Service Area TTHM LRAA at Stage 2 Sample Locations

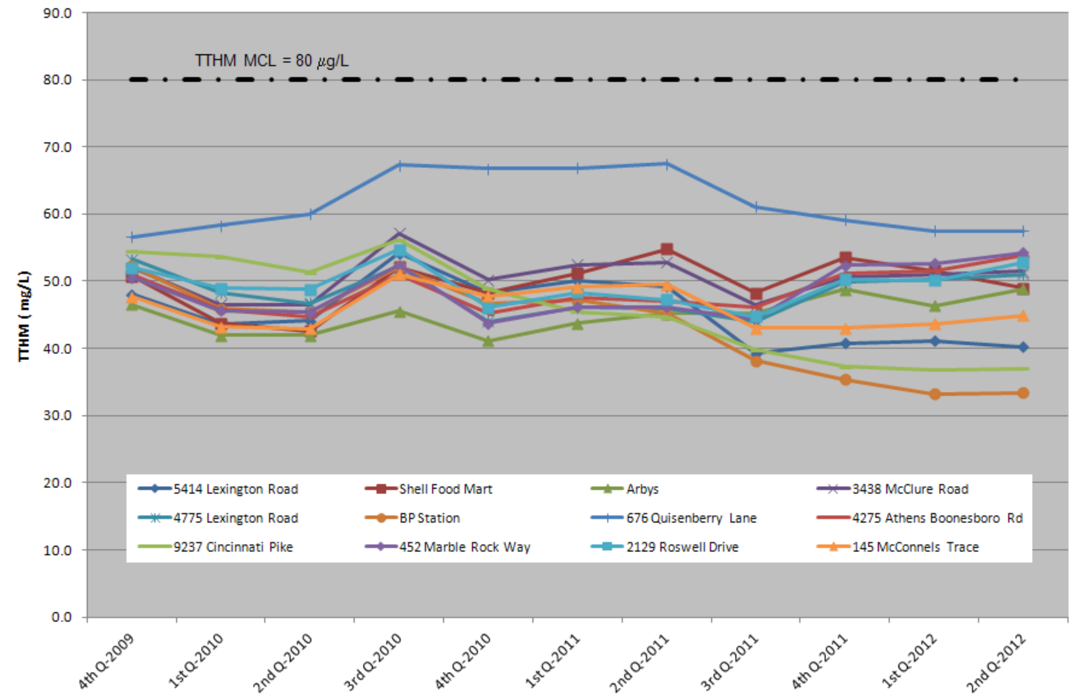
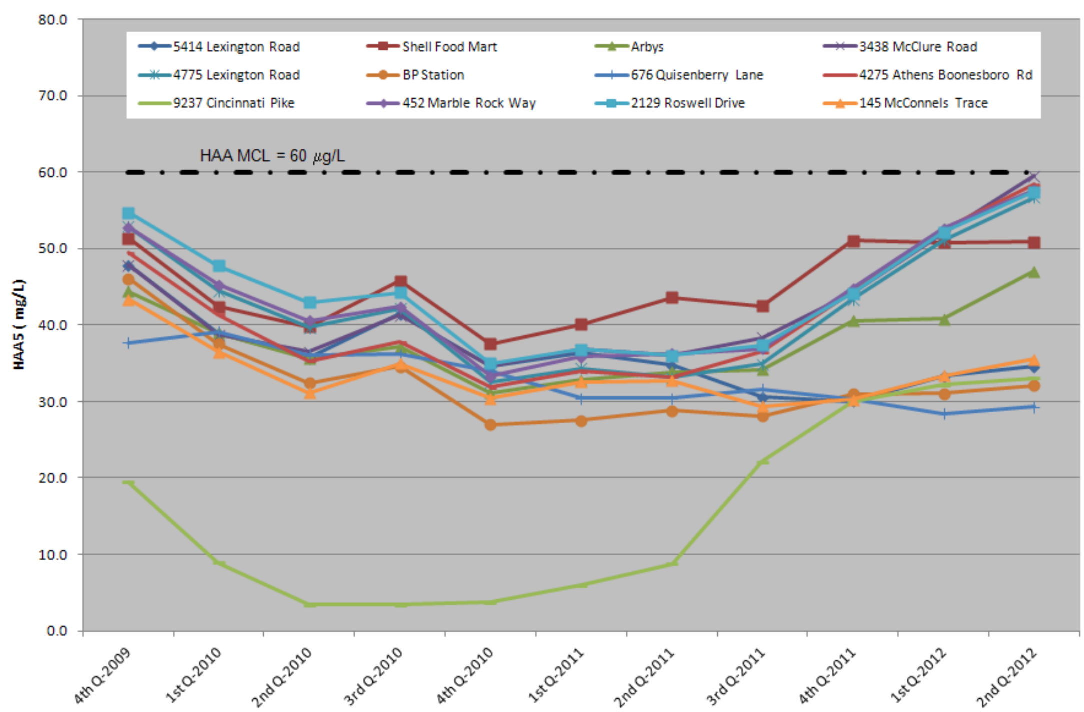


Exhibit 5-2
KAW Service Area HAA LRAA at Stage 2 Sample Locations



Based on the results shown in the Exhibits above, KAW meets the MCLs for TTHM and HAA5 under the Stage 2 D/DBP Rule. It should be noted that some of the historic quarterly results for the monitoring sites exceeded the MCL for TTHM and/or HAA. All the monitoring site's historic quarterly results exceeded the MCL for TTHM or HAA5 at least one quarter from 2008 to 2011 except Arbys and 4775 Lexington Road for TTHM and BP Station, 676 Quisenberry Lane, 9237 Cincinnati Pike, and 145 McConnells Trace for HAA5. It should be noted that no monitoring site exceeded the MCL for TTHM or HAA5 in the 2nd quarter of 2012, when regulated monitoring began for the twelve (12) sites. KAW is advised to utilize the historic monitoring results as a guide to ensure future compliance. No modifications to the current disinfection schemes at KRS-1, RRS, and KRS-2 are recommended.

KAW staff is also having difficulty removing TOC in the fourth quarter, as show in Table 5-16. At a certain threshold of removal, KAW must develop a memorandum to the KDOW explaining how the situation will be mitigated, and also meet with KDOW. KAW staff believes that residual and difficult organics in the river are part of the cause, as well as a need to re-visit the current coagulant.

5.2.1.8 Treatment Residuals Handling

Treatment residuals which collect in the sedimentation compartments of the purification units were formerly withdrawn by a gravity batch process to a common residuals well, 12' x 12' x 10' deep, containing two submersible residuals pumps, each rated for 500 gpm. The residuals well, however, is no longer utilized and residuals currently flow by gravity to one of four sludge lagoons for settling and decanting via a new, electric actuated 12-inch drain line (installed around 2006-2007). The practice of utilizing the 3-inch drain lines and sludge pit was discontinued because the valves in the sludge pit had to be manually opened/closed. Also, the sludge pumps were clogging when the sludge was too thick.

Sludge blowdown is conducted in the absence of sludge concentration monitoring, introducing the possibility of extending the blowdown period to allow clear supernatant to be discharged, increasing the loading rate to the lagoons. Based on discussions with plant personnel, the valve opening may vary from partially open to only cracked, and the blowdown duration also varies.

The four lagoons, numbered 1-4, have volumes of 1.4, 1.9, 1.6, and 0.5 MG respectively. Historically, they have been cleaned approximately once every two years and the residuals retained on site for possible future beneficial reuse applications. Lagoon No. 1 is normally kept empty in order to accept emergency overflow discharges from the below ground clearwells and supernatant pit. The lagoons can also accept accidental overflows or controlled discharges from the wash water storage tanks, the above ground clearwell, and laboratory sample water.

Each lagoon is equipped with a telescoping valve which allows for proper decanting to a common 12' x 14' x 8' deep lagoon supernatant collection vault. Two (2) vertical turbine pumps, each rated for 1,000 gpm, recycle this supernatant from the collection vault back to the Kentucky River. A sodium thiosulfate feed system is flow-paced to ensure adequate dechlorination of any supernatant directed to the river, however flow volume is not recorded. The collection vault includes an overflow into Lagoon No. 1 which would prevent an accidental discharge to a nearby creek should the recycle pumps fail. The inlet and outlet piping diameter to/from the supernatant tank was increased from recently to support higher flows. The vault size, however, was not increased. KAW operations have indicated that it is difficult to keep up with backwash cycles and decanting during periods of high flow and/or turbidity.

Tables 5-17 and **5-17A** below show the theoretical solids and sludge generation using data obtained from 2006-2010.

Table 5-17
Kentucky River Station 1
Theoretical Solids Generation (January 2006 - December 2010)

lbs/day	Solids Source	lbs/year	% of Total Solids Production
7,298	Raw(1)	2,663,588	70%
1,599	PACL	583,517	15%
1,261	Ferric Cl	460,268	12%
521	CAT	190,256	5%
-312.75	To River(2)	(114,154)	-3%
10,366	TOTAL	3,783,475	100%

1. Assumes a flow of 25 mgd and average raw water turbidity = 35 NTU.
2. Assumes 7.5% of daily flow is returned to river and TSS = 20 mg/l.

Table 5-17A Kentucky River Station 1 Theoretical Residuals Generation (January 2006 - December 2010)					
% Solids in Lagoon	Mass, lbs	lbs/cf	CF	gal	% of Lagoon Vol(1)
3%	126,115,845	64	1,970,560	14,739,789	259%
5%	75,669,507	65	1,164,146	8,707,814	153%
7.5%	50,446,338	67	752,930	5,631,920	99%
10%	37,834,754	68	556,393	4,161,823	73%
12%	31,528,961	70	450,414	3,369,095	59%

1. Assumes utilization of all four lagoons (5.7 mg).

Table 5-17 illustrates the estimated solids production, which includes contribution from each of the coagulants and actual raw water turbidity. The mass balance assumes 3% of the solids are released back to the river. **Table 5-17A** indicates that at 7.5% solids, the lagoons would be filled to capacity in one year – at a more reasonable assumption of 5%, the lagoons would be filled in nine months. The filling of the lagoons would accelerate at higher flow and would increase even more if additional flow were being added to the lagoons from the WWHT's and/or over-release during a sludge blowdown. The table also assumes that all four of the lagoons are being utilized.

Filter Run and Filter Wash - The calculation of Unit Filter Run Volume (UFRV) identifies the amount of water produced per unit filter area between backwashes. Historical data shows an average UFRV of 4,200 gallons at KRS-1. This is considered a low number with 7,500 to 10,000 gallons being considered excellent. It is likely that the elevated settled turbidities are responsible for the low UFRV at KRS-1. The Unit Backwash Volume (UBV) identifies the amount of washwater used per unit area. The UBV at KRS-1 is 89 gal/sf based on 64,000 gallons per wash. A typical range of UBV, without filter to waste, is 100 to 200 gallons per sf, so the KRS-1 value is slightly below the low end of the expected range. The overall recovery is the ratio of net to total water filtered [(UFRV – UBV)/UFRV)] and is calculated as 97.8 percent for KRS-1 with the identified UFRV and UBV. Should the UBV increase to 200 gallons per sf (140,000 gallons per wash), then the recovery would be 95%. Recovery values less than 95% are considered highly inefficient and should be avoided.

Wash water waste flows are normally directed to one of the two existing holding tanks. It can also be directed to either of the lagoons via a 12-inch drain line. Discharge to the lagoons is currently employed during periods of high turbidity when filters are being backwashed in rapid succession.

Wash Water Waste Holding Tank No. 1 is 70 feet in diameter with a center depth of 9 1/2 feet, holding a total volume of approximately 250,000 gallons. Wash Water Waste Holding Tank No. 2 is 70 feet in diameter with a center depth of 12 1/2 feet, holding a total volume of approximately 290,000 gallons. Tank No. 1 is equipped with two (2) vertical turbine pumps, each rated for 2,100 gpm, which recycle supernatant to the river, and two (2) vertical turbine residuals pumps for ultimate discharge to the lagoons. Tank No. 2 is equipped with three (3) similar supernatant pumps and two (2) similar residuals pumps. All of the residual system supernatant pumps discharge through a common recycle venturi meter back to the river, after dechlorination with sodium thiosulfate in compliance with a KYPDES permit.

Operations staff indicated that each wash water holding tank is able to contain 2 to 3 backwashes. Based on discussions with KRS-1 operations staff, an average wash volume is approximately 70,000 gallons per filter, indicating at least three backwash volumes.

In summary, intermittent high rate wastewater discharges from the Aldrich units and WWHTs challenge the ability of the lagoons to provide adequate settling of residual solids and also challenges the hydraulic capacity of the wastewater discharge system (ie; sludge lines, sludge pumps, supernatant pumps etc). The high waste flows are largely attributed to frequent and/or high volume sludge blow downs performed to limit settled water turbidities in the Aldrich units. Elevated settled water turbidities are due to a high hydraulic loading rate in the Aldrich units and lack of flocculation in the absence of a sludge blanket. As the lagoons fill with solids, their ability to provide settling is diminished and there is increased risk of excessive suspended solids in the return to the river.

5.2.1.9 Electrical

The existing plant electrical distribution system consists of a feed from a Kentucky Utilities step down transformer rated at 69kV - 4160V. Two incoming aerial feeds are terminated into a switchgear lineup located in the Incoming Switchgear Building which is adjacent to the Raw Water Transfer Station. This switchgear facility feeds plant Substations A, E, the Intake Electrical Building, the Raw Water Transfer Building, and the High Service Building which includes Substation B. Substation A primarily feeds the distributive pumping facilities while substation E feeds the rapid mix tank and incline car building. Substation B is located closest to and currently feeds the existing residual handling facilities. The feeds from the incoming switchgear building are overhead lines. This equipment is predominantly manufactured by

Westinghouse and has been installed at various times as the plant was expanded over the years. The equipment is aged and in need of replacement due to the unavailability of spare parts. The breakers inside the Incoming Switchgear Building that feed the various substations are rated for 1200A continuous.

Distributive Pump No. 15 is the only pump at the plant which is equipped with a direct drive diesel engine for standby power. Two emergency generators are also located at the plant. Emergency Generator No. 1, located in the distributive pump room, is rated for 90 KW and supplies power for lighting and electrical outlets in the main control building. Emergency Generator No. 2, located outside of the third lift pump building, is rated for 75 KW and supplies power for the incline car as well as lighting and electrical outlets at the intake. There is no standby power for the raw water intake pumps.

A Bristol Babcock distributed control system (DCS) was initially installed in 1992 to incorporate monitoring and control of the intake pumping facilities and was expanded several times since. Currently, the majority of the plant functions, with the exception of the distributive pumps, are monitored and controlled through the DCS across fiber optic cabling. Bristol 3330 RTUs and 3331 RIO racks are utilized throughout the plant in a master/slave configuration with redundant data concentrators. ACCOL measurement and control communication software is installed in each of the RTUs. The control room is equipped with redundant workstations running Genesis for DOS, a dedicated e-mail computer running Lotus Notes across the company's frame relay WAN, and a dedicated particle counter computer. A Motorola People Finder pager system is used by the plant operators when outside of the control room for notification of critical alarms. Laptop computers are also available for programming RTUs.

The current DCS is no longer supported by the software vendors, and limitations in its ability to handle the increased demands as the system was expanded have resulted in frequent system crashes and loss of data. In 2003 the DCS underwent workstation hardware and software upgrades to ensure system reliability and allow for increased production. Control logic, alarming logic, reporting capabilities, and system displays will be modified as necessary to ensure that the efficiency of the system is being maximized. The modified system includes improved remote access capabilities including handheld Pocket PCs on site, WAN from the RRS plant, and supervisory access via the Internet. The result has been an improvement in response time to alarm events and more efficient use of the system.

5.2.2 Richmond Road Station

The Richmond Road Station (RRS) was the site of the original plant that was established in 1885. The current plant was constructed in 1924 and is located in the southeastern portion of the City of Lexington, adjacent to both Lake Ellerslie and the Water Company office complex at 2300 Richmond Road. The plant has undergone expansions and improvements in 1937,1988, 1992, and 2003 to bring its current rated capacity to 25 mgd. However, the plant is permitted to treat 30 mgd by the DOW on a temporary basis as long water quality requirements are met. As noted earlier, the DOW does not strictly mandate a plant's maximum permissible production, but rather mandates the maximum filter loading rate (5.0 gpm/sf) to which a plant may operate as long as water quality requirements are met. The DOW focuses on the source water allocation allowance as the primary governor in a plant's operation. The reliable capacity of the RRS is 25 mgd since taking one filter would still limit the filter loading rates to less than 5 gpm/sf.

The station is a conventional surface water treatment plant with concrete settling basins and granular activated carbon gravity filters and derives the majority of its source of supply primarily from Pool 9 of the Kentucky River. The supply can be augmented with raw water from Jacobson Reservoir; however, the reliance on Jacobson Reservoir is minimized due to inferior water quality in the reservoir, particularly following rain events. Under emergency conditions, the station can also supplement its source of supply from Lake Ellerslie although this source has limited capacity and also exhibits poor water quality.

5.2.2.1 Raw Water Pumping Facilities

Pumping facilities are in place at each of the three sources which provide raw water to RRS (Kentucky River Pool 9, Jacobson Reservoir, and Lake Ellerslie). The total pumping rate from either the Kentucky River or Jacobson Reservoir is controlled by two venturi raw water rate-of-flow controllers with one dedicated to each of the inlets to the plant's two sedimentation basins. These rate-of-flow controllers were installed in 2003 and allow flow to be regulated to each basin (3 - 15 mgd). Although the controllers communicate with the SCADA system, they are not controlled through SCADA but are manually opened and closed by the operators.

The majority of the raw water withdrawn from the Kentucky River is used to supply KRS-1, however, some raw water is diverted to RRS by means of a transfer pumping station (hereinafter referred to as "KRS Transfer") which is located on the KRS-1 site. KRS Transfer houses two horizontal split case pumps, each with a rated capacity of 18.1 mgd at 270' TDH. However, with both pumps in operation, the flow from KRS Transfer is limited to 24 mgd due to hydraulic limitations. The transfer pumps direct raw water through a 30-inch main to either Jacobson Reservoir or directly to the raw water vault at RRS depending on the valving arrangement at Jacobson Reservoir.

Raw water can also be withdrawn from Jacobson Reservoir and pumped to RRS through three parallel mains by means of three horizontal pumps located in a station below the dam of the reservoir. There is no existing permit restricting withdrawals from Jacobson Reservoir. The suction line to the pumping units from Jacobson Reservoir to RRS tees off of the pipe that supplies water to Jacobson Reservoir from KRS Transfer. Therefore, if water is being transferred into the reservoir from the Kentucky River at a rate less than that which is being pumped from the reservoir, the Kentucky River water will by-pass the reservoir and be transferred directly to the suction of the reservoir pumping units in combination with Jacobson Reservoir water. The name plate capacities of the Jacobson Reservoir pumps are equal to 4, 4 and 12 mgd for a total capacity of 20 mgd. However, the installation of a 30-inch discharge pipe from the station has resulted in a reduction in discharge head so that the actual pumping capacities of the pumps are 6, 6, and 16 mgd, individually. When all three pumps are operating together, total capacity of the station is 24 mgd. The reliable name plate capacity is 8 mgd and the reliable field capacity is 12 mgd. The Jacobson Reservoir pumps are worn, which reduces pumping capacity and results in a significant loss of energy efficiency. In addition, at times, a portion of the discharge flow is allowed to recirculate back to the suction side of the pumps to provide adequate flow control to RRS.

The capability to withdraw raw water from Lake Ellerslie, adjacent to the RRS, also exists by means of two horizontal pumps located in the High Service Pump Building. Lake Ellerslie had at one time been the primary source of supply for the City of Lexington. These pumps, however, are only used in emergency conditions, such as drought conditions or during emergency loss of power at the other raw water pump locations, due to the limited safe yield of the lake. A summary of all the pumping facilities that provide raw water to RRS is provided in **Table 5-18.**

The actual peak capacities in **Table 5-18** are the capacities with all pumps running at all three locations. If only the pumps at the KRS Transfer Station and Jacobson Reservoir were in operation, which are the primary source of supply locations, the reliable raw water pumping capacities would be 42 mgd (24 + 18) with one pump out of service at KRS Transfer and 36 mgd (24 + 12) with one pump out of service at Jacobson Reservoir which is adequate for meeting maximum demand needs. It should be noted that the KRS Transfer station cannot deliver its maximum capacity when KRS-1 treats more than: 1) 46 mgd given that the Kentucky River intake pumps are withdrawing a maximum 70 mgd (all pumps operating); or 2) 33 mgd when the Kentucky River intake pumps withdraw 57 mgd (reliable operation) from the river.

Table 5-18
Richmond Road Station
Raw Water Pumping

Raw Water Pump	Driver	Location	Motor HP	Individual Field Rated Capacity (mgd)	Station Max/Reliable Capacity* (mgd)
Pump No. 8	Electric	KRS Transfer	1,000	18.0	24.0/18.0
Pump No. 9	Electric	KRS Transfer	1,000	18.0	
Low Service Unit No. 1	Electric	Jacobson Reservoir	100	6.0	24 / 12.0
Low Service Unit No. 2	Electric	Jacobson Reservoir	100	6.0	
Low Service Unit No. 3	Electric Diesel	Jacobson Reservoir	400 368	16.0	
Low Service Unit No. 4	Elec/Gen	Lake Ellerslie	40	4.0	10.0 / 10.0
Low Service Unit No. 5	Elec/Gen	Lake Ellerslie	60	6.0	
		Total Capacity		58.0	All 7 pumps
		Reliable Capacity*		52.0	No. 8 out
		Reliable Capacity*		46.0	No. 3 out

* - When the largest pump at each facility is out of service the remaining pumps operate near their individual rated capacities.

As discussed previously, the maximum flow treated at KRS-1 over the last five years was 46.4 mgd. Based on the reliable capacity of the KRS-1 intake pumps of 57 mgd, there would be 10.6 mgd available for transfer to RRS if KRS-1 was operating at the maximum flow. With the 24 mgd supply from Jacobson Reservoir with all pumps in operation, there would be sufficient raw water pumping capacity to meet the RRS plant capacity of 25 mgd. If the largest unit was out of service at Jacobson Reservoir as well, supply to RRS would be 22.6 mgd (10.6 mgd from KRS Transfer and 12 mgd from Jacobson Reservoir). Additional supply could be obtained from Lake Ellerslie, if needed, under these emergency conditions.

5.2.2.2 Raw Water Quality and Chemical Pretreatment

Approximately 70% of the plant's annual source of supply is derived from the Kentucky River while the remaining 30% is derived from Jacobson Reservoir. Although the Kentucky River source can experience much higher turbidity spikes, the Jacobson Reservoir source water is more difficult to treat due to the presence of manganese and taste and odor causing compounds. Raw water quality at RRS will vary depending on the flow split between Kentucky River water Jacobson Reservoir.

The quality of the raw water entering RRS is generally good. TOC concentrations are relatively high, and the SUVA concentrations are relatively low reflecting a natural organic matrix that is difficult to remove. **Table 5-19** lists raw water characteristics at RRS for the 5-year period from 2006 to 2010. In addition to the overall daily values for the entire year, the table includes a breakdown of raw water quality conditions for the periods of each year between June 1 to September 30 and October 1 to May 30. This information is provided to confirm that more desirable conditions would occur during the June 1 to September 30 period during which maximum production capacity is likely to be needed to meet peak demands.

Jacobson Reservoir is aerated by an underwater piping system that distributes compressed air throughout the reservoir with the purpose of oxidizing iron and manganese. Additionally, potassium permanganate and copper sulfate can be added for control of taste and odors. The aeration system compressor and potassium permanganate feed equipment are both located at the Jacobson Reservoir pumping station. Copper sulfate is applied directly to the reservoir surface from a boat as needed. Permanganate is automatically fed at the Jacobson Reservoir pump intake based on pump operating status. Feeding at this location provides good contact time. However, because the raw water meter to RRS measures combined flows from the river and Jacobson Reservoir, the operator must manually adjust the feed rate depending upon the quantity of flow from each source which is determined by subtracting whatever flow is sent to KRS-1. In order to mix batches of potassium permanganate, about 100 lb of dry potassium permanganate is manually fed into a hopper each day which feeds into the mix tank from which the solution is pumped to the reservoir. The pumping station is located at a distance from RRS so it is labor intensive for operators to travel to the pump station to mix the solution each day. In addition, lifting and pouring the 55 lb bags of permanganate puts workers at risk for injuries.

Table 5-19 Richmond Road Station Raw Water Quality (January 2006 - December 2010)										
Raw Water Parameter	**Units**	**Daily Range**			**Jun 1 to Sep 30 only**			**Oct 1 to May 31 only**		
		Max	**Avg**	**Min**	**Max**	**Avg**	**Min**	**Max**	**Avg**	**Min**
Pumpage (mgd)	mgd	23.8	12.1	3.7	23.8	13.3	8.3	22.3	11.5	3.7
Turbidity	NTU	1,121	15	1	257	11	1	1,121	17	2
pH	Units	8.9	7.9	7.0	8.4	7.7	7.3	8.9	8.1	7.0
Total Alkalinity	mg/L	150	95	21	150	91	50	140	96	21
TOC *	mg/L	6.9	3.5	1.3	6.9	3.3	1.8	6.3	3.6	1.3
SUVA	L/mg-m									
Iron	mg/L									
Manganese	mg/L									
Total Hardness	mg/L									
Calcium Hardness	mg/L									
Total Dissolved Solids	mg/L									
Fluoride	mg/L	0.23	0.14	0.07	0.19	0.13	0.07	0.23	0.14	0.07

* TOC data is collected twice per month.

Pretreatment chemicals at RRS are primarily fed either upstream of or within two-stage rapid mix tanks at the inlets of each of two independent sedimentation basins. These chemicals include chlorine for pre-disinfection, caustic soda for pre-pH control, and polyaluminum chloride (PACl) and a cationic polymer for coagulation.

Hydrated lime can also be fed for pre-pH control, but would only be needed if high dosages of coagulant would deplete the alkalinity. Powdered activated carbon (PAC) is fed for taste and odor control at the effluent of the flocculation basin in each sedimentation basin. This is the only feed point for PAC. Previously PAC was fed at the rapid mix tank but was relocated because of the affinity between PAC and chlorine. Feeding PAC at this location minimizes residuals build up in the flocculation basins which are not equipped for automatic cleaning however this feed location is not ideal for providing sufficient contact time for PAC to effectively treat taste and odor episodes. KYAW should consider switching the feed points for PAC and pre-chlorine to allow for more contact time for the PAC. In addition, this will also slightly reduce some of the chlorine contact time which may help with reduce DBP formation within the plant. Calculations based on historic data indicate that the plant would still be able to meet CT requirements as long as the clearwell underneath the filters is included in disinfection credit. The capability to feed a filter aid directly to the filter influent also exists at RRS but has not been needed over the last five years. **Table 5-20** shows the pretreatment chemical usages experienced at RRS for the 5-year period from January 2006 to December 2010.

Two separate chemical buildings house the majority of the pretreatment chemicals. Chemical Building No. 1 houses the chlorine (gas) and caustic soda feed systems. A chlorine scrubber system is located outside of Chemical Building No. 1 in case of a chlorine leak. Chemical Building No. 2 includes the PACl, polymer, fluoride, corrosion inhibitor and powdered activated carbon (PAC) feed systems. Chemical Building No. 2 also houses the ammoniators used to feed ammonia from the storage tanks located outside the building. The filter aid feed facilities are located at the point of application in the filter gallery.

TABLE 5-20
Richmond Road Station
Pretreatment Chemical Usage (January 2006 - December 2010)

Pretreatment Chemical	Form *	Min Dosage (mg/L)	Avg Dosage (mg/L)	Max Dosage (mg/L)	No. of Days Used
Potassium Permanganate ****	Dry	0.05	0.76	2.43	793
Copper Sulfate	Dry				
Chlorine **	Gas	2.6	8.0	15.3	1,826
Caustic Soda	Dry	0.7	21.2	123.3	79
PACl	Dry	14.7	50.7	206.5	1,826
Cationic Polymer	Product	0.1	2.2	5.4	1,825
Powdered Activated Carbon ***	Dry	0.1	1.1	5.2	449

* The form of the chemical indicated in the table is not necessarily the form in which the chemical is fed; rather, it is the form in which the dosage numbers are based.

** This is all the chlorine fed at the plant during this time period. Post-chlorine has not been fed for the past five years.

*** Data is only available for the period from January 2007 through December 2010.

Two tables are presented below which compare the actual size or capacity of chemical storage and feed components with that which would be required when sizing these components using actual chemical dosages experienced over the five year period from 2006 to 2010. **Table 5-21** compares actual storage volumes to a range of calculated volumes based on [max day/avg dose] and [avg day/max dose] using 25 mgd as the max day to generally assess storage adequacy. **Table 5-22** compares total and reliable (largest feed unit out of service) feed equipment capabilities to actual feed rates, and also to a projected feed rate based on a 25 mgd production rate and the maximum dosage experienced over the last five years between

June 1 and September 30 when peak demands are most likely to occur.

TABLE 5-21 Richmond Road Station Pretreatment Chemical Storage						
Pretreatment Chemical	**Type of Storage**	**Actual Storage**	**Storage Required for 31 days**		**Actual Day Storage**	**Day Storage Required**
			Avg Day/ Max Dose	**Max Day/ Avg Dose**		
Potassium Permanganate	110 lb. drum	12 drums	71 drums	45 drums	N/A	N/A
Copper Sulfate	drums	N/A	N/A	N/A	N/A	N/A
Chlorine *	Ton cylinder	8 containers	13 containers**	12 containers**	N/A	N/A
Caustic Soda ***	Bulk tank	10,000 gal	12,300 gal	3,800 gal	500 gal	500 gal
PACl	Bulk tank	20,152 gal	64,000 gal	32,000 gal	1,100 gal	2,600 gal
Cationic Polymer	Bulk tank	3,742 gal	1,600 gal	1,900 gal	100 gal	75 gal
PAC	900 lb. bag	36 bags	18 bag	8 bags	N/A	N/A
Filter Aid Polymer	55 lb. bag	2 bags	N/A	N/A	300 gal mix	N/A

* This is all based on pre-chlorine dose. Post-chlorine has not been fed for the past five years.

** Based on 15 days for chlorine gas storage.

*** Includes both pre- and post-treatment feed.

The analysis in **Table 5-21** would infer that storage capabilities for some chemicals at RRS are inadequate. However, each system should be evaluated with respect to realistic use factors, e.g., not all of the chemicals are utilized daily nor were they used for 31 continuous days which would appropriately reduce the amount of storage required over a 31-day period. A lack of a full 31 days of storage does not necessarily mean the facilities are deficient provided that more frequent chemical shipments can be reliably received which is the case at RRS. Safety guidelines for chlorine gas recommend on-site storage be minimized so chlorine storage evaluation was based on a 15 day storage requirement.

<u>Potassium Permanganate</u> - Granular potassium permanganate is delivered in 55-pound pails to the RRS where it is stored before being transferred to the Jacobson Reservoir PS every few days. To prepare batches of potassium permanganate, several pails of dry potassium permanganate are manually fed into a hopper every few days. The hopper then feeds the permanganate into a dissolver/mix tank from which the solution is applied to the intake well. The Jacobson Reservoir PS is located several miles from RRS, so it is time consuming and labor intensive for operators to travel to the pump station to refill the hopper. Also, operators manually

carry the pails out to the intake enclosure and empty the pails into the dry hopper feeder. Lifting and pouring the 55-pound pails of permanganate puts workers at risk for injuries and creates some risk of a spill into the reservoir during the handling operation. The steel operating floor of the intake well is also exhibiting signs of corrosion.

Coagulants - Ferric chloride was formerly fed at RRS for coagulation of Jacobson Reservoir water. However, it was determined that coagulation with PACl was sufficient and preferred for operation since it is less expensive than ferric chloride. As a result, the ferric chloride storage and feed system was converted to a PACl storage and feed system in May 2011. Even with the increased volume from the ferric tank, the bulk tanks for PACl are slightly undersized. However, RRS has a reliable supplier for the PACl and delivery does not appear to be an issue. Therefore, no recommendations for increasing PACl storage are included in this study.

TABLE 5-22
Richmond Road Station
Pretreatment Chemical Feed

Pretreatment Chemical	Feed Units	Total Feed Capacity	Reliable Feed Capacity	Actual Feed Range
Potassium Permanganate	lb/day	1,580	790	110
Copper Sulfate	N/A	N/A	N/A	N/A
Chlorine (feeders) * Chlorine (evaporators) *	lb/day	9,000 8,000	6,000 5,000	80 - 2,040
Caustic Soda Pre- Post-	gal/hr	368 165	240 37	0.2 - 70.7
PACl	gal/hr	423.9	302.2	3.9 – 92.4
Cationic Polymer	gal/hr	8.4	5.6	0.04 – 3.74
PAC	ft^3/hr	45	0.00	0.02 – 1.11

* This is all based on pre-chlorine dose. Post-chlorine has not been fed for the past five years.

pH Adjustment - The caustic feed system was originally sized when lime was fed in pretreatment daily. With the discontinuance of lime, additional reliance on caustic soda has resulted. As such, the caustic soda feed system was improved in 2003 to meet 30 mgd production demands. Specifically, the feed pumps were replaced and more units were added to provide two (2) pre-pumps each with a max capacity of 120 gph and one (1) one post- pump with a max capacity of 37 gph. Additionally, one (1) swing pump with a max capacity of 128 gph was provided to provide reliability to either the pre- or post- feeds. The day tank was replaced with a 500 gal unit and the chemical spill containment was structurally repaired and relined. Caustic soda is used every day but only 7 days in the past five years the dose was above 50 mg/L. Therefore, the 10,000 gallon bulk storage tank is sufficient.

Pre-Disinfection - Although the chlorine feeder capacity is adequate and reliable, there is only a single evaporator, with no room for an additional evaporator. In 2003 improvements were made to ensure the evaporator could be by-passed and chlorine gas fed directly via a manifolded vacuum regulator arrangement in the event of an evaporator failure or service outage. The by-pass system consists of four to five individual vacuum regulators, each dedicated to a chlorine container. The chlorine containers are linked together via a standby piped manifold assembly to ensure a minimum capacity of 5,000 lbs/day can be provided. To implement the emergency gas feed system the ton containers within the chlorine feed room must be re-oriented and the vacuum regulators and manifold installed. It is estimated that such a switchover would take 2-3 hours. Manifolding cylinders is not typically a practical operational procedure since the withdrawal from the cylinders could be unequal, resulting in liquid chlorine remaining in some cylinders once the entire bank is taken out of service. However, in an emergency situation where the single evaporator is out of service, manifolding is a cost effective means of ensuring adequate feed rates up to approximately 4,500 lbs/day (withdrawal rate from 8 cylinders at 70 degrees F) which is more than enough to meet maximum feed requirements experienced over the last five years.

The existing storage for chlorine is of concern. The existing chlorine storage room can only accommodate eight (8) containers which equates to about 10 days of storage under maximum flow/average dose conditions. As a result, it is necessary to receive frequent shipments. However, receiving frequent shipments has not been a concern since the chorine vendor has been historically reliable. Therefore, no improvements for chlorine storage are recommended at this time.

Taste & Odor Control - The powdered activated carbon (PAC) feed and storage facilities were expanded in 2003 to accommodate a greater reliance on Jacobson Reservoir water during peak demand periods. This was done because Jacobson Reservoir has a higher probability of taste and odor episodes. Also, considered as a motive to expanding the PAC system was the concept that peak demands would reduce the empty bed contact time in the GAC filters, thereby, reducing the effectiveness of the GAC to remove taste and odor sources. Thus, the PAC feed capability at RRS was increased to provide a feed rate of 45 dry cf/hr which would equate to a maximum dosage of 97 mg/L at a 25 mgd production rate when using the currently applied PAC (Norit's Hydrodarco B PAC) which has a density of 18.75 lb/cf. Feeding PAC with 12 lb/cf density results in a maximum dosage capacity of 62 mg/L which meets the DOW's guideline of a 50 mg/L dosage to handle chemical spills (plumes) at the water source. Over the last five years, the maximum PAC dose needed to be fed at RRS was only 5.2 mg/L.

PAC is supplied in 900 lb bulk bags which are fed via a volumetric feeder and a screw conveyor to the top of the dry feeder. A bulk storage room which can accommodate a maximum of 36 bags when double stacked was also added as part of the PAC expansion. Applying a dosage of 50 mg/L for 30 days would necessitate a PAC bulk storage of 359 bulk bags. Such a provision is not practical, however, as the excessive dosage would only be warranted when treating a catastrophic "spill" in the source (e.g., river). Typically, a "spill" threat would only be realized for 1-3 days. **Table 5-21** indicates that bulk storage for PAC is adequate under a maximum dose/average flow condition.

Polymers – The filter aid, cationic and coagulant aid polymer feed system are all adequately sized to treat a production rate of 25 mgd and, therefore, no improvements are recommended.

5.2.2.3 Mixing/Flocculation/Sedimentation

Raw water is mixed, flocculated, and settled in two independent concrete basins which allow for separate chemical feed rates and degrees of mixing. The two adjacent basins operate in parallel, and are a mirror image of each other. Raw water first enters a two stage variable speed rapid mix chamber where coagulation occurs. Caustic soda, coagulant and coagulant aid polymer are fed directly into this chamber. From the rapid mix chamber, the chemically treated water enters a baffled zone where tapered flocculation occurs by means of turbine flocculators. This flocculation zone extends the entire length of the basin. From the flocculation zone, water is evenly dispersed to the sedimentation basin through a distribution flume extending the width of the basin. Each sedimentation basin is 83.5 feet wide, 200.6 feet long, and 8.75 feet deep with a total volume of approximately 1.1M gallons. The water then flows the length of the basin and ultimately exits over serpentine weirs into an effluent launder. Each basin is also equipped with a de-icing system to prevent structural damage to the concrete walls. The de-icing system equipment for the basins is housed in separate brick structures adjacent to each basin.

Tables 5-23 through **5-25** presented below evaluate the process adequacy of the rapid mixing, flocculation, and sedimentation processes at RRS. Over the five year period from 2006 to 2010, the settled water turbidity has averaged 0.88 NTU which is comfortably below the EPA partnership goal of 2.0 NTU. The primary focus of the evaluation in these tables is to check that the design parameters are adequate at the average and maximum flow rates of 12 and 25 mgd, respectively.

TABLE 5-23
Richmond Road Station
Rapid Mixing

Production Rate	**Mean Velocity Gradient, G**			**Detention Time, t**		
	Actual (sec^{-1})	**AWWA/ASCE** (sec^{-1})	**Ten State** (sec^{-1})	**Actual** (sec)	**AWWA/ASCE** (sec)	**Ten State** (sec)
12 mgd	528	600 - 1000	≥ 750	102	10 - 60	≤ 30
25 mgd	528			49		

The velocity gradient values indicated above are at average water temperature (66 deg F) and are not affected by plant production rates, but are shown here to confirm adequacy of the process. The combination of slightly low velocity gradient and slightly high detention time results in an acceptable degree of mixing.

<table>
<tr><th colspan="7">TABLE 5-24
Richmond Road Station
Flocculation</th></tr>
<tr><th rowspan="2">Production Rate</th><th colspan="3">Flow Through Velocity, V</th><th colspan="3">Detention Time, t</th></tr>
<tr><th>Actual (ft/min)</th><th>AWWA/ASCE (ft/min)</th><th>Ten State (ft/min)</th><th>Actual (min)</th><th>AWWA/ASCE (min)</th><th>Ten State (min)</th></tr>
<tr><td>12 mgd</td><td>5.8</td><td rowspan="2">0.9</td><td rowspan="2">0.5 - 1.5</td><td>50.0</td><td rowspan="2">10 - 30</td><td rowspan="2">> 30</td></tr>
<tr><td>25 mgd</td><td>12.1</td><td>24.0</td></tr>
<tr><th rowspan="2">Production Rate</th><th colspan="3">Velocity Gradient, G</th><th colspan="3">G x t</th></tr>
<tr><th>Actual (sec^{-1})</th><th>AWWA/ASCE (sec^{-1})</th><th>Ten State (sec^{-1})</th><th>Actual (unitless)</th><th>AWWA/ASCE (unitless)</th><th>Ten State (unitless)</th></tr>
<tr><td>12 mgd</td><td>47 - 62</td><td rowspan="2">20 - 70</td><td rowspan="2">n/a</td><td>141-186 x 10^3</td><td rowspan="2">30 - 60 x 10^3</td><td rowspan="2">n/a</td></tr>
<tr><td>25 mgd</td><td>47 - 62</td><td>68-89 x 10^3</td></tr>
</table>

The flocculation basins are designed with tapered flocculation (i.e. mixing intensity decreases though the basin) which reflects the range in actual G values. The parameter of most significance in a flocculation basin is the G x t value which is a measure of energy input over a period of time. Even though the “flow-through” velocities are high, the basins have historically performed extremely well which confirms the importance of the G x t value. The higher velocities help to prevent sedimentation in the flocculation basins.

Table 5-25 Richmond Road Station Sedimentation						
Production Rate	**Flow Through Velocity, V**			**Detention Time, t**		
	Actual (ft/min)	**AWWA/ASCE** (ft/min^1)	**Ten State** (ft/min^1)	**Actual** (hr)	**AWWA/ASCE** (hr)	**Ten State** (hr)
12 mgd	0.8	2.0 - 4.0	< 0.5	4.4	1.5 - 2.0	> 4.0
25 mgd	1.6			2.1		
Production Rate	**Surface Loading Rate**			**Weir Overflow Rate**		
	Actual (gpd/sf)	**AWWA/ASCE** (gpd/sf)	**Ten State** (gpd/sf)	**Actual** (gpd/ft)	**AWWA/ASCE** (gpd/ft)	**Ten State** (gpd/ft)
12 mgd	358	800 - 1,200	N/A	12,000	n/a	< 20,000
25 mgd	746			25,000		

At the maximum production rate of 25 mgd, the sedimentation basins are being operated within acceptable ranges for all criteria. In 2003 butterfly isolation valves with electric actuators were provided to replace manually operated gate valves. The valves allow either basin to be taken out of service and cleaned while the other remains in operation.

5.2.2.4 Filtration

Settled water flows by gravity from the sedimentation basins to sixteen (16) concrete box filters, each 20' x 17' with a surface area of 340 sf. The piping between the sedimentation basins and the filters was improved in 2003, whereby two 30-inch lines were installed to eliminate the potential for surcharging the sedimentation basin effluent weirs at plant flows greater than 25 mgd. The filter media provides filtration and adsorption with 8-inches assorted size silica gravel, 6-inches of 0.45-0.55 mm sand, and 24-inches of 0.8-1.0 mm granular activated carbon (GAC). Each filter is equipped with an outlet rate of flow control valve which is controlled by filter level, and with air wash for backwashing. Each of the RRS filters is equipped with SCADA turbidity monitoring such that a filter can be shut down immediately if its effluent turbidity reaches 1.0 NTU.

The equipment (including valves, operators, electrical, etc.) and piping in the filter gallery are in poor condition due to corrosion. The filter pipe gallery is extremely congested which makes working in this area difficult. Inadequate ventilation and dehumidification has accelerated the deterioration of the piping and valve actuators. Additionally, there is visible cracking of the filter walls and leaking in the filter gallery. In 2003, a dehumidifier was installed in the filter building to try to control the humidity in the filter gallery. However, there is still a considerable amount of moisture in the filter gallery. It is unknown how much of the moisture in the filter gallery is due to air flow conditions and pipe sweating and how much is related to leaking pipes and filters. There also appear to be leaks between the filter gallery and the chlorine contact chamber that is below the filters as there is a chlorine odor in the room. Expedited corrosion of equipment and piping could be a result of the chlorine. In addition, to the equipment and pipes, the aggressive environment in the filter gallery has resulted in the deterioration of the concrete and structural elements in the filter gallery. A project recommended to address these issues is described in Section 1.

Filter backwash water supply is stored in a 24-foot diameter, 15 foot high, bolted steel tank with a total volume of 50,000 gallons. The tank operates in parallel with a vertical turbine wash water pump located in the filter gallery, with a rated capacity of 1,000 gpm. The flow rate is controlled by a wash water rate of flow controller also located in the filter gallery. A high service connection is available for backwashing filters in emergencies or when the wash water tank is out of service for maintenance. Spent backwash is sent to one of the two wash water waste holding tanks, as detailed in the Residuals section. In early 2003 the filters were equipped with filter-to-waste capability. Filter-to-waste is automated through SCADA and is discharged to the wash water holding tanks.

Table 5-26 below evaluates the process adequacy of the filtration processes at RRS. Over the five year period from 2006-2010, the finished water turbidity has averaged 0.1 NTU and has never been higher than 0.37 NTU, well below the IESWTR values of 0.3 NTU and 1.0 NTU, respectively.

TABLE 5-26 Richmond Road Station Filtration						
Production Rate	**Surface Loading Rate**			**Surface Loading Rate (1 unit out)**		
	Actual (gpm/sf)	**AWWA/ASCE** (gpm/sf)	**Ten State** (gpm/sf)	**Actual** (gpm/sf)	**AWWA/ASCE** (gpm/sf)	**Ten State** (gpm/sf)
12 mgd	1.5	2.0 - 7.0	n/a	1.6	2.0 - 7.0	n/a
25 mgd	3.2			3.4		
Production Rate	**Empty Bed Contact Time, EBCT**			**Empty Bed Contact Time, EBCT (1 unit out)**		
	Actual (min)	**AWWA/ASCE*** (min)	**Ten State** (min)	**Actual** (min)	**AWWA/ASCE*** (min)	**Ten State** (min)
12 mgd	9.8	5 - 25	n/a	9.2	5 - 25	n/a
25 mgd	4.7			4.4		

* The guidelines shown above pertain to only the GAC portion of the media.

The DOW has indicated that filtration rates of 5.0 gpm/sf should not be exceeded. With a plant production rate of 25 mgd and one filter out of service for backwashing, the loading rate is 3.4 gpm/sf. Even with two filters out of service the loading rate is 3.6 gpm/sf at 25 mgd, well below the DOW limit. The filters are equipped with GAC for taste and odor control. Due to limited physical depth of the filter boxes, the EBCT is relatively low at maximum flow. However, as previously discussed the PAC feed capability at the sedimentation basins has been improved to ensure taste and odor issues can be adequately handled when treating Jacobson Reservoir water.

5.2.2.5 Clearwell and Finished Water Pumping Facilities

Filtered water enters a concrete clearwell located below the filters, designated as Clearwell No. 2, having a total volume of 600,000 gallons. Water from this clearwell flows through two 30-inch pipes to a second concrete clearwell, Clearwell No. 1, which is adjacent to the distributive pump building and has a total volume of 450,000 gallons. Water from Clearwell No. 1 flows to the distributive pumping units through a series of pipes. In 2003, two 30” pipes were provided between the clearwells to minimize head losses (which will help to maintain pumping capacity at higher flow rates) and prevent stagnation of water. Both clearwells are unbaffled; however, the majority of the disinfection credit at this plant is achieved in the sedimentation basins. Additional detention time in the clearwells might only be needed under extreme conditions.

Finished water is delivered to the distribution system by means of six (6) horizontal split case pumps located adjacent to Clearwell No. 1, and is metered by two (2) high service venturi meters located in vaults outside of the pump building. A summary of the distributive pumping capabilities at RRS is provided in **Table 5-27**.

Table 5-27
Richmond Road Station
Distributive Pumping

Distributive Pump	Driver	Individual Rated Capacity (mgd)	Worst Case Peak Capacity (mgd)	Best Case Peak Capacity (mgd)	Driver Size (HP)
Distributive Unit No. 6	Electric	6.5 @ 190’	5.4 @ 205’	7.4 @ 175’	250
Distributive Unit No. 7	Electric	10.0 @ 240’	11.4 @ 205’	12.0 @ 175’	500
Distributive Unit No. 8	Electric	4.0 @ 240’	4.7 @ 205’	5.0 @ 175’	200
Distributive Unit No. 9	Diesel	7.0 @ 235’	7.2 @ 205’	7.3 @ 175’	400
Distributive Unit No. 10	Electric Diesel	5.5 @ 231’	6.1 @ 205’	6.4 @ 175’	250 580
Distributive Unit No. 11	Diesel	4.0 @ 220’	4.5 @ 205’	5.0 @ 175’	200
		Total Capacity	39.3 @ 205’	43.1 @ 175’	
		Reliable Capacity	27.9 @ 205’	31.1 @ 175’	

The worst case and best case peak capacities in the table above reflect the two extremes of the operating conditions that will exist in the distribution system on a single day. The required capacity leaving the plant is 24.3 mgd which reflects a plant capacity of 25 mgd minus in-plant needs of 0.7 mgd. Based on computer modeling, the worst case scenario reflects a minimum hour system demand of 37.6 mgd when all the distribution storage tanks are filling. The discharge pressure from the plant in this scenario would be 205 feet. The best case scenario reflects a maximum hour system demand of 107 mgd with the Hume Road, Mercer Road, and Parker’s Mill pump stations operating. The discharge pressure from the plant in this scenario would be 175 feet. The respective flow rates were then determined from pump curves which were developed from field tests. It can be seen in the table above that the total and reliable capacities of the distributive pumping units at RRS are adequate to deliver 24.3 mgd.

5.2.2.6 Chemical Post Treatment and Finished Water Quality

All post treatment chemicals are fed within or at the discharge of Clearwell No. 1. These chemicals include chlorine (if needed) and ammonia for disinfection, caustic soda for pH adjustment, hydrofluosilicic acid for fluoridation, and zinc orthophosphate for corrosion control. **Table 5-28** shows the post treatment chemical usages experienced at RRS for the 5-year period from 2006 to 2010. The chemical usage for caustic soda was previously shown in the table for pretreatment chemical usage.

TABLE 5-28
Richmond Road Station
Post Treatment Chemical Usage (January 2006 - December 2010)

Post Treatment Chemical	Form *	Min Dosage (mg/L)	Avg Dosage (mg/L)	Max Dosage (mg/L)	No. of Days Used
Anhydrous Ammonia	Gas	0.9	1.2	2.3	1,725
Hydrofluosilicic Acid	Product	0.3	5.0	14.7	1,826
Corrosion Inhibitor	Product	0.1	3.5	14.7	1,824

* The form of the chemical indicated in the table is not necessarily the form in which the chemical is fed; rather, it is the form in which the dosage numbers are based.

Chlorine and caustic soda are located in Chemical Building No. 1 while hydrofluosilicic acid and corrosion inhibitor are located in Chemical Building No. 2. The feed equipment for ammonia is also located in Chemical Building No. 2 while the anhydrous ammonia storage tanks are located outside of the building. Two tables are presented below, similar to those which were presented for pretreatment chemicals, which compare the actual size or capacity of chemical storage and feed components with that which would be required when sizing these components using actual chemical dosages experienced over the five year period from 2006 to 2010. **Table 5-29** compares actual storage volumes to a range of calculated volumes based on [max day/avg dose] and [avg day/max dose] using 25 mgd as the max day. **Table 5-30** compares total and reliable (largest feed unit out of service) feed equipment capabilities to actual feed rates.

TABLE 5-29 Richmond Road Station Post Treatment Chemical Storage						
Post Treatment Chemical	**Type of Storage**	**Actual Bulk Storage**	**Bulk Storage Req'd for 31 days**		**Actual Day Storage**	**Day Storage Required**
			Avg Day/ Max Dose	**Max Day/ Avg Dose**		
Anhydrous Ammonia	Bulk cylinder	6,000 gal	1,428 gal	1,564 gal	N/A	N/A
Hydrofluosilicic Acid	Bulk tank	4,000 gal	4,570 gal	3,196 gal	150 gal	184
Corrosion Inhibitor	Bulk tank	4,000 gal	3,790 gal	1,853 gal	150 gal	153

The analysis in **Table 5-29** indicates that the storage facilities at RRS are adequate. However, the anhydrous ammonia tanks (two 3,000 gallon tanks) are located outdoors. Previous practice in the water industry had been to locate bulk anhydrous ammonia tanks outdoors since pure ammonia is lighter than air and would not create a safety hazard. However, in reality, if a large leak from one of these storage tanks would occur, the liquid ammonia would not vaporize immediately but would form a puddle on the ground which would vaporize over a period of time, possibly several hours. An ammonia scrubber is not provided at RRS. However, such a provision would require the capability to adequately neutralize the volume of a 3,000 gallon tank. The tanks would also need to be enclosed in a building in order to contain the leak and allow for scrubbing. Additionally, KYAW would need to make arrangements for discharge of the waste from the scrubber, if needed. A more practical and safer solution would be to convert to an aqua ammonia storage and feed system. However, based on costs, it is not recommended to convert to aqua ammonia at this time. If major improvements are needed for the anhydrous ammonia storage and feed system, it would be recommended to convert to aqua ammonia at that time.

TABLE 5-30 Richmond Road Station Post Treatment Chemical Feed				
Post Treatment Chemical	**Feed Units**	**Total Feed Capacity**	**Reliable Feed Capacity**	**Actual Feed Range**
Anhydrous Ammonia	lbs/day	985	510	55 - 251
Hydrofluosilicic Acid	gal/hr	12.2	6.1	0.1 – 6.2
Corrosion Inhibitor	gal/hr	21.4	6.1	0.1 – 15.8

Finished water quality from RRS is good, and all current Federal and State regulations are met. **Table 5-31** lists finished water characteristics at RRS for the five year period from 2006 to 2010.

TABLE 5-31
Richmond Road Station
Finished Water Quality (January 2006 - December 2010)

Finished Water Parameter	Units	Daily Range			Jun 1 to Sep 30 only			Oct 1 to May 31 only		
		Max	Avg	Min	Max	Avg	Min	Max	Avg	Min
Turbidity	NTU	0.37	0.10	0.04	0.32	0.11	0.04	0.37	0.10	0.04
pH	Units	7.8	7.3	6.9	7.8	7.2	6.9	7.8	7.3	6.9
Total Alkalinity	mg/L	450	177	60	320	194	86	450	169	60
TOC	mg/L	3.1	1.9	0.8	2.6	1.8	1.2	3.1	1.9	0.8
Iron	mg/L									
Manganese	mg/L									
Total Hardness	mg/L									
Calcium Hardness	mg/L	280	122	28	200	120	60	280	123	28
Total Dissolved Solids	mg/L									
Chlorine Residual (total)	mg/L	7.6	3.6	2.7	4.3	3.7	3.1	7.6	3.6	2.7
Fluoride Residual	mg/L	1.2	1.1	0.8	1.2	1.1	0.8	1.2	1.1	0.8
Orthophosphate	mg/L	1.6	1.2	0.9	1.4	1.2	1.0	1.6	1.2	0.9
Odor	TON									
Total Coliform	/100 mL									

Disinfection at RRS is achieved primarily in pretreatment and in Clearwell No. 2. The actual log inactivation is calculated daily based on the following parameters.

- 192’ of 30” pipe with a 1.0 baffling factor (Total flow).
- 64’ of 24” pipe with a 1.0 baffling factor (1/2 of flow).
- Rapid Mix/Flocculator volume of 224,000 gallons with a 0.5 baffling factor (1/2 of flow).
- Sedimentation basin volume of 1,271,600 gallons with a 0.3 baffling factor (1/2 of flow).
- 92’ of 24” pipe with a 1.0 baffling factor.
- Clearwell No.2 volume of 600,000 gallons with a 0.3 baffling factor (Total flow).

The Commonwealth of Kentucky requires a log inactivation of 1.0 for *Giardia.* The chlorine residual on top of the filters and the settled water pH values are used to calculate the required CT. **Table 5-32** below shows the actual disinfection achieved over the five year period from 2006 to 2010. Since it was recommended previously that KAW should consider switching the pre-chlorine and PAC feed points, the table includes log inactivation with and without the processes prior to the sedimentation basins for comparison.

Table 5-32
Richmond Road Station
Disinfection (January 2006- December 2010)

	Including All Pretreatment Processes	Without the Flocculation Basin and Upstream Piping
Minimum	0.67	0.55
Average	3.81	3.15
Maximum	15.86	13.08
Minimum Needed Chlorine Residual in Winter (mg/L)	1.1	1.5
Minimum Needed Chlorine Residual in Summer (mg/L)	0.5	0.6

It can be seen in the table that 1.0 log inactivation of *Giardia* is being met under typical plant conditions. Table 5-32 includes a recommendation for the minimum chlorine residuals that should be maintained in the summer and winter time in order to ensure that the plant achieves 1.0 log inactivation of *Giardia* at all times.

Source water for RRS has been classified into Bin 1 as part of the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR). As a result, no additional removal or inactivation of *Cryptosporidum* is required. Kentucky River raw water quality is discussed in **Section 5.2.1.3**.

The Stage 2 D/DBPR requires the RAA of the percentage of TOC removed divided by the required percentage of TOC removed exceeds 1.0. It can be seen in **Table 5-33** below that the precursor removal meets the regulatory limit for TOC removal. Disinfection by-product formation in the distribution system is discussed in Section **5.2.1.7**. As shown, there are no anticipated issues in meeting the Stage 2 D/DBPR under current operations. No modifications to the current system are recommended.

Table 5-33 Richmond Road Station Precursor Removal					
Finished Water Parameter	**Qtr 1**	**Qtr 2**	**Qtr 3**	**Qtr 4**	**MCL**
TOC Removal Ratio 2006	1.40	1.43	1.45	1.47	> 1.0
TOC Removal Ratio 2007	1.69	1.68	1.73	1.75	> 1.0
TOC Removal Ratio 2008	1.47	1.54	1.54	1.51	> 1.0
TOC Removal Ratio 2009	1.52	1.42	1.44	1.50	> 1.0
TOC Removal Ratio 2010	1.63	1.60	1.56	1.45	> 1.0

5.2.2.7 Treatment Residuals Management

Currently RRS does not recycle any waste streams to the head of the Plant as it had in the past in order to assist with enhanced coagulation efforts and to reduce the manganese levels. Upgrades of the residuals handling equipment were performed in 2008. Treatment residuals which accumulate in the sedimentation basins are removed by a mechanical system consisting of sludge collection headers traveling continuously along rails at the bottom of each basin with a hose connection for sludge withdrawal. The system includes two (2) residuals removal pumps in each basin which create a siphon that draws sludge through hose connection. The sludge collection headers travel on rail systems and are driven by an electrical drive unit. The sludge pumps are each rated for 200 gpm, and no more than two pumps (one in each basin) are ever operated concurrently

The residuals drawn from the sedimentation basins are directed to a splitter box that divides flow evenly to three (3) gravity thickeners which are each equipped with a center drive unit and rake arms. All three tanks are identical, each with a diameter of 35'-0", a depth of 11'-0", and an approximate volume of 80,000 gallons. A polymer feed system is available to improve the thickening process and prevent residuals carry over. Dry polymer is mixed into a solution for feeding. Approximately 25 lb of polymer is fed every day.

Residuals are withdrawn from the thickener(s) into an adjacent filter press building where dewatering occurs. The dewatering facilities consist of one (1) residuals belt press, two (2) VFD driven progressive cavity residuals pumps (each rated for up to 200 gpm), a cake conveyor/dumpster arrangement, and a polymer feed system. The belt filter press, residuals pumps and polymer feed system were all replaced as part of the 2008 upgrades. The dewatered residuals (cake) from the press are collected and conveyed to a dumpster outside of the filter press building. Dewatered sludge is periodically applied in a beneficial reuse area on the plant property. The press is capable of handling a loading of 80 gpm and is operated 16 hours per day, 5 days per week. The flow to the press is limited by the dry polymer mixing system for which batches cannot be mixed fast enough to keep up with higher flows. Since there have not been issues in treating residuals produced under these conditions, improvements to the polymer feed system are not recommended at this time.

Supernatant from the thickeners and filtrate from the belt filter press discharges to the wash water waste tanks. Each wash water waste tank is 50 feet in diameter with a center depth of 27 feet and an approximate volume of 350,000 gallons. Each tank is equipped with two (2) submersible 250 gpm residuals pumps which discharge periodically to the thickeners. Three (3) vertical turbine pumps are also installed in each tank which direct supernatant to Lake Ellerslie. As part of the 2008 improvements, a wall was constructed to separate the supernatant pump wells from the settling area of the tanks. Decant pipes were installed with floats and swivel joints to withdraw settled water from 2 feet below the surface of the tanks. Each supernatant pump is rated for 1,600 gpm. A venturi tube located in a vault next to the tanks meters the discharged supernatant.

Discharge of supernatant from the wash water tanks and dewatering process to Lake Ellerslie is granted under a NPDES permit. The NPDES permit requires that the total suspended solids (TSS) of the discharge be less than 30 mg/L. Samples are collected monthly for evaluating the TSS of the discharge. Sampling has shown that discharge requirements are being met. Dredging of Lake Ellerslie has to be performed approximately every two years due to deposition of solids and it is costly to perform. At low water elevations, the center line elevation of the decant piping is only about 3.5 feet above the invert of the wash water tank. This could result in the discharge of solids with the supernatant. This issue can be addressed by modifying the controls for the motor operated plug valve on the decant pipe so that the valve is closed when the water level in the wash water tank is higher than the current set elevation. Additional operation modifications can be made so that the timing of the decant is adjusted to allow for proper settling time prior to pumping of the supernatant. Since 2003, the waste stream to Lake Ellerslie is dechlorinated with sodium thiosulfate prior to discharge.

Table 5-34 below shows the theoretical sludge generation over the five year period from 2006 to 2010. The concentration in the sedimentation basins was assumed at 0.5%, and the effluent concentration from the thickeners was assumed at 2.0%.

TABLE 5-34 Richmond Road Station Theoretical Residuals Generation (January 2006 - December 2010)						
Frequency	**Dry Solids**		**0.5% Concentration (from Sed Basins)**		**2.0% Concentration (from Thickeners)**	
	Avg (lbs)	**Max (lbs)**	**Avg (gal)**	**Max (gal)**	**Avg (gal)**	**Max (gal)**
Daily	5,788	119,654	138,812	2,869,399	34,703	717,350
3-Day Average	5,787	89,337	138,783	2,142,246	34,696	535,581
7-Day Average	5,783	65,670	138,686	1,574,813	34,671	393,703
15-Day Average	5,768	37,422	138,319	897,411	34,580	224,353
30-Day Average	5,747	22,437	137,825	538,064	34,456	134,516

Based on the theoretical residuals produced and an assumed solids concentration from the sedimentation basins of 0.5%, the flow from the sedimentation basins is greater than 400 gpm (the capacity of the residuals pumps) about 54% of the time. Based on this, it can be assumed that the concentration of solids in the sedimentation basins is greater than 0.5%. High solids concentrations can create a problem for the solids removal equipment in the sedimentation basins. In order to reduce the concentration of solids in the sedimentation basins, it is recommended that KAW consider increasing the size of the residuals pumps.

The hydraulic loading rate when all three thickeners are in service and two sedimentation basin sludge pumps are operating (one in each basin) is 0.14 gpm/sf which meets the recommendations in AWWA/ASCE literature which identifies typical hydraulic loading rates of 0.12 – 0.15 gpm/sf. If the size of the residuals pumps is increased, then the recommended hydraulic capacity of the gravity thickeners may be exceeded. However, the more critical design criterion is solids flux with a AWWA/ASCE guideline of 4 - 10 ppd/sf for alum sludges. These guidelines are met when plant solids production is less than 30,000 ppd. Solids production was greater than 30,000 ppd only 28 days out of the five year period from 2006 to 2010 (less than 1% of the time). Therefore, the gravity thickeners appear to be adequately sized.

Assuming a 2% solids concentration from the gravity thickeners, the flow to the belt filter press is estimated to be 80 gpm over a 16 hour period when total solids production is less than 13,000 ppd. Solids production was greater than 13,000 ppd only 51 days out of the five year period from 2006 to 2010 (about 2.8% of the time). Therefore, the belt filter press appears to be adequately sized.

5.2.2.8 Electrical and Instrumentation

The existing RRS electrical distribution system consists of a single feed from Kentucky Utilities which supplies 480V, 3-phase power via power company owned transformers. Five transformers are fed from the power company including a pole mounted transformer which supplies the Electrical Control Building, a pad mounted transformer which powers the Distributive Pump Building, and a second pad mounted transformer which supplies Chemical Building No. 1, Chemical Building No. 2, and The Filter Building. Additional services include a pole mounted transformer which supplies the Sludge Removal Building, and a pad mounted transformer which powers the Filter Press Building.

Low Service Unit No. 3 at Jacobson Reservoir is equipped with a direct drive diesel engine which can deliver 13.4 mgd in a peak pumping scenario. Distributive Unit No. 9, No. 10, and No. 11 are also equipped with direct drive diesel engines which can deliver as much as 18.7 mgd during peak demand periods. In addition to the diesel engines, a 115 KW emergency generator is located in the distributive pump room to power the lights and chemical feed facilities in Chemical Building No. 1, or one of the low service pumps at Lake Ellerslie. Two (2) other emergency generators, one 250 KW and the other 500 KW, are located near Chemical Building No. 2 and the Filtration Building. These would run the chemical feed systems and the sedimentation basin mixing and flocculation equipment in the event of a power failure.

A Bristol Babcock distributed control system (DCS) was initially installed at RRS in the late 1980's and has expanded significantly since that time. Currently, the majority of the plant functions, with the exception of the distributive pumps, are monitored and controlled through the DCS across fiber optic cabling. Monitoring and control of most of the remote distribution sites (tanks, booster stations, and pressure monitoring stations) is accomplished via radio telemetry with RRS. KRS-1 and RRS are not currently able to communicate with each other nor is KRS-1 able to access the remote sites. Bristol 3330 RTUs and 3331 RIO racks are utilized throughout the plant in a master/slave configuration with redundant data concentrators while 3330 and 3305 RTUs, 3331 RIO racks, and 3508 transmitters are utilized at the remote (distribution) sites. ACCOL measurement and control communication software is installed in each of the RTUs. The control room is equipped with redundant workstations running Genesis for DOS, a dedicated e-mail computer running Lotus Notes across the company's frame relay WAN, and a dedicated particle counter computer. A Motorola People Finder pager system is used by the plant operators when outside of the control room for notification of critical alarms. Laptop computers are also available for programming RTUs.

Upgrade of the DCS was completed in 2003. Improvements include RTU and workstation hardware and software upgrades to ensure system reliability and allow for future expansion. Control logic, alarming logic, reporting capabilities, and system displays will be modified as necessary to ensure that the efficiency of the system is being maximized. Additionally, the modified system will include improved remote access capabilities including handheld Pocket PCs on site, WAN access to the KRS-1 plant, and supervisory access via the Internet. This will improve response time to alarm events and allow for more efficient use of the system.

5.2.3 Kentucky River Station 2 – Pool 3 Water Treatment Plant

The Kentucky River Station 2 (KRS-2) was constructed and placed into service in 2010 and is located approximately thirty (30) miles northwest of the City of Lexington on the Kentucky River Pool 3 at river mile 47.8. This is the newest addition to Kentucky American Water. KRS-2 has a rated (reliable) capacity of 20 mgd, with the potential for future expansion to 25 mgd. The station is a conventional surface water treatment plant utilizing flocculation, sedimentation with plate settlers, and filters. It derives its total source of supply from the Kentucky River with current withdraw permit allotment of 6 mgd from September to May and 20 mgd from June to August. It is connected to the KAW distribution system by approximately 31 miles of 42 inch transmission main and a booster station.

5.2.3.1 Raw Water Pumping Facilities

Raw water is withdrawn from the Kentucky River by means of three (3) parallel wedgewire intake screens that are located in the river approximately 25 feet offshore, connected by three (3) 30-inch intake mains to a wet well located below the raw water pump station. Potassium permanganate is injected at the screens for zebra mussel control and at the intake prior to entering the wet well for oxidation of some taste and odor. The pump station floor elevation is 502 feet, which is above the 500 year flood level of 499.4 feet. The raw water pump station is equipped with raw water appurtenances consisting of sample analyzers, turbidimeter, pH meter, temperature transmitter. In addition, there is also an ultrasonic level transmitter for wet well, air burst equipment for intake screen cleaning, potassium permanganate day tank and feed area with spill containment, eyewash and shower, and electrical room.

The four (4) vertical turbine raw water pumps (two VFD, two constant speed) withdraw the water from the wet well. The discharge piping from each raw water pump is equipped with combination air/vacuum relief valve, pressure gauge, check valve, butterfly valve, and surge anticipator valve. The water from Pump 1 and Pump 3 (and future Pump 5 for future expansion) exits through one 30-inch pipe, while the water from Pump 2 and Pump 4 exits a separate 30-inch pipe. The water from both 30-inch pipes combines on the south side of the raw water pump building and continues through a 42-inch pipe approximately 1,300 feet to the treatment plant.

A summary of the intake pumping facilities that provide raw water to KRS-2 is provided in **Table 5-35**.

Table 5-35 Kentucky River Station 2 Raw Water Intake Pumping					
	Pump No. 1	**Pump No. 2**	**Pump No. 3**	**Pump No. 4**	**Pump No. 5**
Type	**Vertical Turb.**	**Vertical Turb.**	**Vertical Turb.**	**Vertical Turb.**	**Future**
Rated Capacity (MGD)	10	10	7	7	6
Rated Head (feet)	325	325	325	325	325
Horsepower	700	700	500	500	500
Nominal Motor Speed	1,200	1,200	1,200	1,200	1,200
Motor Voltage	4160	4160	4160	4160	4160
Constant Speed or VFD	VFD	VFD	Constant	Constant	Constant
Discharge Diameter (inches)	20	20	16[1]	16[1]	16[1]
Discharge Centerline Elevation (feet)	505 +/_	505 +/_	505 +/_	505 +/_	505 +/_

(1) Increase to 20-inch connection at discharge isolation valve for future pump change-out to larger capacity

The total rated capacity of the raw water pumps (without including Pump 5) is 34 mgd, with a reliable capacity of 24 mgd. With the addition of Pump 5, the reliable capacity will increase to 30 mgd. Depending on system demands, the capacity of raw water pump station can range from 4 to 24 mgd.

5.2.3.2 Raw Water Quality

All of the plant's source of supply is derived from the Kentucky River. The quality of the raw water entering KRS-2 is generally good. **Table 5-36** lists raw water characteristics at KRS-2 since beginning operation in September 2010. Similar to KRS-1 and RRS, the table also includes a breakdown of raw water quality conditions for the periods between June 1 to August 31 and September 1 to May 31. This information is provided to confirm that more desirable conditions would occur during the June 1 to August 31 period during which a plant production capacity increases.

Table 5-36 Kentucky River Station 2 Raw Water Quality (September 2010 – July 2011)										
Raw Water Parameter	Units	Daily Range			Jun 1 to Aug 31 only			Sept 1 to May 31 only		
		Max	Avg	Min	Max	Avg	Min	Max	Avg	Min
Pumpage (mgd)	mgd	10.008	6.250	1.434	10.008	8.630	5.447	9.636	5.538	1.434
Turbidity	NTU	316.0	48.0	2.1	131.0	28.0	6.0	316.0	61.7	2.1
pH	Units	8.4	7.9	7.4	7.9	7.7	7.4	8.4	7.9	7.7
Total Alkalinity	mg/L	240	89.1	45	110	84.7	64	240	84.3	45
TOC (1)	mg/L	4.0	3.0	1.8	3.7	3.1	2.4	4.0	2.9	1.8
SUVA	L/mg-m	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Iron	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Manganese	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Total Hardness	mg/L	368	173.7	96	290	167.1	120	368	166.9	96
Calcium Hardness	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Total Dissolved Solids	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Fluoride	mg/L	0.21	0.14	0.08	0.19	0.14	0.11	0.21	0.14	0.08
Sodium	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Bromide	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Chloride	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Color	units	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Odor	TON	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Fecal Coliform	/100 mL	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Cryptosporidium (2)	Oocysts/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a

NOTE: n/a means data was not available

(1) TOC data is collected twice per month.

(2) Based on the results between July 1997-December 1998, the *Cryptosporidium* concentrations were less than 0.075 oocysts/L requiring no additional treatment (removal or inactivation) at the RRS treatment plant. Additional sampling conducted in 2006 were below the trigger of 0.075 oocysts/L, exempting KAW from a Bin 2 Classification. However, it should be noted that the need for additional *Cryptosporidium* treatment will be based on future sampling results.

5.2.3.3 Chemical Treatment

Chemical treatment is provided for oxidation of soluble iron and manganese, taste and odor control, pH adjustment, coagulation, corrosion control, disinfection, and emergency mitigation of chemical spills in the river. Both pre and post chemical treatment is utilized at KRS-2. A list of the chemicals, purposes, feed points, and controls are shown in **Table 5-37**. Secondary feed points are provided for some chemicals to allow for flexibility.

Table 5-37
Kentucky River Station 2
Pre and Post Chemicals

Chemical	Purpose		Feed Points	Control
Aqua Ammonia (NH_3)	Post	• Formation of chloramines to minimize DBP formation and provide total chlorine residual	Primary: Clearwell effluent (provide redundant feed lines)	Flow paced based on finished water flow and trimmed based on ammonia and total chlorine residual
			Secondary: Clearwell influent	Flow paced based on filter effluent flow and trimmed based on total chlorine residual
			Secondary: Influent to second cell of the clearwell	
Caustic Soda (NaOH)	Pre	Secondary: • pH adjustment and alkalinity • Addition for coagulation • Oxidation catalyst	Secondary: Raw water main, upstream of rapid mixers	Raw water flow paced and trimmed based on mixed water pH analyzer
			Secondary: Common filter influent	Flow paced based on filter effluent flow
	Post	Primary: • pH adjustment • Corrosion control	Primary: Clearwell Effluent (provide redundant feed lines)	Flow paced based on finished water flow and trimmed based on finished water pH analyzer
			Secondary: Clearwell Influent	Flow paced based on filter effluent flow and trimmed based on finished water pH analyzer
Chlorine (Cl)	Pre	Primary: • Disinfection • Inhibit microorganism growth in filter media and enhance particle removal	Primary: Common filter influent (provide redundant feed lines)	Flow paced based on filter effluent flow and trimmed based on filter influent sample

		Secondary: • Oxidation of iron and manganese	Secondary: Raw water main, upstream of rapid mixers.	Flow paced based on raw water flow
	Post	• Disinfection	Primary: Clearwell Influent, normally fed (provide redundant feed line)	Clearwell influent flow paced based on filter effluent flow and trimmed based on clearwell influent chlorine analyzer
			Primary: Clearwell effluent, boost residual	Flow paced based on finished water flow and total chlorine residual
			Secondary: Influent to second cell of the clearwell.	Flow paced based on filter effluent flow
Coagulant Aid Polymer	Pre	• Enhance coagulation process	Primary: Rapid mixer No. 2	-Flow paced based on raw water flow
			Secondary: Rapid mixer No.1	
			Secondary: Downstream of pre-treatment rapid mixer	
Corrosion Inhibitor (CI)	Post	• General corrosion control • Lead and copper solubility reduction	Primary: Clearwell effluent, downstream of the washwater suction, but upstream of the high service pumps (provide redundant feed lines)	Flow paced based on finished water flow
Dewatering Polymer	Post	• Improve dewatering	Primary: Belt filter press feed	Manually adjusted at the Dewatering Building
Ferric Chloride[1] (FeCl)	Pre	• Coagulation	Primary: Rapid mixer No. 1	Flow paced based on raw water flow and trimmed based on streaming current
			Secondary: Rapid mixer No. 2	
Filter Aid Polymer	Pre	• Enhance filter performance	Primary: Common filter influent	Flow paced based on filter effluent flow
Fluoride (Hydrofluosilicic Acid)	Post	• Fluoridation of the water to prevent dental cavities	Primary: Clearwell effluent, downstream of the washwater withdrawal, but upstream of the high service pumps (provide redundant feed lines)	Flow paced based on finished water flow
			Secondary: Clearwell influent	Flow paced based on filter effluent flow
Polyaluminum Chloride[1]	Pre	• Coagulation	Primary: Rapid mixer No. 1	Flow paced based on raw water flow and

(PACl)			Secondary: Rapid mixer No. 2	trimmed based on streaming current
	Post	• Assist in ripening filters	Primary: Filter washwater	Washwater coagulant based on concentration target, filter basin volume, and low wash rate
Potassium Permanganate ($KMnO_4$)	Pre	Primary • Oxidation of iron and manganese • Zebra mussel control	Primary: Raw water pumping station wet well, just after intake pipe entrance	Flow paced based on raw water flow
		Secondary • Oxidation of some tastes and odors	Secondary: Raw water intake screens	
Powder Activated Carbon (PAC)	Pre	Primary • Taste and odor control	Primary: Raw water pipe, as far upstream from WTP as practical (Provide multiple access points along feed line route for maintenance of feed hose - leave secondary containment ends open into access chamber for leak detection)	Flow paced based on raw water flow
		Secondary • Adsorption of color • Adsorption of NOM • Spill control		
Sodium Thiosulfate	Post	• Dechlorinating process wastewater	Primary: Clarified wastewater discharge pipe, at point where pipe enters WTP	Flow paced based on wastewater discharge flow and trimmed based on wastewater discharge chlorine residual
Wastewater Polymer	Post	• Enhance settling of residuals	Primary: Wastewater drain	Flow paced based on backwash flow
			Primary: Common sed basin blowdown	Flow paced based on sedimentation basin blow down

(1) The WTP does not be feed PACl and FeCl simultaneously, but there are multiple tanks to allow switching between the two coagulants as water quality necessitates.

The majority of the pretreatment chemicals at KRS-2 are fed either upstream of or within a two stage mechanical rapid mix tank located after the raw water flow meter and before entering the flocculator. These chemicals include chlorine for pre-disinfection, polyaluminum chloride (PACl) for coagulation, and potassium permanganate (at the raw water pump station) for oxidation. As shown above, other chemicals can be fed if needed during pretreatment, but KAW currently does not utilize those injection points. Also, currently ferric chloride and powdered activated carbon are not being fed into the system.

Post-treatment at KRS-2 occurs after the filters and/or clearwell. These chemicals include aqua ammonia for DBPs, caustic soda for pH adjustment and corrosion control, chlorine for post-disinfection, a corrosion inhibitor for general corrosion control, fluoride for fluoridation, and a wastewater polymer to enhance settling of residuals. As stated for the pretreatment, other chemicals can be fed if needed during post-treatment, but KAW currently does not utilize those injection points.

The plant also has facilities to feed powdered activated carbon for the adsorption of organic contaminants in the event of a chemical spill in the river. The ability to feed a filter aid, caustic soda, and/or chlorine directly on top of the filters also exists; however, the application of filter aid is rarely utilized. All the chemicals listed in **Table 5-37** are shown in **Table 5-38**, even if they are not currently utilized.

Table 5-38
Kentucky River Station 2
Chemical Usage

Chemical	Min Dosage (mg/L)	Avg Dosage (mg/L)	Max Dosage (mg/L)	No. of Day Used
Aqua Ammonia	2.21	5.81	9.68	281
Caustic Soda(1)	1.18	12.48	29.10	117
Chlorine(1)	4.67	5.86	8.47	291
Coagulant Aid Polymer	-	-	-	-
Corrosion Inhibitor (Zinc Ortho)	0.47	2.71	6.80	312
Dewatering Polymer	-	-	-	-
Ferric Chloride	-	-	-	-
Filter Aid Polymer	0.02	0.35	1.08	28
Fluoride (23%)	0.31	4.24	6.58	312
Polyaluminum Chloride	8.24	46.90	130.27	312
Potassium Permanganate	0.16	0.65	1.40	291
Powder Activated Carbon	-	-	-	-
Sodium Thiosulfate (30% by weight)	-	-	-	-
Wastewater Polymer-Backwash	-	-	-	-
Wastewater Polymer-Blowdown	-	-	-	-

(1) Includes pre and post.

All of the plant's chemicals are housed in the Chemical Building with the exception of the dewatering polymer, which is housed in the Residuals Handling Building. The first floor of the chemical building houses the ammonia, caustic soda, corrosion inhibitor (zinc orthophosphate), ferric chloride, fluoride, polyaluminum chloride, powder activated carbon feed system, and sodium thiosulfate. The second floor houses the chlorine, coagulant aid polymer, filter aid polymer, potassium permanganate, wastewater polymer, and stores the power activated carbon. There is a chlorine scrubber system provided and located on the north side of the building.

Table 5-38 shows the storage associated with the chemicals at KRS-2 based on the actual chemical dosages experienced from September 2010 to July 2011. A storage analysis was performed based on the chemical dosages experienced from September 2010 to July 2011. **Table 5-39** compares actual storage volumes to a range of calculated volumes based on [max day/avg dose] and [avg day/max dose] using 20 mgd and 6 mgd as the maximum and average day production rates, respectively. The storage analysis was not performed on any chemical that was not utilized during the timeframe.

Table 5-39 Kentucky River Station 2 Chemical Storage						
Chemical	**Type of Storage**	**Actual Bulk Storage**	**Bulk Storage Req'd for 31 days**		**Actual Day Storage**	**Day Storage Required**
			Max Day (20 mgd) /Avg Dose	**Ave Day (6 mgd) /Max Dose**		
Aqua Ammonia	Bulk	8,000 gal	6,008 gal	3,128 gal	150 gal	1,211 gal
Caustic Soda	2 Bulk Tanks	5,000 gal each; Total 10,000 gal	5,024 gal	3,661 gal	1,000 gal	203 gal
Chlorine	Ton Cylinder	10 cylinders (20,000 lbs)	30,301 lb (15 ton cylinders)	13,686 lb (7 ton cylinders)	NA	NA
Coagulant Aid Polymer	55 gal drum	4 drums (220 gal)	-	-	NA	NA
Corrosion Inhibitor (Zinc Ortho)	Bulk	5,000 gal	1,151 gal	902 gal	150 gal	46 gal
Dewatering Polymer(1)	Bulk	6,300 gal	-	-	100 gal	-
Ferric Chloride	Bulk	12,000 gal (1 future at 12,000 gal)	-	-	1,500 gal	-
Filter Aid Polymer	55 gal drum	1 drum	1,810 gal	1,745 gal	NA	NA
Fluoride	Bulk	6,000 gal	1,707 gal	828 gal	150 gal	69 gal
Potassium Permanganate	55 gal drum (330 lb net weight)	20 drums (6,600 lb total)	3,361 lbs	2,262 lb	NA	NA
Polyaluminum Chloride	2 Bulk Tanks	12,000 gal each; Total 24,000 gal	23,834 gal	20,688 gal	4,200 gal	961 gal
Powder Activated Carbon	Super sack	15,000 lbs	-	-	NA	NA
Sodium Thiosulfate	Bulk	5,000 gal	-	-	50 gal	-
Wastewater Polymer	55 gal drum	2 drums (110 gal)	-	-	NA	NA

(1) The largest amount of dewatering polymer used in a month since operation was 2,700 lbs, approximately 294 gal (see **Table 5-49**)

When analyzing **Table 5-39**, it should be noted that not all the chemical are used daily which would reduce the amount of storage required over a 31 day period. For example, the filter aid polymer was used for a total of 28 days of 312 days during September 2010 to July 2011. Refer to the last column of **Table 5-40** to see the frequency of the chemical usage.

Table 5-40 **Kentucky River Station 2** **Chemical Feed**				
Chemical	**Feed Units**	**Feed Rate Range**	**Number of Pumps**	**Actual Feed Range (min - max)**
Aqua Ammonia	gal/hr	0.42 – 6.30	2	0.29-3.43
Caustic Soda	gal/hr	0.57 – 82.27	3	0.19-5.35
Chlorine (feeders)	lbs/hr	33.36 – 1,500	4	3.08-13.34
Polymer Pumps[1]	gal/hr	0 – 1.0	6	0.005 – 0.23
Corrosion Inhibitor (Zinc Ortho)	gal/hr	0.11 – 3.95	2	0.09 – 1.33
Ferric Chloride	gal/hr	4.74 – 118.50	3	-
Fluoride	gal/hr	0.63 – 7.66	2	0.02 – 1.79
Potassium Permanganate	gal/hr	3.06 – 137.61	2	0.02 – 0.14
Polyaluminum Chloride	gal/hr	2.11 – 173.80	4	1.61 – 39.87
Powder Activated Carbon (feeder)	ft^3/hr	0.12 – 21.72	-	-
Sodium Thiosulfate	gal/hr	0.40 – 12.62	2	-

(1) THE PUMPS ASSOCIATED WITH COAGULANT AID POLYMER, FILTER AID POLYMER, DEWATERING POLYMER, AND WASTEWATER POLYMER. THE ACTUAL FEED RANGE IS THE MAX AND MIN BASED ALL THE CHEMICALS.

In general, the pretreatment chemical feed capabilities at KRS-2 are adequate. The minimum range of the actual feed falls below the range of the feed pump. These numbers are based on a 24 hour use and the actual numbers may not be for a total of 24 hours. No improvements recommended.

5.2.3.4 Rapid Mixing

Raw water from the raw water pump station enters the treatment plant, passes through the 30-inch raw water flow meter and is chemically dosed prior to entering the two (2) stage rapid mixers where coagulation occurs. Rapid Mixer No 1 flows from low to high, while Rapid Mixer No. 2 flows from high to low. Depending on the flow through the plant, only one mixer may be needed. All pretreatment chemicals are fed either upstream of or directly within the rapid mixers. **Table 5-41** presented below evaluates the process adequacy of the rapid mixing processes at KRS-2. The primary focus of the evaluation is to compare AWWA/ASCE design guidelines and Ten State Standards with the operation of the plant at various production rates.

TABLE 5-41 Kentucky River Station 2 Rapid Mixing						
Production Rate	Velocity Gradient, G			Detention Time, t		
	Actual (sec^{-1})	AWWA/ASCE (sec^{-1})	Ten State (sec^{-1})	Actual (sec)	AWWA/ASCE (sec)	Ten State (sec)
6.0 mgd[1]	1000	600 - 1000	n/a	26	10 - 60	< 30
20.0 mgd[2]	1000			15		

(1) With one mixer in service.

(2) With both mixers in service.

G values indicated above are at 0.5 degrees centigrade. G values are not affected by plant production rates, but are shown here to confirm adequacy of the process. Detention times are adequate, and no improvements are recommended.

5.2.3.5 Flocculation and Sedimentation

Coagulated water leaving the rapid mix tank(s) travels through piping to the lower influent flume for the four (4), three (3) stage flocculators, with two (2) parallel compartments per stage. The water travels from the lower influent flume, to the upper influent flume via mud valves, and enters the first stage of the flocculator. There is a bypass to allow the coagulated water to enter the upper influent flume so that the lower flume can be cleaned. Each parallel flocculation compartment is 14.7 feet wide, 15.8 feet long, and 14.7 feet deep with a total stage volume of 51,000 gallons. Each compartment has one horizontal paddle wheel assembly oriented parallel to the flow, with one drive for Stage 1 and a common drive for Stage 2 and Stage 3 per flocculator. The overall dimensions of each three (3) stage flocculator is 30 feet wide, 47.5 feet long, 14.7 feet deep with a total volume of 153,000 gallons. From the flocculation zone, water enters the sedimentation basin through flow diffusers. Each sedimentation basin is 30.5 feet wide, 36.08 feet long, and 19 feet deep with a volume of 156,000 gallons. To increase detention time, each sedimentation basin is equipped with 5 rows of plate settlers (total of 500 plates per basin), with an effective surface area of 10,324 square feet per basin. The basins are equipped with sludge scrapers, sludge collector, and a 4 inch blowdown.

Tables 5-42 and **5-43** presented below evaluate the process adequacy of the flocculation and sedimentation processes at KRS-2. The primary focus of the evaluation in these tables is to check that the design parameters are adequate at various production rates.

TABLE 5-42
Kentucky River Station 2
Flocculation

Production Rate	Flow Through Velocity, V			Detention Time, t		
	Actual (ft/min)	AWWA/ASCE (ft/min)	Ten State (ft/min)	Actual (min)	AWWA/ASCE (min)	Ten State (min)
6 mgd[1]	0.9	0.9	0.5 - 1.5	110	10 - 30	> 30
6 mgd[2]	0.6			147		
20 mgd[2]	2.1			44		
Production Rate	**Velocity Gradient, G**			**G x t**		
	Actual[3] (sec^{-1})	**AWWA/ASCE (sec^{-1})**	**Ten State (sec^{-1})**	**Actual (unitless)**	**AWWA/ASCE (unitless)**	**Ten State (unitless)**
6 mgd[1]	35 – 75	20 - 70	n/a	231 – 496 x 10^3	30 - 60 x 10^3	n/a
6 mgd[2]	35 – 75			308 – 661 x 10^3		
20 mgd[2]	35 – 75			92 – 198 x 10^3		

(1) With three flocculators in service.

(2) With four flocculators in service.

(3) Velocity Gradient range is for three stages, Stage 1 is 75, Stage 2 is 50, and Stage 3 is 35.

The velocity gradient, G value, is based on the amount of horsepower that is put into the each flocculator. The range is due to the decrease in horsepower for each stage. There is high detention time, resulting in a high Gxt value. The flow through velocity is high, but this will prevent sedimentation in the flocculation basin. At low flows (6 mgd or less), the treatment plant is normally operated with only three flocculation basins online. No improvements are recommended.

TABLE 5-43
Kentucky River Station 2
Sedimentation

Production Rate	Flow Through Velocity, V			Detention Time, t		
	Actual (ft/min)	AWWA/ASCE (ft/min[1])	Ten State (ft/min[1])	Actual (hr)	AWWA/ASCE (hr)	Ten State (hr)
6 mgd(1)	0.32	2.0 - 4.0	< 0.5	1.9	1.5 - 2.0	> 4.0
6 mgd(2)	0.24			2.5		
20 mgd(2)	0.8			0.8		

Production Rate	Surface Loading Rate			Weir Overflow Rate		
	Actual (gpd/sf)	AWWA/ASCE (gpd/sf)	Ten State (gpd/sf)	Actual (gpd/ft)	AWWA/ASCE (gpd/ft)	Ten State (gpd/ft)
6 mgd(1)	1,817	800 - 1,200	< 720	889	n/a	< 20,000
6 mgd(2)	1,363			667		
20 mgd(2)	4,544			2,222		

Production Rate	Plate Surface Overflow Rate		
	Actual (gpm/sf)	AWWA/ASCE (gpm/sf)	Ten State (gpm/sf)
6 mgd(1)	0.13	n/a	<0.5 at 80% efficiency
6 mgd(2)	0.10		
20 mgd(2)	0.34		

(1) With three sedimentation basins in service.

(2) With four sedimentation basins in service.

The surface loading rate calculated is high at both capacities and also when only three sedimentation basins are in service. Due to the plate settlers, the basins are capable of being sized smaller, therefore resulting in a higher surface loading rate. The main parameter is the plate surface overflow rate, which for all flows is adequate, resulting in adequate settling of the solids. No improvements are recommended.

5.2.3.6 Filtration

Settled water flows by gravity from the sedimentation basis, through the clarifier effluent flume and filter influent flume, to the five (5), concrete box filters, each 27'x 13'. A common filter influent turbidimeter, level transmitter, and sample tap is provided. Each filter has two cells totaling 702 square feet. As stated earlier, KAW has the ability to feed chlorine, caustic soda, and filter aid prior to the settled water entering the filters. The filter media consists of 12-inches of support gravel, 12-inches of 0.45-0.55 mm silica sand, and 18-inches of 0.95-1.05 mm

anthracite. There is additional depth provided for granular activated carbon (GAC) in lieu of anthracite in the future, if necessary, for taste and odor control.
The flow to the filters is equally split among the online filters while maintaining constant influent level. The filters are sized so that a maximum surface loading rate of 5 gpm/sf is not exceeded with one filter out of service. Each filter is equipped with a head loss transmitter with high alarm, flow transmitter, effluent turbidimeter with high alarm, and effluent particle monitor. A common filter effluent sample tap and turbidimeter is also provided. As stated earlier, KAW has the ability to feed chlorine, caustic soda, ammonia, and fluoride on the combined filter effluent pipe, prior to entering the clearwell.

Filter backwash is initiated based on loss of head, filter effluent turbidity, or time in service. The backwash method is air scour and upflow water wash, with the source of backwash water coming from finished water in the clearwell via backwash pumps (one duty, one standby each rated at 8,780 gpm). The backwash pumps are located in the High Service Pump Station. The backwash rate of control is based on a venturi meter and rate of flow control valve. The filters are designed so that one cell can be washed while the other remains in service. The filter backwash and rinse water is directed to one of the two 313,700 gallon wastewater clarifiers.

Table 5-44 below evaluates the process adequacy of the filtration processes at KRS-2. The filter effluent turbidity from September 2010 to July 2011 averaged 0.06 NTU with a maximum value of 0.24 NTU, which is below the IESWTR MCL of 0.3 NTU and 1.0 NTU, respectively. The primary focus of the evaluation in these tables is to check that the design parameters are adequate at various production rates.

<table>
<tr><th colspan="7">TABLE 5-44
Kentucky River Station 2
Filtration</th></tr>
<tr><th rowspan="2">Production Rate</th><th colspan="3">Surface Loading Rate</th><th colspan="3">Surface Loading Rate (1 filter out)</th></tr>
<tr><th>Actual (gpm/sf)</th><th>AWWA/ASCE (gpm/sf)</th><th>Ten State (gpm/sf)</th><th>Actual (gpm/sf)</th><th>AWWA/ASCE (gpm/sf)</th><th>Ten State (gpm/sf)</th></tr>
<tr><td>6 mgd</td><td>1.19</td><td rowspan="2">2.0 - 7.0</td><td rowspan="2">n/a</td><td>1.48</td><td rowspan="2">2.0 - 7.0</td><td rowspan="2">n/a</td></tr>
<tr><td>20 mgd</td><td>3.96</td><td>4.95</td></tr>
<tr><th rowspan="2">Production Rate</th><th colspan="6">Surface Loading Rate (3 filters in service)</th></tr>
<tr><th colspan="2">Actual (gpm/sf)</th><th colspan="2">AWWA/ASCE (gpm/sf)</th><th colspan="2">Ten State (gpm/sf)</th></tr>
<tr><td>6 mgd</td><td colspan="2">1.98</td><td colspan="2">2.0 - 7.0</td><td colspan="2">n/a</td></tr>
</table>

With the plant production rate of 20 mgd and one filter out of service, the loading rate is 4.95 mgd/sf, which is below the design loading rate of 5 gpm/sf. As stated earlier, GAC media is not utilized at this time, but the filters are designed to install GAC if taste and odor becomes an issue. No improvements are recommended.

5.2.3.7 Clearwell and Finished Water Pumping Facilities

Filtered water is directed to the two (2) cell clearwell that is located partially below the filters. Each cell of the clearwell is 99.25 feet long, by 54.42 feet wide, by 20 feet high with a capacity of 808,000 gallons (total 1,616,000 gallons). Each cell has three (3) baffled walls for a baffling factor of 0.7. The influent to each cell is equipped with ammonia feed points, but this is typically injected at the clearwell effluent. Chlorine is injected at the influent to the clearwell for disinfection.

Finished water is delivered to the distribution system through a 42-inch main by means of four (4) pumps (two VFD, two constant speed) which take suction from the clearwell. The water withdrawn from the clearwell travels through a 48-inch pipe. Chemical injection points consist of ammonia, chlorine, caustic soda, and fluoride. Ammonia is the most common chemical injected; the other chemicals are secondary injection points that allow KAW flexibility. The pump capacities are identical to the raw water pumps as shown below in **Table 5-45**. The discharge piping from High Service (HS) Pump 1 and Pump 2 are each equipped with combination air/vacuum relief valve, pressure gauge, check valve, butterfly valve, and surge anticipator valve. HS Pump 3 and Pump 4 are each equipped with combination similar appurtenances as HS Pump 1 and Pump 2, except a hydraulic ball valve is utilized instead of a check valve. All the pumps discharge through a 30-inch main that connects to the 42-inch transmission main. The 30-inch main is equipped with a venturi flow meter.

Table 5-45 Kentucky River Station 2 High Service Pumping					
	Pump No. 1	Pump No. 2	Pump No. 3	Pump No. 4	Pump No. 5
Type	**Vertical Turb.**	**Vertical Turb.**	**Vertical Turb.**	**Vertical Turb.**	**Future**
Rated Capacity (MGD)	10	10	7	7	6
Rated Head (feet)	324	324	324	324	324
Horsepower	700	700	500	500	500
Nominal Motor Speed	1,200	1,200	1,200	1,200	1,200
Motor Voltage	4160	4160	4160	4160	4160
Constant Speed or VFD	VFD	VFD	Constant	Constant	Constant
Discharge Diameter (inches)	20	20	12[1]	12[1]	12[1]
Discharge Centerline Elevation (feet)	745.5	745.5	745.5	745.5	745.5

[1]Increase to 20-inch connection at discharge isolation valve for future pump change-out to larger capacity

The total rated capacity of the high service pumps (without including Pump 5) is 34 mgd, with a reliable capacity of 24 mgd, similar to that of the raw water pump station. With the addition of Pump 5, the reliable capacity will increase to 30 mgd. Depending on system demands, the capacity of high service pump station can range from 4 to 24 mgd.

5.2.3.8 Finished Water Quality

Finished water quality from KRS-2 is generally good, and all current Federal and State regulations are routinely met. **Table 5-46** lists finished water characteristics at KRS-2 for the period from September 2010 – July 2011.

TABLE 5-46 Kentucky River Station 2 Finished Water Quality (September 2010 – July 2011)										
Finished Water Parameter	Units	Daily Range			Jun 1 to Sep 30 only			Oct 1 to May 30 only		
		Max	Avg	Min	Max	Avg	Min	Max	Avg	Min

Turbidity	NTU	0.24	0.06	0.02	0.08	0.05	0.03	0.24	0.05	0.02
pH	units	7.8	7.2	7.0	7.2	7.1	7.0	7.8	7.3	7.0
Total Alkalinity	mg/L	220	79.6	40	90	76.1	58	220	166.9	96
TOC	mg/L	2.5	2.0	1.3	2.4	2.3	1.7	2.5	2.0	1.3
Iron	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Manganese	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Total Hardness	mg/L	310	177.5	94	270	162.4	105	310	163.4	94
Calcium Hardness	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Total Dissolved Solids	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Chlorine Residual (total)	mg/L	4.6	1.6	0.2	0.9	0.3	0.2	4.6	4.1	3.7
Fluoride Residual	mg/L	1.27	0.14	0.08	0.19	0.14	0.11	0.21	0.14	0.08
Sodium	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Bromide	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Chloride	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Orthophosphate	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Zinc	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Aluminum	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Odor	TON	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Ammonia	mg/L	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Total Coliform	/100 mL	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a

NOTE: n/a means data was not available

Disinfection credit at KRS-2 can be achieved through the combination of the flocculation basins, sedimentation basins, filters, and clearwell. In the event of filter failure, the clearwell was sized to provide all the required CT. During typical operation, ammonia is fed at the clearwell effluent for to prevent the formation of disinfection by-products (DBPs) in the distribution system. There are also ammonia feed points on the influent to cell 1 or cell 2 of the clearwell, but they are not utilized. The actual log inactivation is calculated daily based on flow rates through the following processes with capacities:

- Four Flocculation Basins for a total of 611,200 gallons with a baffling factor of 0.5
- Four Sedimentation Basins for a total of 267,700 gallons with a baffling factor of 0.5 (actual total capacity is 624,000 gallons, the 267,700 gallons does not include the area below the plate settlers).
- Four Filters for a total capacity of 185,000 gallons (assumes one out of service and does not include media and underdrain volume)
- Clearwell with two cells for a total capacity of 1,393,700 (original capacity is 1,616,000 gallons, but used a reduced volume for filter backwash water)

The Commonwealth of Kentucky requires a log inactivation of 1.0 for Giardia. **Table 5-47** below compares the adequacy of the actual disinfection to the required disinfection (in terms of log inactivation) from September 2009 to July 2010. The table shows the effectiveness of the plant in meeting disinfection requirements for historic flows (from September 1 to May 31 and June 1 to August 31) and the maximum production allowance based on withdraw limit of 20 mgd.

Table 5-47
Kentucky River Station 2
Disinfection (September 2009 – July 2011)

Period	**Actual Log Inactivation @ Historic flows**			**Required Log Inactivation**	**Log Inactivation at Maximum flow rate**			
Jun 1 - Aug 31	*Best Case*	*Typ. Case*	*Worst Case*	1.0	**Jun 1 - Aug 31**	*Best Case*	*Typ. Case*	*Worst Case*
Max Flow 10.01 mgd	7.44	2.69	1.01		Max Production 20.0 mgd	3.72	1.34	0.51
Avg Flow 8.63 mgd	8.63	3.12	1.18					
Min Flow 5.45 mgd	13.67	4.48	1.86					
Sept 1 - May 31					**Sept 1 - May 31**			
Max Flow 9.64 mgd	24.86	9.14	6.07		Max Production 20.0 mgd	11.98	4.40	2.92
Avg Flow 5.54 mgd (1)	41.18	15.79	10.54					
Min Flow 1.43 mgd (1)	159.03	60.98	40.72					

Worst Case: calculated using the following historic records: highest settled water pH, the lowest free chlorine residual for each treatment step, and the lowest temperature recorded for the season identified.
Best Case: calculated using the following historic records: the lowest settled water pH, the highest free chlorine residual for each treatment step, and the highest temperature recorded for the season identified.
Typ. Case: calculated using the following historic records: the average settled water pH, the average free chlorine residual for each treatment step, and the average temperature recorded for the season identified.
(1) Only includes three flocculation basins, three sedimentation basins, and three filters since common operation of 6 mgd and lower.

It can be seen in the table that the disinfection requirements at KRS-2 are met comfortably met under all conditions, except from June 1 to August 31 at worse case conditions at a maximum flow of 20 mgd. KAW should be advised that the when operating at the maximum flow under the worst case, a minimum chlorine residual 0.48 mg/L through the clearwell should be maintained in order to provide 1.0 log inactivation. If pre-chlorination is provided, the chlorine residual through the clearwell can be adjusted. Also, if the temperature decreases and the pH increases, the minimum chlorine residual may need to be adjusted when operating at 20 mgd.

Table 5-48 below shows the TOC removal ratio as well as the disinfection by-product concentrations for September 2010 to July 2011. TOC is sampled twice a month. A sample is taken at the source water and the filter effluent in order to determine the percent that is removed.

TABLE 5-48
Kentucky River Station 2
Precursor Removal

Finished Water Parameter	Units	Qtr 1	Qtr 2	Qtr 3	Qtr 4	MCL
TOC Removal Ratio 2010	unitless	-	-	1.33	1.26	> 1.0
TOC Removal Ratio 2011	unitless	1.26	1.33	-	-	> 1.0

It can be seen in the table above that the precursor removal meets the regulatory limits. No modifications to the current system are recommended.

Based on the new Stage 2 D/DBP Rule, the TTHM and HAA5 concentrations will be based on LRAA instead of a RAA per area served by a treatment plant. **Exhibit 5-1** and **5-2** (found under **Section 5.2.1.7**) show the TTHM LRAA concentrations and the HAA5 LRAA concentrations for the Stage 1 sites for KAW compared to the MCLs of 80 ppb for TTHM and 60 ppb for HAA5. Although the sites are different for Stage 2, based on the results shown in the Exhibits above, KAW will meet the MCL for MCLs for TTHM and HAA5 under the Stage 2 D/DBP Rule. No modifications to the current disinfection schemes at KRS-1, RRS, and KRS-2 are recommended.

5.2.3.9 Treatment Residuals Handling

The wastewater treatment system includes two wastewater clarifiers for the filter backwash and the rinse water, and one residuals thickener for sludge from the sedimentation basin and settled sludge from the wastewater clarifiers. The wastewater clarifiers are 60 feet diameter by 14 feet high with a total volume of 313,700 gallons each. The residuals thicker is 113 feet in diameter by 22.5 feet high, with a total volume of (including hopper) 2,041,000 gallons. The sludge thickener has a conic base with a sludge hopper. The decant from the wastewater clarifiers and sludge thickener will be pumped and discharged to the Kentucky River. The NPDES permit requires that the total suspended solids (TSS) of the discharge be less than 30 mg/L on a monthly average, with a maximum daily discharge limit of 50 mg/L. The sludge from the thickener is pumped to the dewatering facility.

The dewatering facility consists of two filter presses and the dewatering polymer feed system. The dewatering polymer is applied to the filter press to assist in dewatering the sludge. The thickened residuals are land applied to a site that is owned by KAW and located south of the raw water pump station. The filtrate from the dewatering process flows by gravity back to the sludge thickener. **Table 5-49** provides a summary of the amount of sludge generated (in tons) and the amount of dewatering polymer used (in lbs) from October 2010 to September 2011.

TABLE 5-49
Kentucky River Station 2
Polymer Usage – Sludge Generated

Month	Polymer (lb)	Tons Generated
October 2010	10	60
November 2010	385	171
December 2010	353	60
January 2011	183	50
February 2011	189	40
March 2011	625	181
April 2011	1350	474
May 2011	900	433
June 2011	900	131
July 2011	2700	494
August 2011	1350	393
September 2011	900	91
Average	820	215

Based on the table above, the size of the sludge thickener is adequate for the amount of sludge generated. No improvements are recommended.

5.2.3.10 Electrical

The existing plant electrical distribution system consists of a feed from a Kentucky Utilities step down transformer rated at 4160V, 3-phase power and is connected at the KRS-2 Water Treatment Plant. Normal and standby power is provided for the raw water pump station with two feeders from the WTP. The multiple switchboards are provided for redundancy of feed to the Raw Water Pump Station as well as the water treatment plant in the event a switchboard is out of service for maintenance. In addition to the electrical service, one stationary diesel powered standby 4160V power generator is located on the east side of the treatment plant. If additional power is needed, besides the generator, a plug receptacle for a mobile generator is also provided. In the event of a power failure, an automatic transfer switch will initiate the use of the generator. The standby generator is capable of operating the plant at 10 mgd.

5.2.4 Northern System Improvements

The Owenton facility was acquired by KAW in 2005 and serves KAW's Northern District System. A 2009 engineering evaluation conducted by KAW compared the cost of supplying water to the northern district via the Owenton facility or via a 16 mile transmission main from newly constructed KRS-2. The assessment evaluated unit processes at the Owenton facility and assessed such factors as regulatory compliance, reliability, safety and efficiency, among others. The assessment concluded that retirement of the Owenton system and construction of a 16 mile transmission main was most cost effective and met the aforementioned factors more effectively than maintaining the current Owenton facility.

The Owenton plant operates with a single treatment process train from the raw water transfer through the sedimentation process into two filters resulting in no redundancy and limited reliability. The reliance on the single claricone is a source of many issues and creates the potential for complete plant failure. Minor treatment upsets within the claricone require the system to rely solely on the limited storage capacity until treatment can be re-established. Portions of the system can be served from Purchase Agreements that KAW maintains with adjacent water districts however the central portion of the Northern Division, including most of Owenton, would still be without water for several hours while the claricone is out of service.

Also, the chemical storage facilities are inadequately sized. Chemicals must be purchased in small batch quantities due to inadequate containment, including lack of chlorine containment. In order to accommodate the lack of containment, small quantities of liquid solutions are purchased and stored elsewhere, and only the container being pumped from is located at the plant.

Additionally, the Owenton WTP has no provisions for residuals processing and the lack of discharge monitoring has been identified by Kentucky Division of Water (DOW) inspectors as a risk for a violation to the discharge permit. Filter backwash water and residuals from the claricone are piped to a settling basin, a remnant from the previous water treatment plant, which is located on an adjacent property and accessed through a narrow gravel road. Periodically, the supernatant is discharged from the top of the settling basin into the creek under a discharge permit. The walls of the concrete settling basin are rapidly deteriorating with rebar exposed nearly all the way around. There is no monitoring equipment to determine the amount of settling that has occurred or the volume of sludge in the basin. Currently there is limited ability to remove the sludge from the basin. The discharge permit limits are monitored through grab sampling.

Finally, there are only two filters at the plant and both are required for operation. While one is out of service for backwash, the plant is generally unable to keep up with normal system demands. Like the claricone, this situation means that the system must rely on its tanks for help in meeting demands even for minor treatment disturbances with either filter. This leaves no ability for extended maintenance, when necessary, for either filter. The sand filters are shallow, and limited in their ability to remove turbidity because of the short detention times required. This puts the entire system at risk for not meeting water quality standards. In fact, before KAW acquired the Owenton WTP, the plant was frequently unable to meet the current THM and HAA standards for disinfection by-products.

Alternative to the Owenton WTP

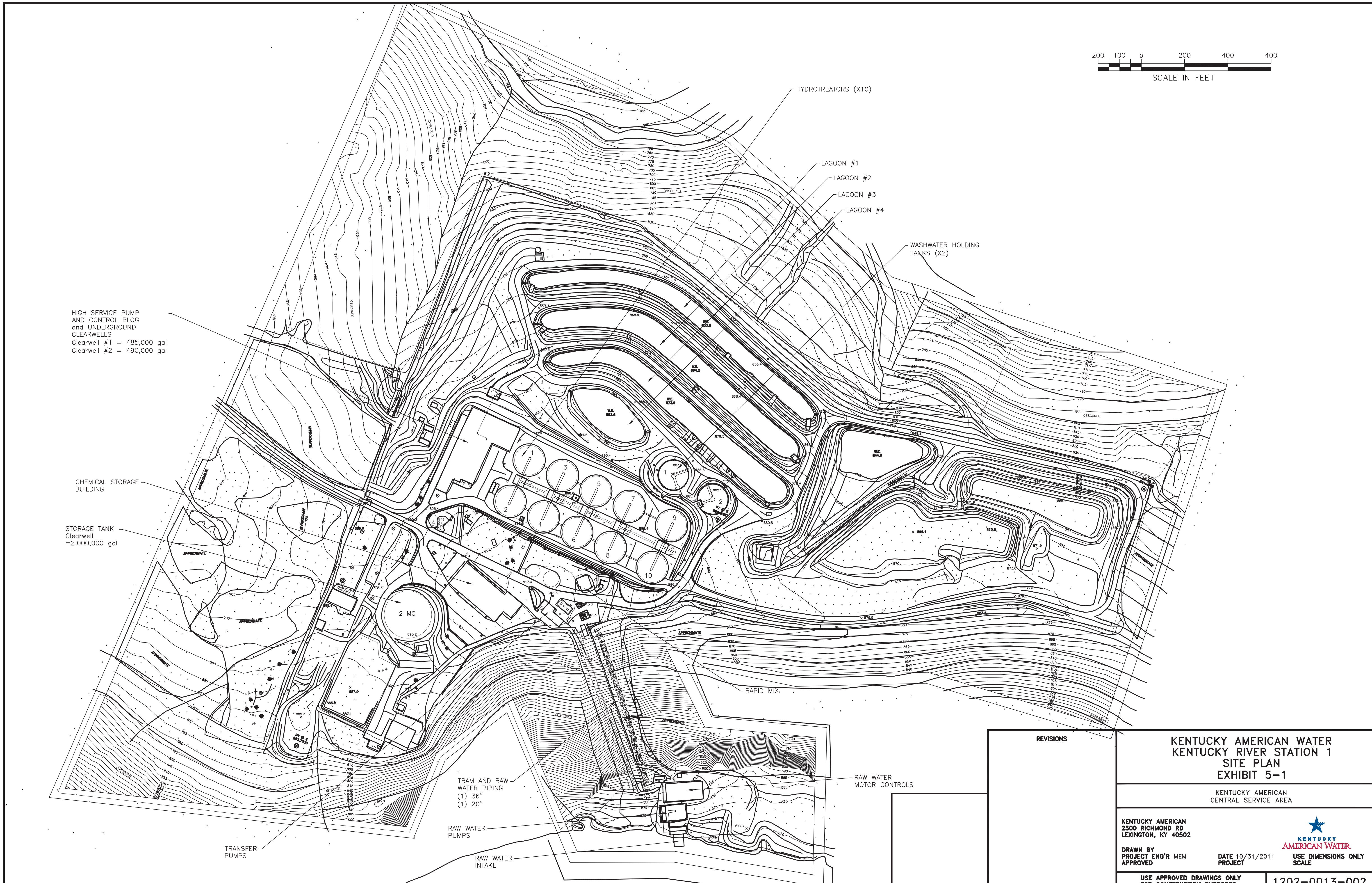
Supplying the Northern Division from the KRS-2 WTP entails constructing approximately 16 miles of 16-inch main along US 127 from the KRS-2 WTP to the intersection of KY 22/US 127 in Owenton and then proceeding east along KY 22 to near the intersection of Old Monterey Road and KY 22. The project phases are described below.

Phase I of the Northern Division Connection project includes the construction of a 16-inch transmission main from the KRS-2 WTP to the north of Monterey. This includes approximately

39,620 linear feet of a 16-inch transmission main and appurtenances. This transmission main will supply flow from KRS-2 to Monterey and enable connections that will allow service to residents who are currently served by the Owenton WTP and that reside south of Monterey along US 127. When all phases of construction are complete, the transmission main's primary purpose will be to supply water to the new 600,000 gallon elevated storage tank that will be constructed outside of Owenton (see Phase III below). Phase 1 of construction includes the decommissioning of the Monterey tank because potable water will be directly supplied by the KRS-2 WTP.

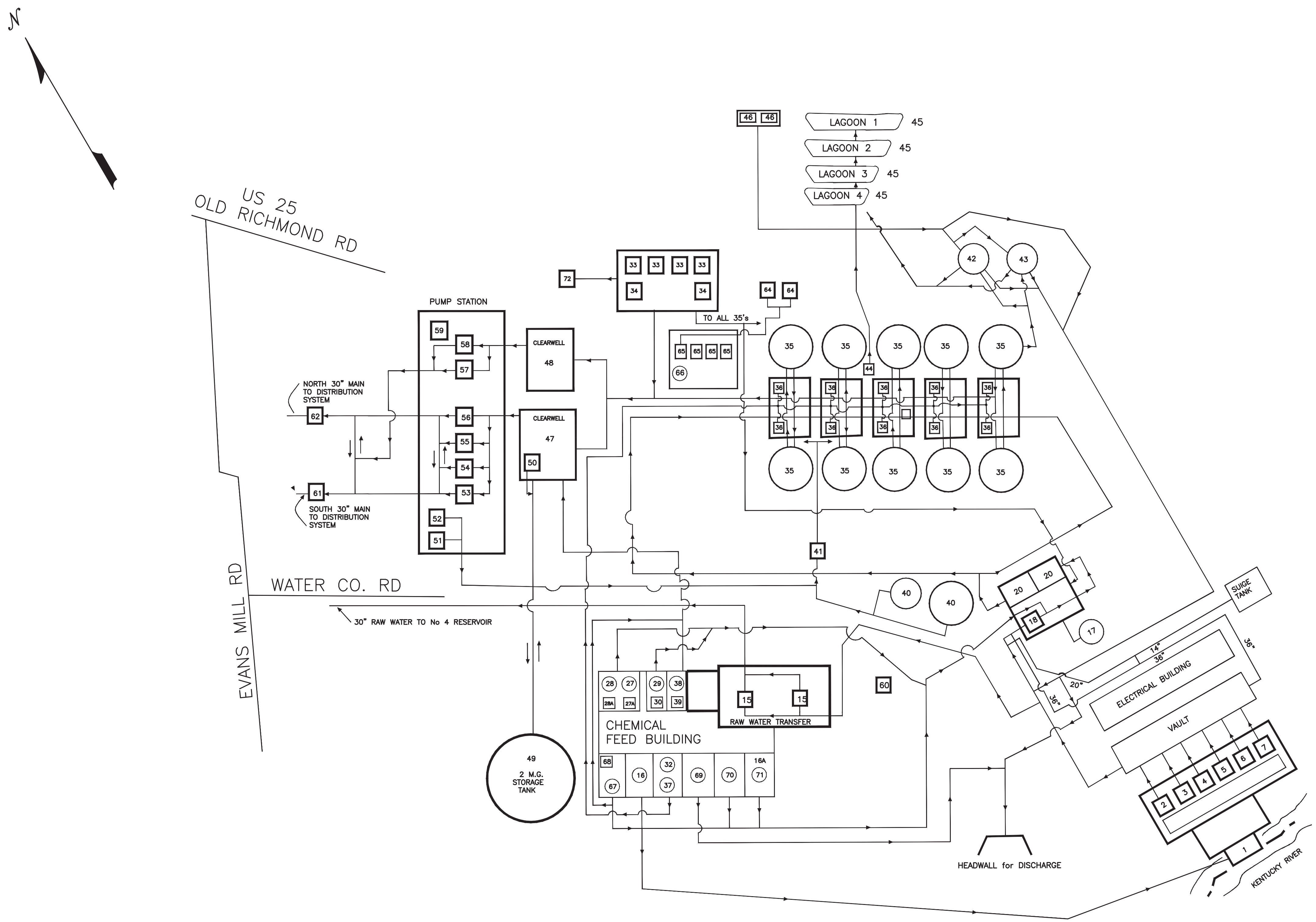
Phase II continues the 16-inch transmission main north along US 127 from Monterey and connects into the Owenton system in three locations: into an existing 6-inch line near the intersection of KY 845 and US 127, into an existing 8-inch line on US 127 near the intersection of US 127 and KY 22, and into an existing 6-inch line on KY 22 near Thomner Trailer Park Road. This includes approximately 44,945 linear feet of a 16-inch transmission main and appurtenances.

Phase III includes the construction of two elevated storage tanks and a booster pump station. The first storage tank will be constructed on the north side of Monterey and will be 300,000 gallons. The second elevated storage tank will be constructed outside of Owenton and will be 600,000 gallons. The new booster pump station will be rated for 2 MGD and will pump directly out of the 300,000 gallon elevated storage tank through the new 16-inch transmission main to Owenton.



FOR COMMENTS

12020013002



NOTE:

1. DRAWING OBTAINED FROM "2010" PLANT DATA REPORT.

REVISIONS

KENTUCKY AMERICAN WATER
KENTUCKY RIVER STATION
SCHEMATIC
EXHIBIT 5-2 (A)

KENTUCKY AMERICAN WATER
CENTRAL SERVICE AREA

AMERICAN WATER ENGINEERING
3906 CHURCH ROAD
MT. LAUREL, NJ 08054

DRAWN BY R. BEATTY
PROJECT ENG'R M. McDONALD DATE 11-01-11
APPROVED PROJECT IP

KENTUCKY AMERICAN WATER

USE DIMENSIONS ONLY
SCALE NONE

USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES

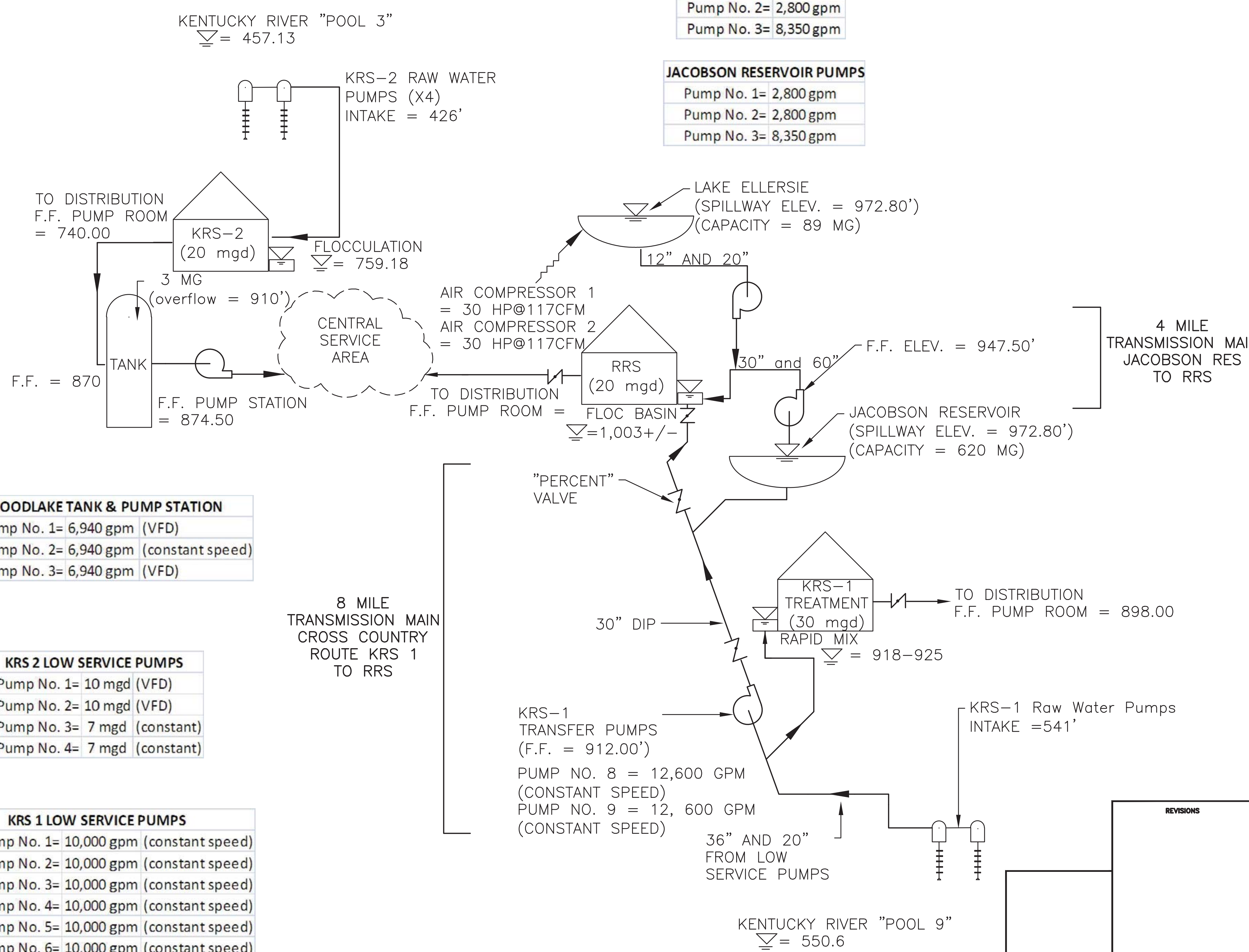
1202-0013-002

FOR COMMENTS

12020013002

KENTUCKY RIVER STATION
FLOW DIAGRAM LEGEND
EXHIBIT 5-2 (B)

1)	Intake	41)	Washwater Vault
2)	Intake Pump # 1	42)	Washwater Holding Tank
3)	Intake Pump # 2	43)	Washwater Holding Tank
4)	Intake Pump # 3	44)	Sludge Well
5)	Intake Pump # 4	45)	Sludge Lagoons
6)	Intake Pump # 5	46)	Return Pump Supernatant
7)	Intake Pump # 6	47)	Clearwell
15)	R/W Transfer Pumps # 8 & 9	48)	Clearwell
16)	Polymer Feed	49)	Clearwell
16A)	Pot. Permang. Feed	50)	Clearwell Transfer Pumps
17)	Lime Feed	51)	Washwater Pump # 1
18)	Raw Water Vault	52)	Washwater Pump # 2
19)	Retired	53)	High Service Pump # 10
20)	Rapid Mix Tanks	54)	High Service Pump # 11
27)	PACL - #2 Tank	55)	High Service Pump # 12
27A)	PACL Pumps	56)	High Service Pump # 13
28)	Ferric Chloride - #1 Tank	57)	High Service Pump # 14
28A)	Ferric Chloride Pumps	58)	High Service Pump # 15
29)	Hydrofluosilicic Acid Tank	59)	Emergency Generator
30)	Hydrofluosilicic Acid Feed System	60)	Emergency Generator
32)	Filter Aid Solution Tank	61)	Meter
33)	Chlorinators	62)	Meter
33A)	Booster Pumps	63)	Meter
34)	Chlorine Evaporators	64)	Ammonia Tanks
34A)	Chlorine Switchover Valves	65)	Ammoniators
35)	Hydrotreators (10)	66)	Water Softener
36)	Filter Aid Pumps	67)	Corrosion Inhibitor Tank
37)	Filter Aid Mixing Tank	68)	Corrosion Inhibitor Pumps
38)	Caustic Tank (New Tank)	69)	Carbon Feed
39)	Caustic Pump (New Pump)	70)	Dechlorination Station
40)	Washwater Tanks		Sodium Thiosulfate
			Day
			Bulk
			Feed Pumps 2 (JAC)
		71)	Settling Aid Polymer
			Polyblend System Pb200-1
			Serial 10941
			Max 1.67 gpm
			Min 0.167 gpm
		72)	Chlorine Scrubber
		73)	Distributive Control System/SCADA



LAKE ELLERSIE PUMPS	
Pump No. 1=	2,800 gpm
Pump No. 2=	2,800 gpm
Pump No. 3=	8,350 gpm

JACOBSON RESERVOIR PUMPS	
Pump No. 1=	2,800 gpm
Pump No. 2=	2,800 gpm
Pump No. 3=	8,350 gpm

RRS HIGH SERVICE PUMPS		
Pump No. 6=	4, 520	(constant speed)
Pump No. 7=	8,333	(constant speed)
Pump No. 8=	2,780	(constant speed)
Pump No. 9=	4,862	(constant speed)
Pump No. 10=	3,850	(constant speed)
Pump No. 11=	2,800	(constant speed)

CLEARWELL MAX = ?
CLEARWELL MIN = ?

KRS 1 HIGH SERVICE PUMPS		
Pump No. 10=	5,560	(constant speed)
Pump No. 11=	5,556	(constant speed)
Pump No. 12=	5,600	(constant speed)
Pump No. 13=	7,000	(constant speed)
Pump No. 14=	7,000	(constant speed)
Pump No. 15=	7,000	(constant speed)

CLEARWELL MAX = 896.50

KRS 2 HIGH SERVICE PUMPS	
Pump No. 1=	6940 gpm (soft start)
Pump No. 2=	6940 gpm (VFD)
Pump No. 3=	4860 gpm (VFD)
Pump No. 4=	4860 gpm (VFD)

CLEARWELL MAX = ?
CLEARWELL MIN = ?

WOODLAKE TANK & PUMP STATION		
Pump No. 1=	6,940 gpm	(VFD)
Pump No. 2=	6,940 gpm	(constant speed)
Pump No. 3=	6,940 gpm	(VFD)

KRS 2 LOW SERVICE PUMPS		
Pump No. 1=	10 mgd	(VFD)
Pump No. 2=	10 mgd	(VFD)
Pump No. 3=	7 mgd	(constant)
Pump No. 4=	7 mgd	(constant)

KRS 1 LOW SERVICE PUMPS		
Pump No. 1=	10,000 gpm	(constant speed)
Pump No. 2=	10,000 gpm	(constant speed)
Pump No. 3=	10,000 gpm	(constant speed)
Pump No. 4=	10,000 gpm	(constant speed)
Pump No. 5=	10,000 gpm	(constant speed)
Pump No. 6=	10,000 gpm	(constant speed)

REVISIONS

KENTUCKY AMERICAN WATER
KENTUCKY RIVER STATION 1
HYDRAULIC SCHEMATIC
EXHIBIT 5–3

KENTUCKY AMERICAN WATER
CENTRAL SERVICE AREA

AMERICAN WATER ENGINEERING
3906 CHURCH ROAD
[illegible] 08054

DRAWN BY R. BEATTY
PROJECT ENG'R M. McDONALD DATE 11–01–11
APPROVED PROJECT IP

AMERICAN WATER

USE DIMENSIONS ONLY
SCALE NONE

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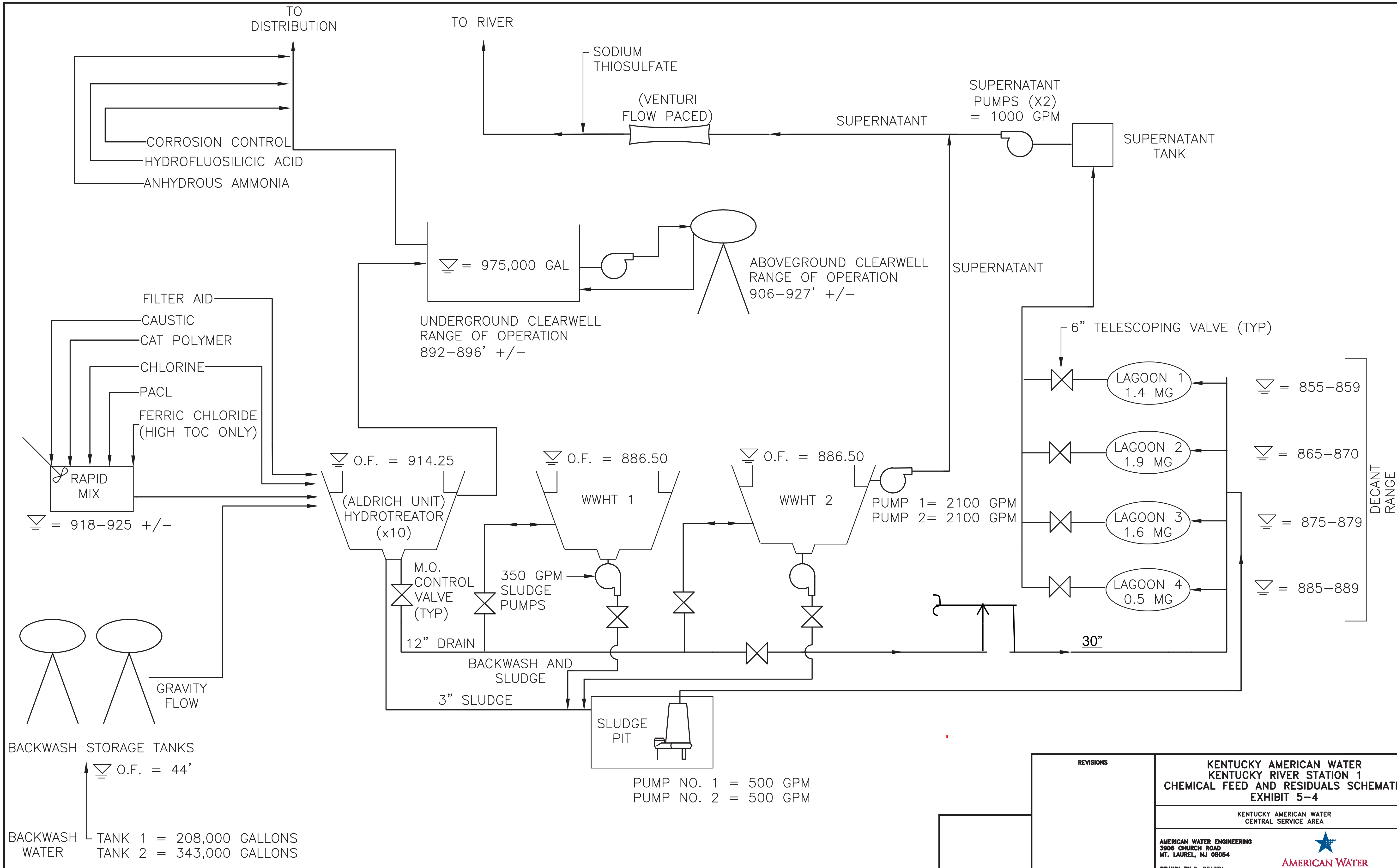
FOR CONSTRUCTION PURPOSES

1202–0013–003

FOR COMMENTS

12020013003



REVISIONS

KENTUCKY AMERICAN WATER
KENTUCKY RIVER STATION 1
CHEMICAL FEED AND RESIDUALS SCHEMATIC
EXHIBIT 5–4

KENTUCKY AMERICAN WATER
CENTRAL SERVICE AREA

AMERICAN WATER ENGINEERING
3906 CHURCH ROAD
MT. LAUREL, NJ 08054

AMERICAN WATER

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PROJECT ENG'R M. McDONALD DATE 11–01–11
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1202–0013–003

FOR COMMENTS

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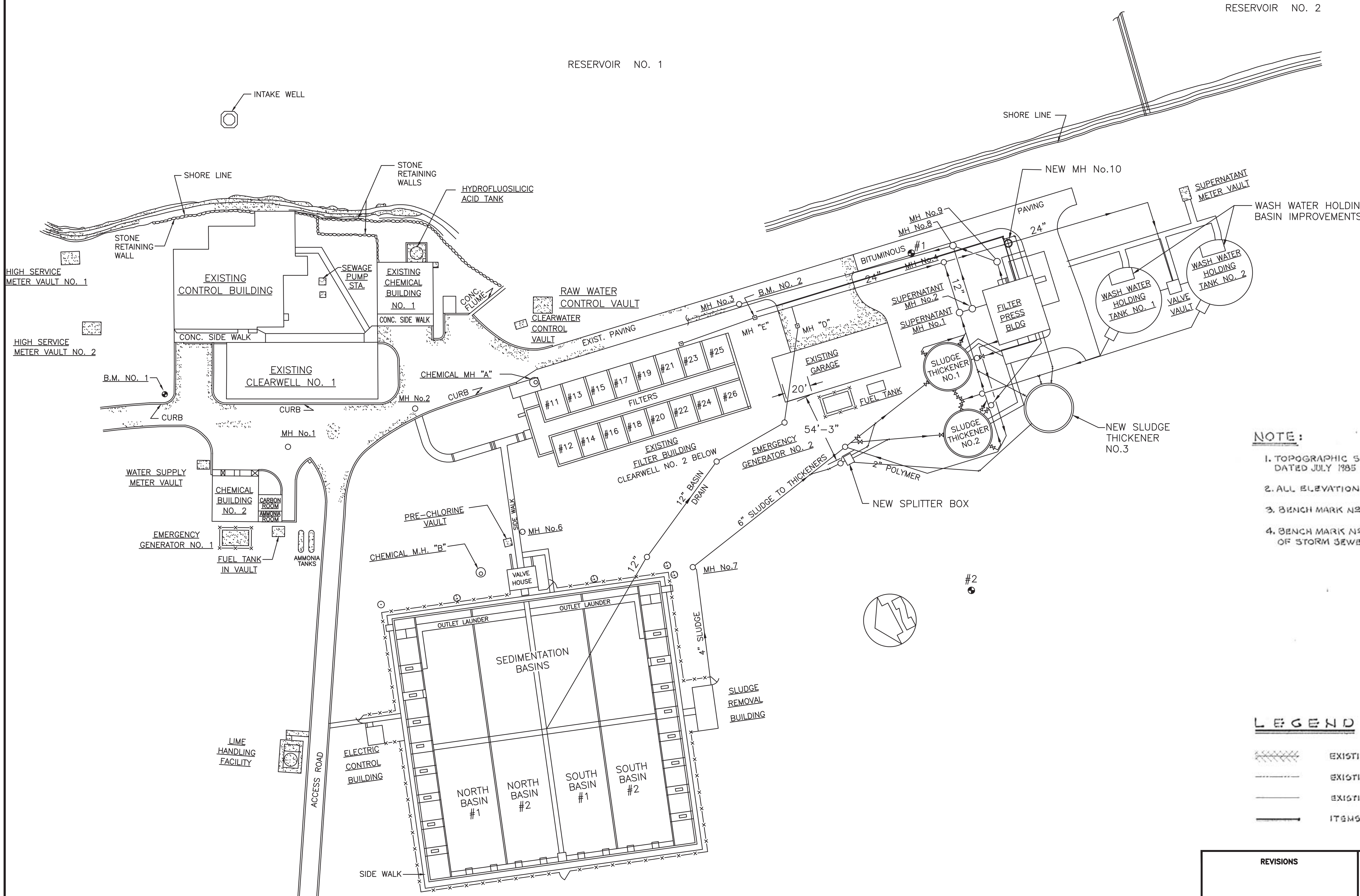

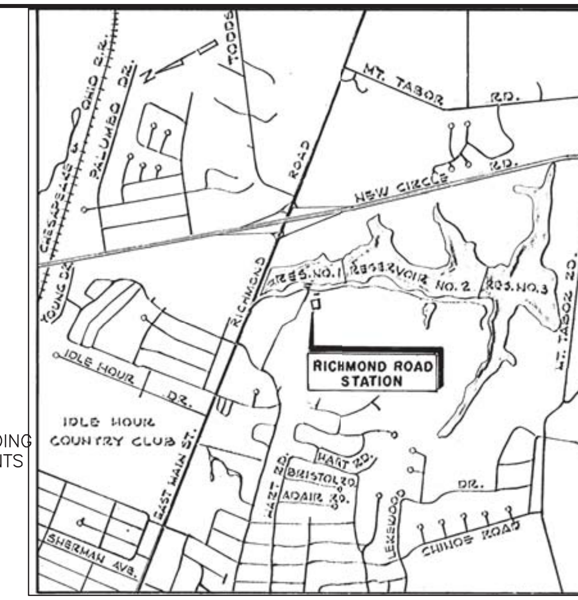

LOCATION PLAN
1" = 2000'

NOTE:
1. TOPOGRAPHIC SURVEY PREPARED BY WILLIAM H. FINNIE & ASSOCIATES DATED JULY 1985 A.W.W. SERV. CO. INC. DWG Nº 330-524.
2. ALL ELEVATIONS SHOWN ARE U.S.G.S.
3. BENCH MARK Nº 1 - DISC SET IN CONC. ELEV. 983.77.
4. BENCH MARK Nº 2 - PAINT MARK TOP OF WALL ON SOUTH EAST CORNER OF STORM SEWER M.H. "E" EL. 984.05.

LEGEND

EXISTING ITEMS & PIPING TO BE REMOVED
EXISTING ITEMS & PIPING TO BE ABANDONED
EXISTING ITEMS & PIPING TO REMAIN IN SERVICE
ITEMS & PIPING TO BE INSTALLED

SITE PLAN
1" = 40'

40 0 20 40 80 120 160
SCALE IN FEET

REVISIONS

KENTUCKY AMERICAN WATER
RICHMOND ROAD STATION
SITE PLAN
EXHIBIT 5-5

KENTUCKY AMERICAN WATER
LEXINGTON WATER SERVICE AREA

AMERICAN WATER ENGINEERING
3906 CHRUCH ROAD
MT. LAUREL, NJ 08054

KENTUCKY AMERICAN WATER

DRAWN BY R. BEATTY
PROJECT ENG'R M. McDONALD DATE 10/17/2011
APPROVED PROJECT IP
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SCALE 1" = 100'

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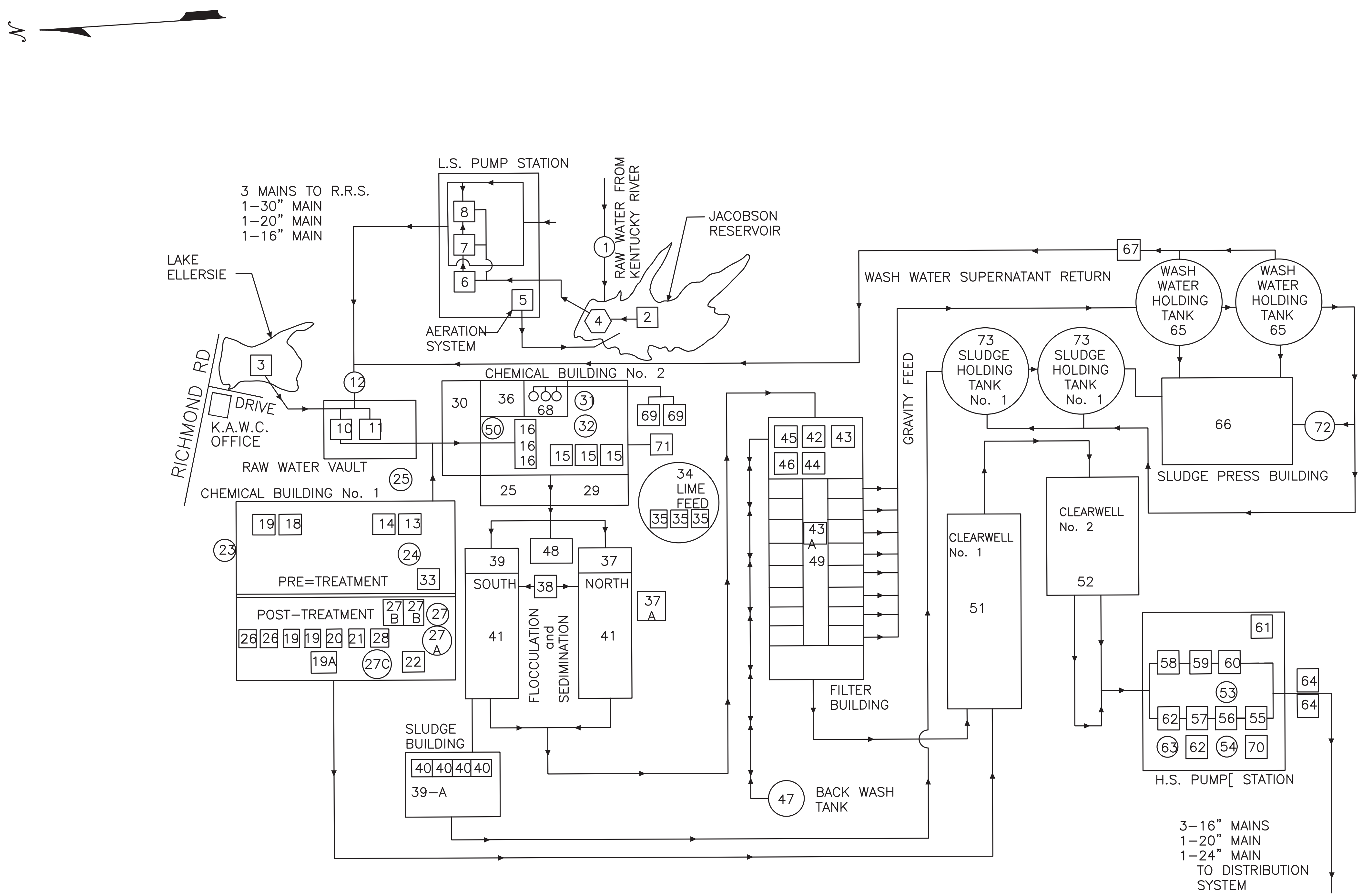
1202-0013-004

FOR COMMENTS

12020013001



RICHMOND ROAD STATION
TREATMENT PLANT

NOTE:

1. DRAWING OBTAINED FROM "2010" PLANT DATA REPORT.

REVISIONS	KENTUCKY AMERICAN WATER CPS RICHMOND ROAD STATION PLANT SCHEMATIC EXHIBIT 5–6 (A)
	KENTUCKY AMERICAN WATER CENTRAL SERVICE AREA
	AMERICAN WATER ENGINEERING 3906 CHURCH ROAD MT. LAUREL, NJ 08054 DRAWN BY R. BEATTY PROJECT ENG'R M. McDONALD DATE 11–01–11 APPROVED PROJECT IP KENTUCKY AMERICAN WATER USE DIMENSIONS ONLY SCALE NONE
	USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES — 1202–0013–005

FOR COMMENTS

12020013005

RICHMOND ROAD STATION
FLOW DIAGRAM LEGEND
EXHIBIT 5-6(B)

1)	Primary Source (Kentucky River)	37)	North Basin Flocculation A. Flocculation/Electric Control Building
2)	Secondary Source (Jacobson)	38)	De-Icer
3)	Emergency Source (Ellerslie)	39)	South Basin Flocculation A. Flocculation/Electric Control Building
4)	Potassium Permanganate Feed Intake Well	40)	Sludge Removal Pumps A. MRI Sludge Collectors/Controllers
5)	Aeration System	41)	Sedimentation Basins (4)
6)	Low Service Pump # 3 Jacobson Reservoir	42)	Filter Air Feed System
7)	Low Service Pump #2 Jacobson Reservoir	43)	Filter Applied Analyzer
8)	Low Service Pump #1 Jacobson Reservoir	43A)	Post Chlorine Analyzer
10)	Low Service Pump #5 Lake Ellerslie	44)	Air Wash Compressor
11)	Low Service Pump #4 Lake Ellerslie	45)	Washwater Pump
12)	Raw Water Vault – N. Basin	46)	Washwater Meter
12a)	Raw Water Vault – S. Basin		
13)	Raw Water Turbidimeter	47)	Washwater Tank
14)	Raw Water Manganese Analyzer & Recorder	48)	North & South Coagulant Control
15)	PACL Feed Pumps	48A)	Pre-Chlorine Analyzer
16)	Polymer Feed Pumps	49)	Filters
18)	Pre- or Post-Chlorination	51)	Clearwell #1
19)	Pre- or Post-Chlorination	52)	Clearwell #2
19a)	Chlorine Evaporator		
19b)	Chlorine header connection	53)	Filtered pH Analyzer
20)	Booster Pumps for Chlorine Feed	54)	Effluent pH Analyzer
23)	Chlorine Scrubber	55)	High Service Pump #8
24)	Post Chlorine Analyzer	56)	High Service Pump #7
25)	Fluoride Tanks & Pumps	57)	High Service Pump #6
26)	Caustic Feed Pumps	58)	High Service Pump #11 Diesel
27)	Caustic Shortage Tank	59)	High Service Pump#10 Electric or Diesel
A.	Caustic Day Tank	60)	High Service Pump #9 Diesel
B.	Caustic Transfer Pumps		
C.	Caustic Bulk Storage Tank	61)	Emergency Generator # 1 Pump Station/440
28)	Post Chlorination	62)	Priming Pump
29)	Corrosion Inhibitor Tanks & Pumps	63)	Post-Chlorinator Analyzer
30)	PACL Feed System	64)	Finished Water Venturis # 1 & # 2
31)	PACL Storage Tank	65)	Washwater Tanks
32)	PACL Day Tank	66)	Sludge Press
33)	Chlorine Detector	67)	Supernatant Flow Control
34)	Lime Feed System (Out of service)	68)	Ammoniators
35)	Peristaltic Lime Feed Pumps	69)	Ammonia Storage Tanks
36)	Carbon Feed System	70)	Distributive Control System/SCADA
		71)	Emergency Generator # 2 / Chemical Feed
		72)	Emergency Generator # 3 / Filter Bldg
		73)	Sludge Holding Tanks
		74)	Phosphate Analyzer
		75)	Chlorine Analyzer
		76)	Mobile Emergency Generator # 4 / Pump Station

SITE PLAN

1" = 100'

100 0 50 100 200 300 400

SCALE IN FEET

REVISIONS

KENTUCKY AMERICAN WATER
KENTUCKY RIVER STATION 2
SITE PLAN
EXHIBIT 5-7

KENTUCKY AMERICAN WATER
LEXINGTON WATER SERVICE AREA

AMERICAN WATER ENGINEERING
3906 CHURCH ROAD
MT. LAUREL, NJ 08054

DRAWN BY R. BEATTY
PROJECT ENG'R M. McDONALD DATE 11-01-11
APPROVED PROJECT

KENTUCKY AMERICAN WATER

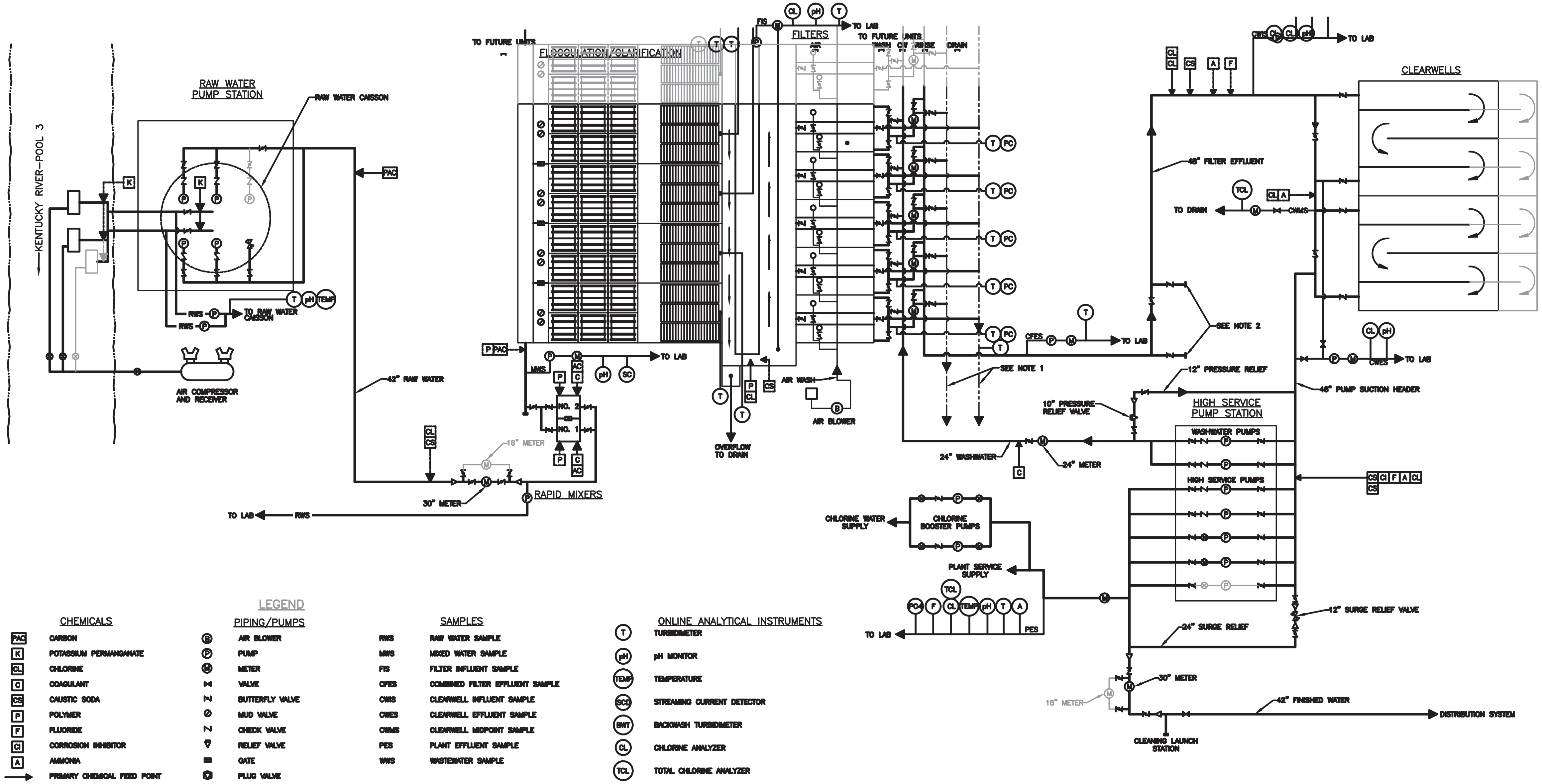
USE DIMENSIONS ONLY
SCALE 1" = 100'

USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES

1202-0013-006

FOR COMMENTS

12020013006



LEGEND

CHEMICALS

PAC	CARBON
K	POTASSIUM PERMANGANATE
CL	CHLORINE
C	COAGULANT
CS	CAUSTIC SODA
P	POLYMER
F	FLUORIDE
CI	CORROSION INHIBITOR
A	AMMONIA
→	PRIMARY CHEMICAL FEED POINT
→	SECONDARY CHEMICAL FEED POINT
AC	ALTERNATE COAGULANT

PIPING/PUMPS

B	AIR BLOWER
P	PUMP
M	METER
	VALVE
	BUTTERFLY VALVE
	MUD VALVE
	CHECK VALVE
	RELIEF VALVE
	GATE
	PLUG VALVE
	BLIND FLANGE
	SURGE ANTICIPATOR VALVE
	PROPOSED 20 MGD
	REQUIRED FOR 25 MGD
	REDUCER
	BALL VALVE

SAMPLES

RWS	RAW WATER SAMPLE
MWS	MIXED WATER SAMPLE
FIS	FILTER INFLUENT SAMPLE
CFES	COMBINED FILTER EFFLUENT SAMPLE
CWIS	CLEARWELL INFLUENT SAMPLE
CWES	CLEARWELL EFFLUENT SAMPLE
CWMS	CLEARWELL MIDPOINT SAMPLE
PES	PLANT EFFLUENT SAMPLE
WWS	WASTEWATER SAMPLE

ONLINE ANALYTICAL INSTRUMENTS

T	TURBIDIMETER
pH	pH MONITOR
TEMP	TEMPERATURE
SCD	STREAMING CURRENT DETECTOR
BWT	BACKWASH TURBIDIMETER
CL	CHLORINE ANALYZER
TCL	TOTAL CHLORINE ANALYZER
F	FLUORIDE ANALYZER
A	AMMONIA ANALYZER
PC	PARTICLE COUNTER
PO4	PHOSPHATE ANALYZER

NOTES:

1. SEE WASTEWATER FLOW DIAGRAM.
2. PROVISIONAL CONNECTIONS FOR UV FACILITIES.

WATER TREATMENT PLANT
PROCESS
PROCESS FLOW DIAGRAM

FILE: D:\PC435\47225\KAW\WTP\PROCESS\12020001P02.DWG
DATE: 07/30/2007 TIME: 9:40:02 am

REVISIONS

KENTUCKY AMERICAN WATER
KENTUCKY RIVER STATION 2
PROCESS FLOW DIAGRAM
EXHIBIT 5-8

KENTUCKY AMERICAN WATER
CENTRAL SERVICE AREA

AMERICAN WATER ENGINEERING
2000 CHURCH ROAD
MT. LAUREL, NJ 08054

KENTUCKY AMERICAN WATER

DRAWN BY R. BEATTY
PROJECT ENG'R M. McDONALD DATE 11-01-11
APPROVED PROJECT IP

USE DIMENSIONS ONLY
SCALE NONE

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1202-0013-002

FOR COMMENTS

12020013002

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SECTION 6
DISTRIBUTION AND STORAGE

6.1 OVERVIEW

This section addresses the condition of Kentucky American Water distribution facilities, including pipelines, storage tanks, and booster stations. The ability of these facilities to provide safe, adequate, and reliable service to customers served by the KAW system was analyzed based on forecasted customer demands guidelines from historical and projected growth patterns developed internally and based on discussions with local planning agencies. Due to the number of tables, they are presented at the end of this section.

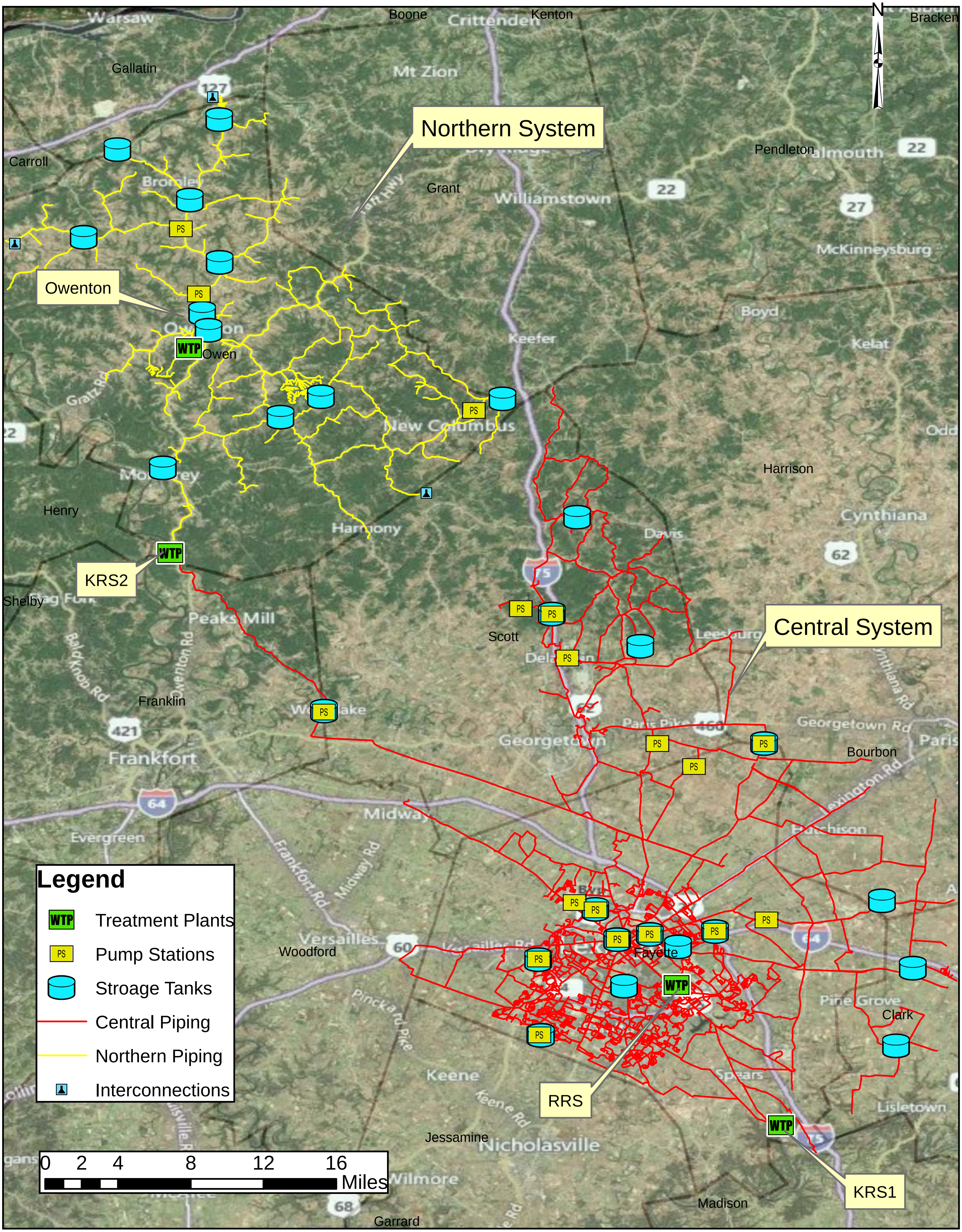
A general layout of the distribution system is shown on **Exhibit 6-1**. Recommended improvement projects are described in **Section 6.6**.

6.2 HYDRAULIC MODELING

A computer model of the distribution system was developed by Strand Associates, Inc. for the March 2012 *Hydraulic Analysis for Comprehensive Planning Study* report (Strand Report) utilizing the KYPIPE computer and a copy is included as **Appendix E**. Data relating to pipe diameter, length, material, age and connection points were obtained from distribution maps and records. Hazen-Williams friction coefficients were calculated for selected pipelines utilizing flow test data or standard graphs based on the age, material and size of pipeline. The results were used to develop friction coefficients for other pipelines of similar diameter, age and material.

The Central model is a skeletonized model of the Fayette County area system including facilities as far north as the Sadieville tank in Scott County with a portion of the smaller mains not included. Base demands were distributed based on demand patterns that were developed from SCADA recorded data. Peaking factors for the minimum and average demand scenarios were developed based on SCADA data from the plant production facilities and tank levels to represent peak hour demands. Peaking factors for maximum demand scenarios were developed from previous system modeling work performed in 2007 and scaled to produce the appropriate peak hour demands.

Exhibit 6-1 - KAW
Distribution System Layout



The Northern model includes the entire distribution system in Owen County and surrounding areas including those areas that are currently served by the Owenton Water Treatment Plant as well as those areas served from bulk purchases of adjoining water utilities. Hydrant flow test data, SCADA information, and customer and meter route usage data for the month of May 2011 were utilized to calibrate the model. Peaking factors were developed using SCADA data and field data provided by KAW.

Projected minimum, average, and maximum day demands were provided by KAW for the Central and Northern Systems. A minimum demand day scenario was utilized as a basis for modeling improvements to water age, and the target year maximum day demand is used as a basis in modeling hydraulic improvements. Distribution system improvement projects for KAW were determined by computer simulations of water system hydraulics and analysis of the computer model results under various present and future demand scenarios. After baseline model scenarios were completed with extended period simulations (EPS) runs, the results were analyzed to identify potential improvements. This analysis identified areas of concern related to tank turnover, adequate system storage and pressure, and water quality related to water age. These findings along with potential solutions were summarized in the Strand Report. In addition, the Strand Report also recommended operational changes to turning over the tanks.

American Water Corporate Engineering further analyzed the areas of concern from the Strand Report and, along with KAW Staff and Strand, identified and prioritized recommendations to address system deficiencies. These recommendations are provided in this section and **Section 1**.

6.3 DESCRIPTION OF EXISTING FACILITIES

The distribution system for Kentucky American Water's service area consists of approximately 2,118 miles of main, ranging in size from 2 inches to 42 inches in diameter. Pipe materials include cast iron, asbestos cement, plastic, ductile iron, and steel. Much of the pipe in the KAW system is over 50 years old, although newer mains have been installed in several areas of the system, primarily as replacement projects for undersized pipelines. There are over 8,570 fire hydrants in the KAW service area.

The distribution system is divided into two major systems: the Central System (Lexington and the surrounding contiguous area also is known as the combination of the Main, Sadieville and

High Service Zones in previous reports) and the Northern System (areas in Gallatin, Grant and Owen Counties). The Central System (shown in **Exhibit 6-2**) is predominately one pressure gradient with two minor pressure gradients in Scott County. The Northern System (shown in **Exhibit 6-3**) is divided into 9 gradients as shown in **Table 6-1**. There are various booster stations and storage tanks that deliver water to each of the pressure gradients in the Systems. The characteristics of the pumps serving each zone are listed in **Table 6-2**, and the characteristics of the storage tanks serving each zone are listed in **Table 6-3**.

<u>Central System</u> - The Central System consists of 15 pump stations, including the high service pumps at the three treatment facilities, and 18 treated water storage tanks (4 standpipes, 8 ground tanks, and 6 elevated tanks) with a total storage capacity of approximately 25.80 MG.

In addition, there are approximately 5.62 million gallons of clearwell storage at the treatment plants. The majority of KAW facilities are concentrated within Fayette County centering on the Lexington-Fayette Urban Service planning area which covers the entire county. Except for the long 24-inch main into Scott County, the balance of the larger transmission and distribution mains supply water to the Central System. Pressures in the Central System are maintained by the distribution pumps at the (KRS-1, KRS-2 and at the Richmond Road Station (RRS)). KRS-1 has a nominal treatment capacity of 40 MGD and a temporary capacity rating of 45 MGD during the summer months (2011 KAW Plant Data Report). RRS has a nominal treatment capacity rating of 25 MGD and a temporary capacity rating of 30 MGD during the summer months (2011 KAW Plant Data Report). KRS-2 has a nominal treatment capacity of 20 MGD. Water pumped from these three treatment plants is supplemented by withdrawals of stored water from the eighteen (18) storage tanks as needed, on peak hour demand periods or to provide water for firefighting purposes.

Growth in most of the Central System is controlled by the Lexington-Fayette Urban County Planning Commission which has channeled development into concentrated sectors of expansion within the Urban Service Area. The areas planned for most of the new growth are north and east of the city between New Circle Road and the I-64 and I-75 highway corridors and southeast of New Circle Road and south of Winchester Road. Most of the new water works facilities to be installed in Fayette County will be in support of the anticipated Urban Service Area growth.

Exhibit 6-2 - KAW
Central System Layout

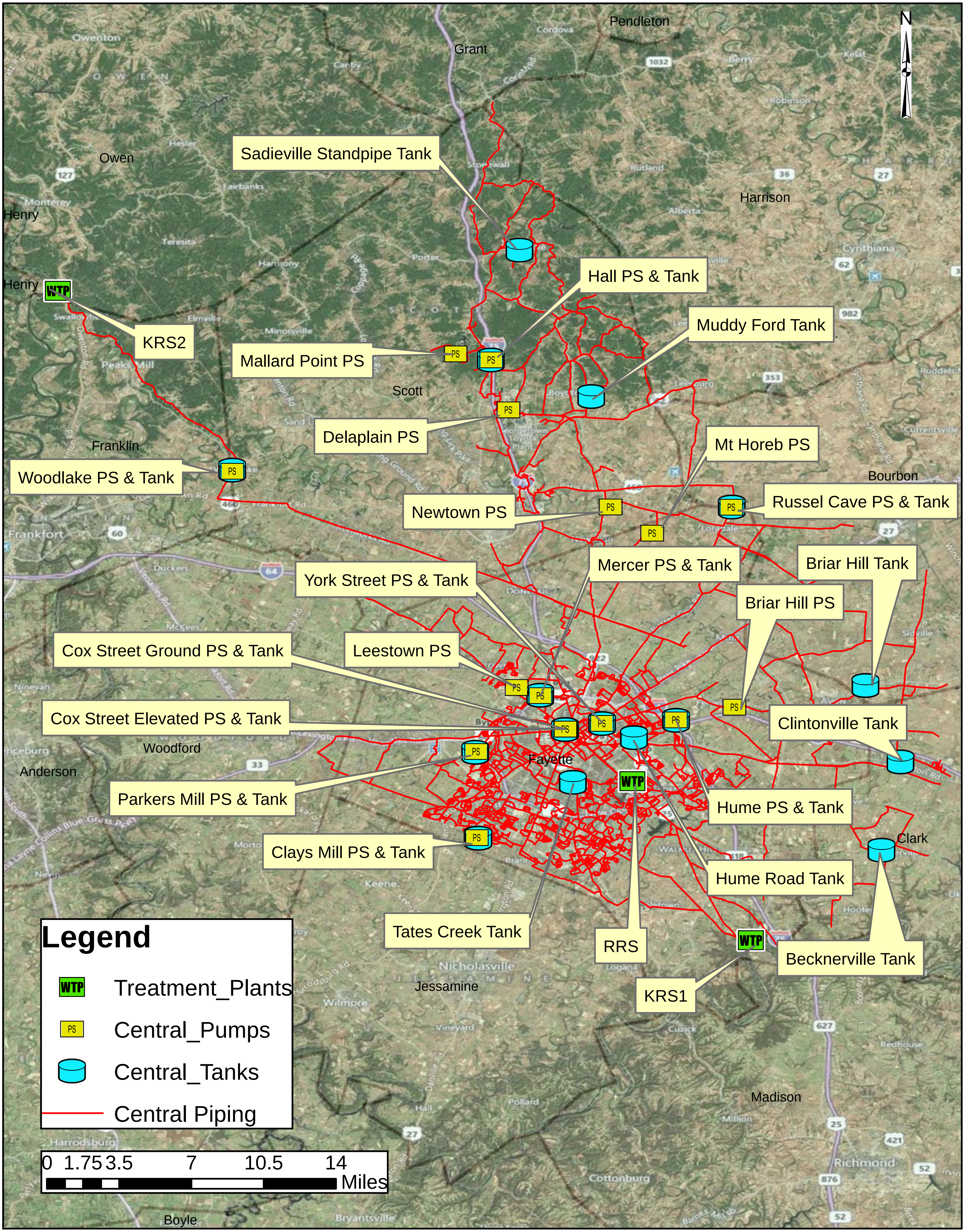
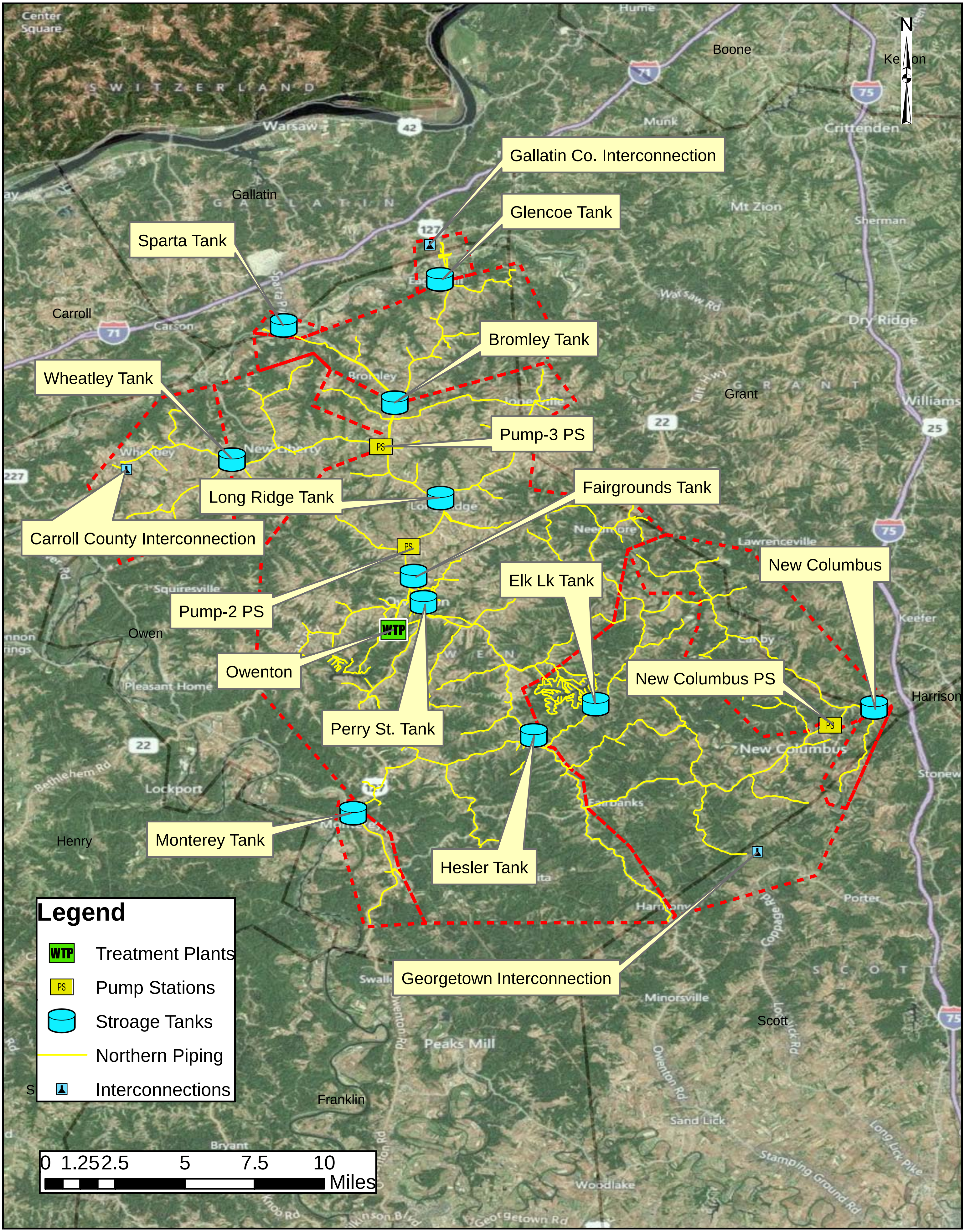


Exhibit 6-3 - KAW
Northern Distribution System Layout



Growth in the Central System north of Fayette County is expected to be moderate during the next few years. Scott County is anticipated to grow more than Harrison and Bourbon Counties according to the population forecasts. Toyota Motor Manufacturing is the largest single customer for KAW. Apart from Toyota, the growth will be mostly residential in nature, with a few, new light industrial and commercial customers being added paralleling Toyota growth. Additionally, growth is expected around the intersection of US-460 and I-75. Under these assumptions, the existing 24-inch transmission main will be adequate throughout the planning period. The planned new mains will be mostly to provide reliability and to stabilize pressures in the areas north of Fayette County rather than to transmit large quantities of water to new demand regions.

Due to the location of the storage tanks in the system with respect to the customer elevation changes, there are 11 storage tanks that require pumping facilities to make full use of the tank volume that are classified as pumped storage. These tanks are Clays Mills 1, Clays Mills 2, Parkers Mill, York Street, Mercer Road, Hume Road, Cox Street-Elevated, Cox Street-Ground, Russell Cave, Woodlake, and Hall. The use of the pumping stations allow for controlled utilization of storage for meeting peak hourly demands and enable full use of tank volume for fireflow and emergencies while also creating turnover of water in the tanks. Providing adequate turnover prevents long detention times which can cause nitrification and taste and order issues, so the tank volumes are turned over every three days. KAW has difficulty turning the water over in several tanks in the Central System. These tanks at times experience nitrification and taste and odor issues, and include the Briar Hill Tank, Sadieville, Russell Cave and sometimes Muddy Ford.

In addition to the counties served by KAW, there are interconnections with neighboring water systems that purchase water from KAW. These systems are North Middletown, Georgetown, Nicholasville, Versailles, Cynthiana, Jessamine, South Elkhorn, Harrison County Water Association, and Midway water systems.

Northern System - The Northern System currently consists of 4 pump stations, including the high service pumps at the Owenton WTP, and 11 treated water storage tanks (9 standpipes and 2 elevated tanks) with a total storage capacity of approximately 1.74 MG.

KAW is in the process of implementing a major capital improvement project to change the source of water being provided to the Northern System. When construction is completed, the Northern System's source of supply will change from the existing Owenton WTP to the KRS-2 WTP on the Franklin/Owenton County border. In addition, two storage tanks and a booster station will be provided. A schematic of the Northern System that includes these capital improvements is shown in **Exhibit 6-4**. The continued discussion assumes these major capital improvements are completed.

The Northern System customers are served by KAW's KRS-2 WTP. KAW also purchases treated water from Georgetown Municipal Water and Sewer Services (Georgetown), Gallatin County Water District (Gallatin), and Carroll County Water District (Carroll) for small areas of the system that are not connected to the distribution system.

The new 0.6 MG storage tank is the largest tank in the Northern System and is located within the Owenton/Rockdale/New Columbus (Owenton) pressure zone. KRS-2 is the primary source for this zone, with water also being purchased from Georgetown. KRS-2 provides water through a 16-inch transmission main to the new 0.3 MG storage tank, located just north of the Monterey North pressure zone. The new booster station (rated capacity of 2.0 MG) provides the water from the 0.3 MG storage tank to the Owenton pressure zone. The New Columbus tank has the same overflow of 1119 feet as the 0.6 MG storage tank. The Fairgrounds tank operates within the same pressure zone as the new 0.6 MG storage tank, but its overflow is 19 feet lower than the new tank. To help the Fairground tank turnover, an automatic flow control valve opens when water is needed to replenish the Fairgrounds tank and closes to allow tank turnover. Similar to the Fairgrounds tank, the Perry Street tank is equipped with a control valve that opens/closes based on the Fairground tank level. The Hesler, Elk Lake, and Long Ridge tanks are also located in the zone, but are to limited in effectiveness with providing service due to water quality issues resulting from difficulty with tank volume turnover. These tanks are periodically removed from service to react to water quality issues on temporary basis. The New Columbus booster station is an in-line booster station that maintains pressures in the system.

In addition to the Owenton pressure zone being served by the KRS-2 WTP, the Monterey North 1 and 2, Monterey South 1 and 2, Bromley, and Sparta pressures zones are served directly by the KRS-2 WTP. The Monterey North and South pressure zones connect to the 16-inch transmission main from KRS-2 and are provided storage from the 0.3 MG storage tank. Various

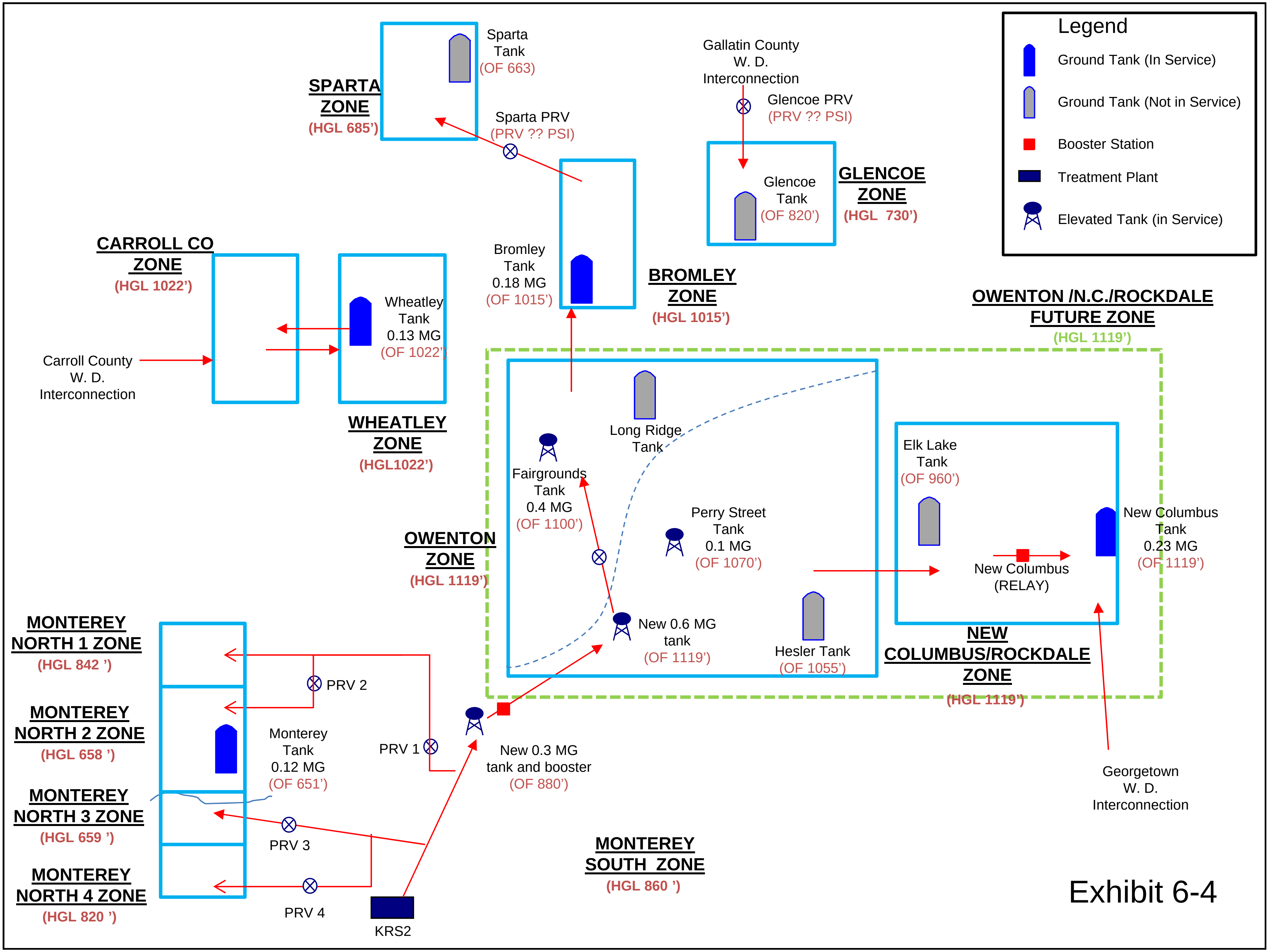

Legend
Ground Tank (In Service)
Ground Tank (Not in Service)
Booster Station
Treatment Plant
Elevated Tank (in Service)
SPARTA ZONE
(HGL 685')
Sparta Tank
(OF 663)
Sparta PRV
(PRV ?? PSI)
Gallatin County W. D. Interconnection
Glencoe PRV
(PRV ?? PSI)
Glencoe Tank
(OF 820')
GLENCOE ZONE
(HGL 730')
CARROLL CO ZONE
(HGL 1022')
Carroll County W. D. Interconnection
Wheatley Tank
0.13 MG
(OF 1022')
WHEATLEY ZONE
(HGL1022')
Bromley Tank
0.18 MG
(OF 1015')
BROMLEY ZONE
(HGL 1015')
OWENTON /N.C./ROCKDALE FUTURE ZONE
(HGL 1119')
Long Ridge Tank
Fairgrounds Tank
0.4 MG
(OF 1100')
Perry Street Tank
0.1 MG
(OF 1070')
New 0.6 MG tank
(OF 1119')
Hesler Tank
(OF 1055')
OWENTON ZONE
(HGL 1119')
Elk Lake Tank
(OF 960')
New Columbus Tank
0.23 MG
(OF 1119')
New Columbus (RELAY)
NEW COLUMBUS/ROCKDALE ZONE
(HGL 1119')
Georgetown W. D. Interconnection
MONTEREY NORTH 1 ZONE
(HGL 842 ')
MONTEREY NORTH 2 ZONE
(HGL 658 ')
MONTEREY NORTH 3 ZONE
(HGL 659 ')
MONTEREY NORTH 4 ZONE
(HGL 820 ')
PRV 2
PRV 1
PRV 3
PRV 4
Monterey Tank
0.12 MG
(OF 651')
New 0.3 MG tank and booster
(OF 880')
KRS2
MONTEREY SOUTH ZONE
(HGL 860 ')

Exhibit 6-4

pressure reducing valves (PRVs) draw supply from the 16-inch transmission main and maintain adequate pressures to the Monterey North and South pressure zones. The Monterey storage tank serves only the Monterey North 2 pressure zone. No storage facilities are located within the other three Monterey pressure zones, however, storage for the entire Monterey pressure zone is provided by the new 0.3 MG storage tank.

The Bromely and Sparta pressure zones are north of Owenton and storage is provided to both zones by the Bromley tank, along with any excess storage available in the Owenton pressure zone. The Sparta pressure zone is equipped with a standpipe (Sparta tank), but also with the Hesler tank, it is limited in effectiveness in providing service and experiences water quality issues as a result from difficulty with tank turnover. The Bromely tank also has a separate fill and drain line. The tank is filled by water from the Fairgrounds tank, but customers downstream of the Bromley tank on the discharge side only see the Hydraulic Grade Line (HGL) of the Bromley tank. The fill line has an altitude valve that opens and closes based on the water level in the Bromley tank to allow adequate tank turn over.

The Wheatley pressure zone is currently not connected to the Owenton distribution system. It is a standalone system that is serviced by the Carroll, but is owned and maintained by KAW. Storage is provided by the Wheatley tank. Like the Wheatley pressure zone, the Glencoe pressure zone is also not connected to the Owenton system, but is owned and maintained by KAW. The source of supply for the Glencoe pressure zone is provided entirely by Gallatin. The storage for this pressure zone is provided by Gallatin. The Glencoe pressure zone is equipped with a standpipe (Glencoe tank), it is limited in effectiveness in providing service and experiences water quality issues as a result from difficulty with tank turnover.

6.4 DISTRIBUTION SYSTEM ANALYSES

Distribution system analyses include evaluations of: 1) pressures and flows in the KAW pipe network, 2) system pumping capacity, 3) distribution storage adequacy, and 4) emergency supply availability. The calibrated hydraulic model of the Central and Northern distribution system was used to perform system investigations.

As shown on **Table 6-1**, the pressure gradients in the KAW service areas include zones served solely by KAW facilities (Central Zone, Owenton, Monterey, Bromley, and Sparta), zones served by other water systems (Carroll Co./Wheatley and Glencoe), and zones served by a combination of the two (Rockdale/New Columbus). A summary of booster pumping equipment adequacy is provided in **Table 6-4**, and system storage adequacy is presented in **Table 6-5**.

The analyses of each gradient considers supply, pumping, storage, and emergency power capabilities. Based on the gradient configuration, supply sources and distribution pumps should be capable of meeting maximum day and peak hour customer demands, respectively. System storage is evaluated based on the volumes of effective storage and fire reserves. Effective storage is defined as that volume of water available while maintaining a minimum of 30 psi pressure to all customers under normal operation conditions and 20 psi under fire flow conditions. Equalization storage volumes were calculated from SCADA data and demand records from a maximum demand day of 72 mgd that was also used in the hydraulic model.

A storage analysis was performed to ensure there is adequate emergency supply provided to meet the Public Service Commission (PSC) requirements and American Water standards. More detail of this requirement is provided under **Section 6.4.4**.

6.4.1 Distribution Pipe Network

The KAW hydraulic model was used to analyze the system pipe network under current and future maximum day and average day conditions. These analyses were performed to determine if system pressures fall within the range specified by KAW. System mains also were evaluated based on velocity and head loss criteria to determine if inadequately sized pipes or old pipes in poor condition limit system flow capacity.

Marginal pressures, in the range of 30 psi under future maximum day demands, were identified primarily in the Briar Hill area and some southeast areas in the Central System under future max day conditions. Marginal pressures were identified in the northern most part of the Central System, between the Fairgrounds tank and Bromley tank during current and future max day conditions. Pressures were below 30 psi at many areas in the Northern System during future max day conditions.

Pressures in these locations are primarily influenced by piping, except for the southeast portion of the Central System, which is due to elevation. The pressures are below the 45 psi that KAW desires to provide in the southeast area, but still acceptable.

6.4.2 Pumping Analyses

Table 6-4 provides a summary of the reliable pumping capacities in the Central System and Northern System for meeting the current and projected maximum day demands for each pressure zone. Reliable capacity is the amount of water that can be delivered with the largest or most critical pumping unit out of service. Actual distributive pumping capacities are used for the analysis, rather than the given design capacities.

The Central System receives its source of supply from KRS-1, KRS-2, and RRS. KRS-1 and RRS high service pumps deliver water directly into the Central System. KRS-2 pumps into the Woodlake storage tank and the Woodlake booster station delivers the water to the Central System. As shown in **Table 6-4**, the Central System has adequate reliable pumping capacity available for the entire planning period.

The Northern System receives its source of supply from KRS-2 and various interconnections. KRS-2 pumps directly to the new 0.3 MG storage tank via a 16-inch transmission main. The Monterey North and South zones receive water directly from KRS-2 through connections off the 16-inch transmission main. As shown, Monterey North and South zones have adequate reliable pumping capacity available for the entire planning period.

The new 2.0 MG booster station delivers the water from the 0.3 MG storage tank to the Owenton pressure zone, with Bromley and Sparta pressure zones also receiving water from the booster station via the Owenton pressure zone. The Owenton pressure zone can also receive water through an interconnection with Georgetown. The rated capacity of the Georgetown interconnection is estimated based on the size of the interconnection (8-inch). As shown, there is adequate reliable capacity to serve the Owenton, Bromley, and Sparta pressure zones throughout the planning period. It is recommended that KAW perform a flow test on the Georgetown interconnection to determine the actual rated capacity and ensure the interconnection can adequately meet the projected future demands.

The Carroll County/Wheatley and Glencoe pressure zones receive water directly through interconnections with Carroll and Gallatin, respectively. Just like the Georgetown interconnection, the rated capacity of the Carroll and Gallatin interconnections is based on the size of the interconnection (8-inch connection with Carroll and 6-inch connection with Gallatin).

As shown, there is adequate reliable capacity to serve the Carroll County/Wheatley and Glencoes pressure zones throughout the planning period.

It is recommended that KAW perform a flow test on the Carroll and Gallatin interconnections to determine the actual rated capacities and ensure the interconnections can adequately meet the current and projected future demands.

6.4.3 Storage Assessment

The storage evaluation is summarized in **Tables 6-5.** Each pressure zone should have sufficient effective storage to satisfy equalization volume and fire flow reserve needs. Effective storage volume is the storage volume available while maintaining a minimum of 30 psi throughout the pressure zone, during normal operating conditions. Based on system analysis, required equalization storage volume is assumed to be 15% of the maximum day demand. The fire flow reserve need is based on providing up to a maximum of 3,500 gpm for up to three hours. This represents a needed fire reserve volume of 0.63 MG. **Table 6-5** indicates that the Central System has adequate total and effective storage to meet system needs throughout the planning period

All of the pressure zones in the Northern System have adequate total and effective storage except the Glencoe pressure zone. Storage for the Glencoe pressure is provided through the interconnection with Gallatin.

As aging assets reach the end of their useful lives, KAW should evaluate the feasibility of rehabilitating or retiring those assets to optimize system operations while continuing to provide adequate storage in the Central and Northern Systems.

6.4.4 Emergency Supply Analyses

The Public Service Commission (PSC) Title 807, Chapter 5 – Utilities, Section 4 – Continuity of Service, paragraph (4) states “the minimum storage capacity for systems shall be equal to the average daily consumption.” Section 4 is entitled “Continuity of Service” and generally deals with provisions to provide continuous supply to customers during various emergency situations. The ”emergency” storage is generally required so that an adequate supply of water is available in the event of a scenario where water cannot be distributed from the system’s source and treatment facilities. Reasons for not being able to supply water to the system could include an emergency in the source of supply (such as a spill), a power failure, or an upset or other treatment problem.

In 1993, the PSC granted KAW a variance from this regulation and reduced the storage requirement to 50% of an average day demand by giving credit for standby-powered distributive pumping facilities at the treatment plants. Consequently, the Central and Northern systems have been evaluated based on this criteria. **Tables 6-6** and **Table 6-7** provide a summary of the emergency supply analyses as per PSC criteria in the Central and Northern Systems.

The Central System has an emergency generator located at KRS-2 and RRS WTPs. The generator at the KRS-2 WTP can operate a 10 mgd raw water pump, 10 mgd high service pump, and associated plant equipment. It should be noted that it is assumed only 8 mgd will be available to the Central System from KRS-2 WTP since it also provides supply to the Northern System. RRS is equipped with three (3) generators. They are utilized to operate the lights, chemical feed systems, the sedimentation basin mixing and flocculation equipment, and limited pumping ability in the from Jacobson Reservoir and Lake Ellerslie. The RRS WTP's distributive pumps 9, 10, and 11 are equipped with direct diesel drive engines. The combination of this standby-powered equipment can provide up to 16.5 mgd of supply from RRS.

Similar to RRS, KRS-1 distributive pump 15 is also equipped with a direct diesel drive engine. However, KRS-1 does not have any emergency generator or direct diesel drive engines on the raw water source. Therefore, it is assumed that there is no available standby-powered supply capability at the KRS-1 WTP.

There are also emergency generators provided at Woodlake booster station, Parkers Mill booster station, Newtown booster station, and Russell Cave booster station. The generators at each booster station have the capability of operating one pump. Clays Mill and Hume Road Tanks each have diesel drive pumps of 6.0 mgd and 9.0 mgd respectfully. All the booster stations that are equipped with generators or diesel drive pumps are utilized for pumped storage, except for the Newtown booster station, which is a transfer station.

The Northern System can receive 2 mgd from KRS-2, as stated earlier. There is also an emergency generator provided at the new Northern booster station that can operate two pumps for a total capacity of 2 mgd. Each pressure zone has at least one storage tank that can be utilized in an emergency.

As can be seen in **Table 6-8**, the Central System provides adequate total storage capacity and standby source capacity to meet the required 50% of the average day demand in storage capacity and 50% of the average day demand in standby distributive pumping facilities. As can be seen in **Table 6-9**, the pressure zones served by KRS-2 provide adequate storage capacity in each pressure zone to meet the required 50% of the average day demand in storage capacity and 50% of the average day demand in standby distributive pumping facilities. In addition, the Carroll County/Wheatley and Glencoe pressure zones provide 100% of the average day demand in storage.

6.4.5 American Water Power Outage Analysis

The ability to provide water during a loss of power was evaluated by zone based on average daily demands. Each pressure zone is recommended to provide a full day supply of water through storage or emergency powered pumping equipment during a power outage. The available storage during a power outage assumes the equalization volume needed for each zone is not available. **Table 6-10** summarizes the emergency power analysis and shows that the combination of storage and auxiliary powered supplies provide nearly a full average day demand for the Central and the Carroll Co./Wheatley pressure zones, while exceeding the goal of providing a full average day demand for Owenton/Rockdale/New Columbus, Monterey North/South, and Bromley/Sparta pressure zones.

In the Northern System, the Carroll Co./Wheatley and Glencoe pressure zone receives water from Carroll and Gallatin. At this time the available emergency powered pumping available through those interconnections is not known. It is recommended KAW evaluate the adequacy of the emergency power capacity available from these interconnections to determine if any additional assets (storage tanks or emergency powered booster stations) are needed.

6.5 IDENTIFIED AREAS OF INTEREST

KAW staff worked closely with the hydraulic modeling effort in helping to identify areas of concern in the distribution system. The issues typically fell into either the category of poor water quality, due to elevated water age, or low pressures during high demand.

Central System - Water quality issues were identified by operations at several locations throughout the Central System under current conditions. The two primary areas of concern are the Briar Hill area and the Sadieville tank area, which both exhibited nitrification. Modeling corroborated the water quality issues via model runs that indicated high water age at these locations during current day conditions and will continue throughout future day conditions as well.

Nitrification can be observed in systems that utilize chloramines as a form of disinfection. When a water system has high water age and minimal tank turnover, the ammonia (from chloramines) can potentially oxidize with oxygen and form nitrite. To prevent nitrification, systems typically perform routine flushing and/or cleaning (draining and disinfecting) of finished water storage tanks. Project recommendations are provided for both of these areas and for the distribution system in general as a means of addressing chlorine demand and nitrification.

The other area that was identified, primarily through modeling analysis, is the development of low pressures in the system under future maximum day conditions. Additional demands put on the system under future conditions resulted in pressures that were at or below 30 psi, the KDOW regulatory threshold.

Finally, there are several areas in the Central System that will be impacted by a projected Kentucky Department of Transportation project to expand US-25, in conjunction with future population increases in that area. System pressures in these areas are shown to be marginal under future peak conditions and are prudent to replace with larger pipe at marginal cost.

Northern System - Distribution issues in the Northern System center around existing main sizes and anticipated increases in once maximum day demand increases over time.

Once the KRS-2 connection is made, piping improvements are needed in the vicinity of the Fairgrounds tank in order to allow for proper tank operation due to an anticipated increase in the

HGL. The improvements will facilitate the adequate filling of the Fairgrounds tank under the new HGL conditions.

6.5.1 CENTRAL SYSTEM

Areas of interest that were identified by KAW staff for the Central System are discussed below. A detailed discussion can be found in the hydraulic modeling report, included as **Appendix E.** Note that not all of the areas of interest that are identified below were addressed with projects.

The following three areas were identified by KAW personnel based on everyday operation and validated through modeling efforts:

1. <u>Russell Cave tank area</u> – When Russell Cave tank is in full operation, KAW indicated high pressures were experienced near the tank and north along Russell Cave Road when the Russell Cave pump station was operating. Pressures north along Russell Cave Road when the Russell Cave PS was operating were high enough to cause water main breaks. KAW also indicated low pressures (at or below 30 psi) were experienced near the intersection of Greenwich Pike and Hume Bedford Pike southeast of the Russell Cave tank when the Russell Cave tank fills.

2. <u>Southeast of Eastland tank</u> – KAW indicated low pressures are experienced in the area roughly bounded by Todds Road, New Circle Road, Winchester Road, and Man O War Boulevard that is immediately southeast of the Eastland tank.

3. <u>Parkers Mill and Clays Mill tank area</u> – KAW indicated high pressures are experienced in the system when the Parkers Mill PS and Clays Mill PS are operating at the same time.

The following eleven areas were also identified during the modeling effort as shown on **Exhibit 6-5**.

1. <u>Briar Hill tank area</u> – The Briar Hill tank service area was identified by KAW staff and through modeling as having slightly elevated water age compared to other portions of the system. KAW staff and modeling results also identified portions of the service area experiencing low pressures below the optimal KAW target of 45 psi. In addition, KAW operations staff indicated they have trouble turning over the tank.

2. Hume Bedford Pike and Greenwich Pike area – Modeling identified this area as having low pressures.

3. Georgetown Bypass and US 25 area – Future demand is anticipated to increase in this area. Modeling of future demands indicated existing infrastructure was not sufficient enough to support predicted demand and maintain optimum system pressure (above 45 psi). KAW identified opportunities to increase the size of US 25 main south of Ironworks Road through highway improvements. KAW also identified improvements that could be made along Lisle Road as a means of providing redundancy to the Newtown Pike transmission main.

4. Muddy Ford tank area – Modeling identified areas around and north of the Muddy Ford tank with low pressures and elevated water age compared to other portions of the system.

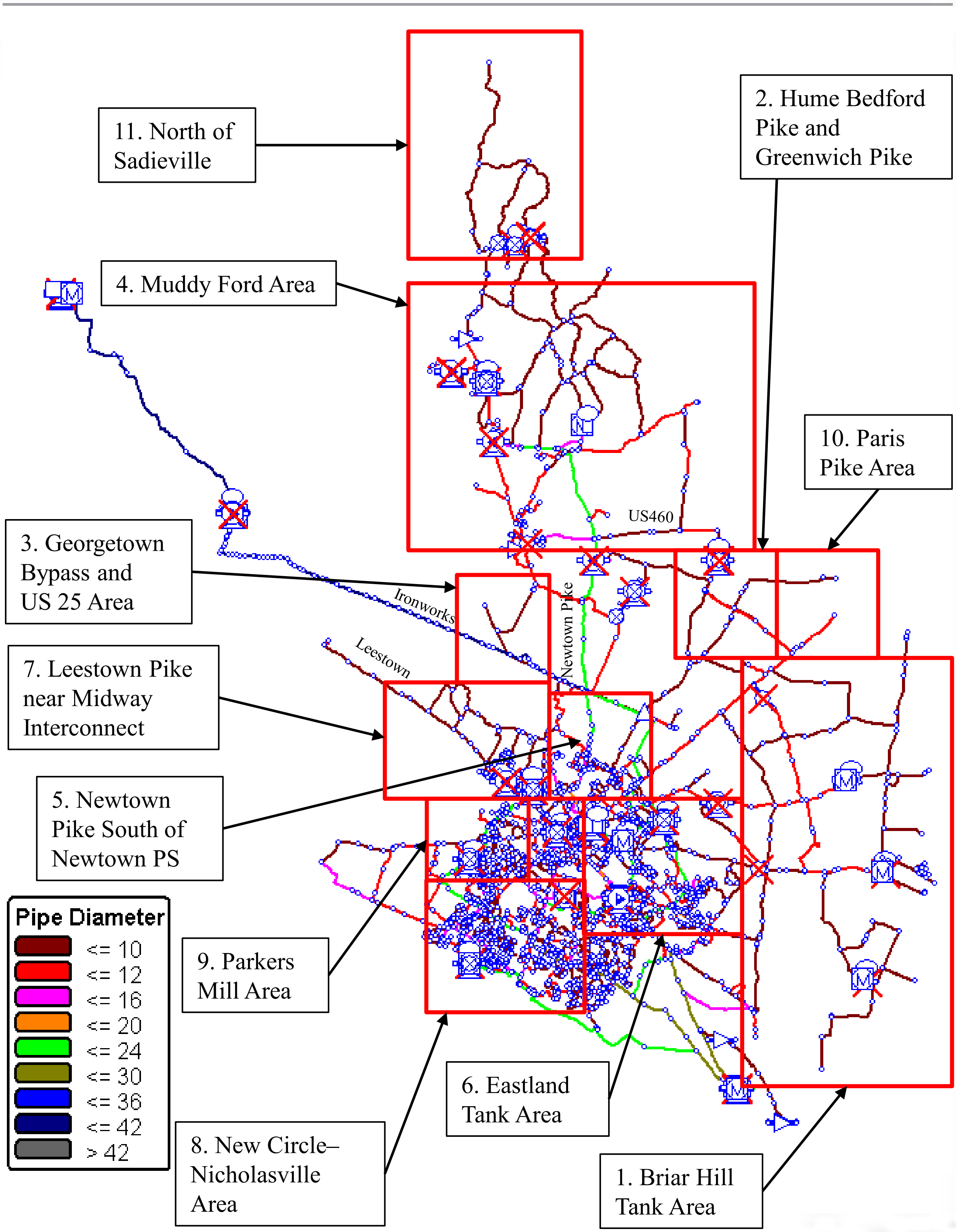
5. Newtown Pike South of Newtown Pump Station – The 24-inch main that runs south from Ironworks along Newtown Pike is one of the primarily avenues for flow from KRS-2 to reach the central portion of the system. The 24-inch main along Newtown Pike reduces to 16 inches between I-64 and New Circle Road, creating a potential bottleneck for flow from KRS-2 to reach the central portion of the system.

6. Eastland tank area – Areas southeast of the Eastland tank in the central portion of the system were identified by KAW staff and through modeling as areas experiencing low pressures. In addition, KAW identified the 6- and 8-inch main between Floyd Drive and Eastland Drive along New Circle Road as a problem. Access to the main is challenging because of bury depth, there have been several main breaks, and it is perceived to be a potential bottleneck.

7. Leestown Pike near Midway Interconnect – The Central System currently provides water for the City of Midway. KAW identified the 8-inch main serving Midway as a potential bottleneck during high demand conditions.

8. New Circle-Nicholasville area – Several high elevation areas along Nicholasville Road heading south out of downtown Lexington were identified as having pressures below the optimal KAAW target of 45 psi through modeling. In addition, modeling identified several areas with

Exhibit 6-5
Central Division Areas of Interest Identified by KAW Staff



AREAS OF INTEREST FOR
CAPITAL IMPROVEMENT PROJECTS
HYDRAULIC ANALYSIS FOR
COMPREHENSIVE PLANNING STUDY
KENTUCKY AMERICAN WATER
LEXINGTON, KENTUCKY

STRAND ASSOCIATES

elevated water ages compared to the rest of the system west of Nicholasville Road near New Circle Road and Man O War Boulevard.

9. Parkers Mill Tank area – Modeling identified areas near the Parkers Mill tank with elevated water age compared to other portions of the system.

10. Paris Pike – Modeling identified areas of elevated water age compared to other portions of the system at the end of the Paris Pike main on the outskirts of the Central System.

11. North of Sadieville – Modeling identified areas north of Sadieville at the edge of the Central System with elevated water age compared to other portions of the system.

6.5.2 NORTHERN SYSTEM

Areas of interest that were identified by KAW staff for the Northern System are shown on **Exhibit 6-6** and discussed below. A detailed discussion can be found in the hydraulic modeling report, included as **Appendix E.**

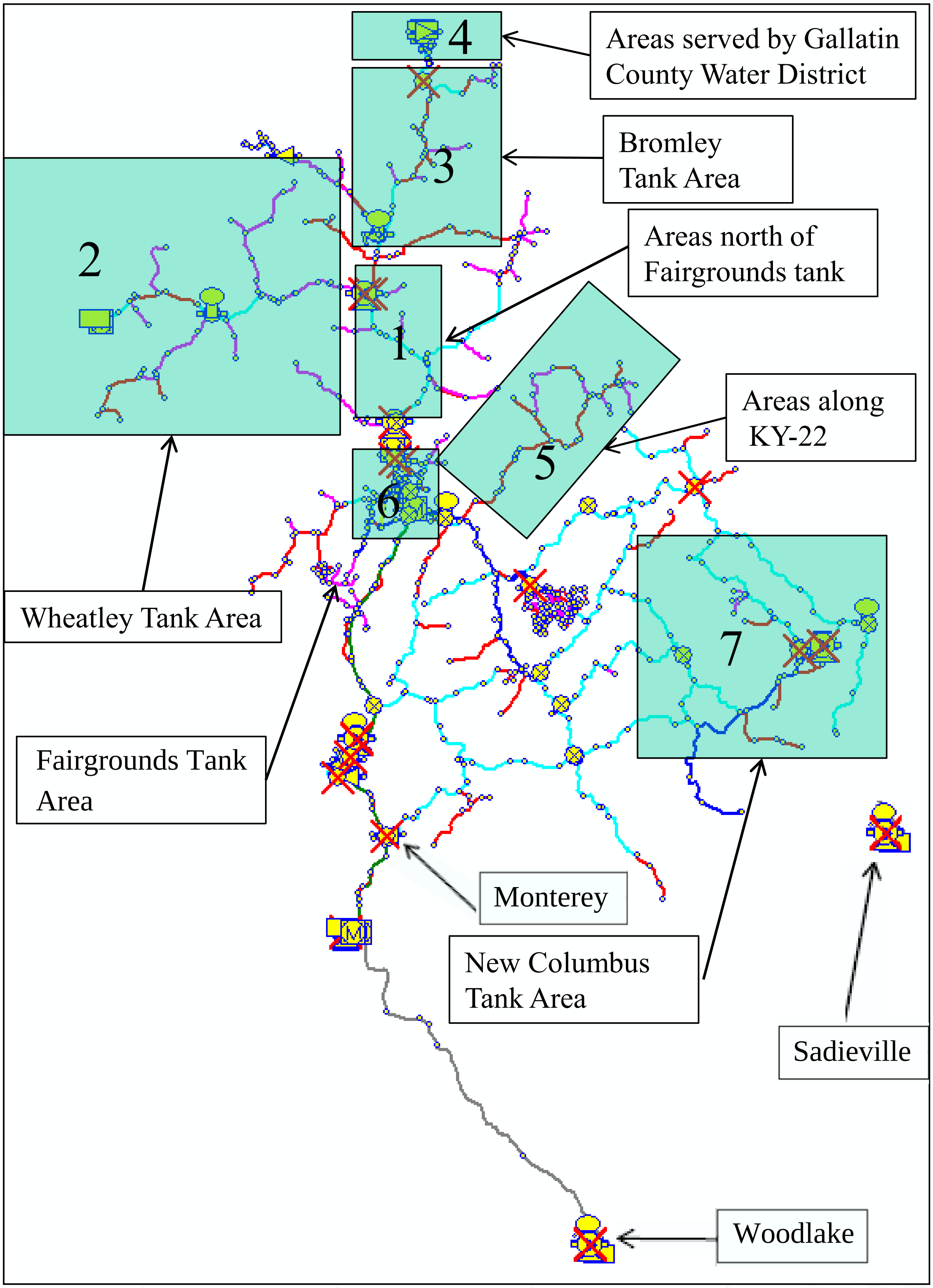
1. Areas North of Fairgrounds tank – Hydraulic modeling of future demands indicated existing infrastructure was not sufficient to support predicted demand and supply water to areas currently served by Carroll and Gallatin while maintaining marginal pressures or pressures below KAW minimum desired pressure of 45 psi.

2. Areas Currently Served by Carroll (Wheatley tank area) – KAW is considering relying less on purchased water from Carroll and serving this area with water pumped from KRS-2. This area is currently in a different pressure zone and modification will be needed to incorporate it into the Fairgrounds pressure zone.

3. Bromley tank area – Modeling identified this area as having low pressures.

4. Areas currently served by Gallatin – KAW is considering relying less on purchased water from Gallatin and serving this area with water pumped from KRS-2. This area is currently in a different pressure zone and modification will be needed to incorporate it to the Northern System pressure zone.

Exhibit 6-6
Northern Division Areas of Interest Identified by KAW Staff



NORTHERN DIVISION IDENTIFIED AREAS OF INTEREST
HYDRAULIC ANALYSIS FOR COMPREHENSIVE PLANNING STUDY
KENTUCKY AMERICAN WATER
LEXINGTON, KENTUCKY

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5. Areas along KY-22 – Modeling identified areas around and north of the New Owenton tank with low pressures.

6. Fairgrounds tank area – Baseline modeling indicated that piping improvements are required to adequately turn the tank volume over during low demand and fill the Fairgrounds tank with the projected increased demands on it.

7. New Columbus tank area – Modeling indicated elevated water age compared to other portions of the system for the New Columbus Tank and areas supplied by this tank. This is because this standpipe currently serves a very small demand. KAW operations staff also indicated they have difficulty turning over the water in this tank. In addition, KAW is considering relying less on purchased water from Georgetown and serve the New Columbus tank area with water pumped from KRS-2.

6.6 RECOMMENDED PROJECTS

The weaknesses remaining in the KAW system under present and projected demand conditions are primarily associated with current and projected water age/water quality in the Central System and projected low pressures in the Northern System.

6.6.1 Central System

Of the eleven areas of interest identified above, Area 1, 3, 5, 7, and 11 are addressed with specific project recommendations in **Section 1**. Area 2, 4, 6, 8, 9, and 10 will require additional analysis through modeling efforts.

6.6.2 Northern System

Of the seven areas of interest identified above, Areas 5 and 6 are addressed with specific project recommendation in **Section 1**. Area 1 and 3 will be further analyzed through modeling, along with the entire Northern System, to identify any low or marginal pressure issues based on the Northern System being served by KRS-2. No project will be recommended to be address Area 2, 4, and 7 since the capital cost to serve this small number of customers outweighs the cost to purchase the water. As growth continues in the Northern System's pressure zones (specifically areas served by Carroll, Gallatin, and Georgetown), and along the pipeline to serve these areas, KAW should reanalyze the feasibility of relying less on purchased water and serving these areas with water pumped from KRS-2.

Table 6-1
Pressure Zone Summary

	Pressure Zone	Customer Elevation Range & Pressure Range	Supply	Storage Facilities in Pressure Zone	Approx. HGL (feet)	Boosters & PRVs from Pressure Zone
Central System	Central	750 feet to 1070 feet 49 psi to 187 psi	KRS1 KRS2 RRS	Clay Mills 1 Clay Mills 2 Parkers Mill York Street Mercer Road Hume Road Cox Street Cox Street Russell Cave Woodlake Eastland Tates Creek Briar Hill Hall	1185	Cox Street (Elevated) Cox Street (Ground) Delaplain Mercer Road York Street Parkers Mills Hume Road Hall Tank Newtown Mt Horeb Clays Mills Briar Hill Woodlake Mallard Point Russel Cave Leestown
				Muddy Ford	1130	
				Sadieville	992	
Northern System	Owenton/Rockdale/ New Columbus	640 feet to 1000 feet 51 psi to 206 psi	KRS2	Hesler Perry Street Fairground Long Ridge	1119	N/A
	Monterey North 1	500 feet to 700 feet 61 psi to 147 psi	KRS2	N/A	842	Monterey PRV 1
	Monterey North 2	482 feet to 520 feet 59 psi to 76 psi	KRS2	Monterey	658	Monterey PRV 2
	Monterey South 1	480 feet to 520 feet 60 psi to 77 psi	KRS2	N/A	659	Monterey PRV 3
	Monterey South 2	500 feet to 700 feet 52 psi to 138 psi	KRS2	N/A	820	Monterey PRV 4
	Carroll Co./Wheatley	700 feet to 910 feet 48 psi to 138 psi	Carroll Co. Interconnection	Wheatley	1022	N/A
	Bromley	500 feet to 910 feet 45 psi to 221 psi	KRS2	Bromley	1015	N/A
	Sparta	490 feet to 540 feet 62 psi to 84 psi	KRS2 (via Bromley Zone)	Sparta	685	Sparta PRV (from Bromley Zone)
	Glencoe	540 feet to 580 feet 65 psi to 82 psi	Gallatin Co Interconnection	Glencoe	730	Glencoe PRV (from Gallatin Co. Inter.)

Note: Table assumes Northern Improvements complete to allow KRS2 to serve Northern System

Table 6-2 Pump Data										
	Pressure Zone	Station	Purpose of Station	Station Elevation (feet)	Pump No.	Year Installed	Rated Motor HP	Drive Type	Rated Flow (mgd)	Rated TDH (feet)
Central System	Central	KRS1	WTP High Service Pumps	901	10	1988	700		8.00	380
					11	1998	700		8.00	380
					12	1966	700		8.50	380
					13	1991	800		10.00	380
					14	1970	800		10.00	380
					15	1981	800		10.00	380
		KRS2[1]	WTP High Service Pumps	746	1	2010	700	VFD	10.00	324
					2	2010	700	VFD	10.00	324
					3	2010	500	Constant Speed	7.00	324
					4	2010	500	Constant Speed	7.00	324
		RRS[2]	WTP High Service Pumps	984	6	1988	250		6.50	190
					7	1988	500		10.00	240
					8	1988	200		4.00	240
					9	1993	400		7.00	235
					10	1988	580		5.50	220
					11	1965	200		4.00	220
		Cox Street (Elevated)	Pumped Storage (from tank)	949	1		40		3.00	60
		Cox Street (Ground)	Pumped Storage (from tank)	964	1		100		2.50	188
		Delaplain	In-line Booster	893	1		40		0.86	
		Mercer Road	Pumped Storage (from tank)	975	1		75		5.00	70
		York Street	Pumped Storage (from tank)	963	1		100		2.50	188
		Parkers Mill[3]	Pumped Storage (from tank)	988	1		200		9.00	100
		Hume Road	Pumped Storage (from tank)	946	1		300		6.00	220
					2		150		3.00	210
					3		150		3.00	210
		Hall Tank	Pumped Storage (from tank)	1015	1		10		0.58	35
					2		10		0.58	35
		Newtown	In-line Booster	883	1		15		2.00	
					2		50		4.00	
					3		50		4.00	
		Mt. Horeb	In-line Booster	905	1		25		1.15	
					2		25		1.15	
		Clays Mill[3]	Pumped Storage (from tank)	988	1		500		9.00	230
					2		500		9.00	230
		Briar Hill	In-line Booster	980	1		125	VFD	1.94	216
					2		125	VFD	1.94	216
		Woodlake[3]	Pumped Storage (from tank)	874	1				10.00	
					2				10.00	
					3				10.00	
		Mallard Point	In-line Booster	988	1				0.14	
					2				0.14	
		Russel Cave[3]	Pumped Storage (from tank)	980	1				1.00	
					2				3.00	
					3				3.00	
		Leestown	In-line Booster		1		20		1.15	
					2		20		1.15	
Northern System	Owenton/Rockdale/ New Columbus	KRS2[1]	WTP High Service Pumps	874	1	2010	700	VFD	10.00	324
					2	2010	700	VFD	10.00	324
					3	2010	500	Constant Speed	7.00	324
					4	2010	500	Constant Speed	7.00	324
		New Booster Station	Pumped Storage (from tank)	805	1	2013	100	Constant Speed	1.00	290
					2	2013	100	Constant Speed	1.00	290
					3	2013	100	Constant Speed	1.00	290
		New Columbus	In-line Booster	910	1					
					2					

Note: Table assumes Northern Improvements complete to allow KRS2 to serve Northern System
1 - Generator to operate one 10 mgd pump
2 - High Service Pump 9, 10, and 11 are equpped with direct diesel drive engine
3 - Pump station equipped with Generator for Pumped Storage

Table 6-3

Tank Data

Pressure Zone HGL		Tank Name	Type	Pumped Storage	Year Installed	Capacity (MGD)	Diameter (feet)	Base Elevation (feet)	Storage Bottom (feet)	Overflow Elevation (feet)	Storage Height (feet)
Central System	Central 1185 feet	Clay Mills 1	Ground	Yes	1998	3.00	120	986	986	1023	37.0
		Clay Mills 2	Ground	Yes	2004	3.00	120	986	986	1023	37.0
		Parkers Mill	Ground	Yes	1968	3.00	113	985	985	1025	40.0
		York Street	Ground	Yes	1948	1.00	70	966	966	1000	34.8
		Mercer Road	Elevated	Yes	1964	2.00	105	982	982	1107	125.0
		Hume Road	Ground	Yes	1987	3.00	120	943	943	980	36.5
		Cox Street	Elevated	Yes	1955	1.00	78	957	1087	1117	30.0
		Cox Street	Ground	Yes	1948	1.00	70	967	967	1002	34.8
		Russell Cave	Ground	Yes	2005	1.00	75	991	991	1021	30.3
		Woodlake	Ground	Yes	2010	3.00	113	870	870	910	40.0
		Eastland	Elevated	No	2006	2.00	96	1034	1130	1170	40.0
		Tates Creek	Elevated	No	1955	0.50	50	1037	1148	1185	37.5
		Briar Hill	Elevated	No	1998	0.75	65	1000	1110	1150	40.0
		Muddy Ford	Elevated	No	1988	0.75	64	1009	1091	1130	39.5
		Hall	Standpipe	Yes	1965	0.20	20	1025	1025	1115	90.0
		Sadieville[1]	Standpipe	No	1975	0.38	30	920	920	992	72.0
Northern System	Owenton/Rockdale/ New Columbus 1119 feet	Hesler[1]	Standpipe	No	1995	0.24	20	953	953	1055	102.3
		Perry Street	Elevated	No	1959	0.10	N/A	946	1062	1070	8.5
		Fairgrounds	Elevated	No	1989	0.40	N/A	971	1066	1100	34.0
		Elk Lake[1]	Standpipe	No	1960s	0.10	20	910	910	960	50.0
		New Columbus	Standpipe	No	2003	0.23	17	980	980	1119	139.0
		Long Ridge[1]	Standpipe	No	1965	0.10	15	965	965	1043	78.0
		New 300,000 Tank	Elevated	Yes	2013	0.30	43	805	850	880	30.0
		New 600,000 Tank	Elevated	No	2013	0.60	56	955	1083	1119	36.0
	Monterey North 2 658 feet	Monterey	Standpipe	No	1995	0.12	20	600	600	651	51.3
	Carroll Co./Wheatley 1022 feet	Wheatley	Standpipe	No	1999	0.13	14	910	910	1022	112.0
	Bromley 1015 feet	Bromley	Standpipe	No	1993	0.18	17	908	908	1015	107.0
	Sparta 685 feet	Sparta[1]	Standpipe	No	1965	0.05	20	640	640	663	23.0
	Glencoe 730 feet	Glencoe[1]	Standpipe	No	1965	0.10	25	793	793	820	27.3

Note: Table assumes Northern Improvements complete to allow KRS2 to serve Northern System

1 - Tank is curently not in service

Table 6-4

Pump Evaluation

	Pressure Zone	Pumping Supply Station	Pumping or Rated Capacity (mgd) [1]	Reliable Capacity (mgd)	Maximum Day Pumping Requirements						Surplus or Deficit					
					Historic 10 year (mgd)	2010 (mgd)	2015 (mgd)	2020 (mgd)	2025 (mgd)	2030 (mgd)	Historic 10 year (mgd)	2010 (mgd)	2015 (mgd)	2020 (mgd)	2025 (mgd)	2030 (mgd)
Central System	Central	KRS1	8.00 8.00 8.50 10.00 10.00 10.00	91.50	71.82	61.36	73.33	76.03	79.17	82.44	19.68	30.14	18.17	15.47	12.33	9.06
		Wooklake[2]	10.00 10.00 10.00													
		RRS	6.50 10.00 4.00 7.00 5.50 4.00													
Northern System	Owenton/Rockdale/ New Columbus Bromley Sparta	New Northern Booster[3]	1.00 1.00 1.00	2.68	N/A	1.29	1.63	1.92	2.15	2.15	N/A	1.39	1.05	0.76	0.53	0.53
		Gerogetown Interconnection[4]	0.68													
	Montery North/South[5]	KRS2	10.00 10.00 7.00 7.00	2.00	N/A	0.11	0.13	0.16	0.18	0.18	N/A	1.90	1.87	1.84	1.83	1.83
	Carroll Co./Wheatley	Carroll Co Interconnection[4]	0.68	0.68	N/A	0.12	0.15	0.18	0.20	N/A	N/A	0.56	0.53	0.50	0.48	N/A
	Glencoe	Gallatin Co Interconnection[4]	0.38	0.38	N/A	0.05	0.06	0.07	0.08	N/A	N/A	0.34	0.32	0.31	0.31	N/A

Note: Table assumes Northern Improvements complete to allow KRS2 to serve Northern System
1 - Represents actual pumping capacities
2 - KRS2 is the supply, but pumps directly to Woodlake
3 - KRS2 is the supply, but pumps directly to New 300,000 gallon storage tank and 2 MG booster station
4 - The interconnection rated capacity is based on the size of the interconneciton (6-inch with Gallatine, 8-inch with Carroll and Georgetown) at a velocity of 3 ft/s.
5 - The reliable capacity of Monterey North/South is the reliable capacity of KRS2 (24 mgd) minus the reliable capaciyt of Woodlake (20mgd) and the reliable capacity of the New Northern Booster (2 mgd)

Table 6-5

Storage Evaluation

System	Pressure Zone	Total Storage Capacity (MG)[1]	Storage from other zone[2]	Max Cust. Elevation (feet)	Effective Storage (MG)	Year	Needed Equalization Storage Volume (MG)	Needed Fire Flow Reserves (MG) [3]	Total Storage Volume Needed (MG)	Total Storage Surplus or Deficit Volume (MG)	Effective Storage Surplus or Deficit Volume (MG)
Central System	Central	25.20	N/A	1070	22.29	2010	9.20	1.92	11.12	14.08	11.16
						2015	11.00		12.92	12.28	9.37
						2020	11.40		13.32	11.88	8.96
						2025	11.88		13.80	11.40	8.49
						2030	12.37		14.29	10.91	8.00
Northern System	Owenton/Rockdale/ New Columbus	1.63	0.00	1000	1.33	2010	0.16	0.06	0.22	1.41	1.12
						2015	0.20		0.26	1.37	1.07
						2020	0.23		0.29	1.33	1.04
						2025	0.26		0.32	1.31	1.01
						2030	0.26		0.32	1.31	1.01
	Monterey North/South[4]	0.42	0.00	700	0.42	2010	0.01	0.06	0.07	0.35	0.35
						2015	0.01		0.07	0.35	0.35
						2020	0.01		0.07	0.35	0.35
						2025	0.01		0.07	0.35	0.35
						2030	0.01		0.07	0.35	0.35
	Carroll Co./Wheatley	0.13	0.00	910	0.05	2010	0.02	0.06	0.08	0.06	0.06
						2015	0.02		0.08	0.05	0.05
						2020	0.03		0.09	0.05	0.05
						2025	0.03		0.09	0.04	0.04
						2030	0.03		0.09	0.04	0.04
	Bromley	0.18	1.47	700	0.06	2010	0.02	0.06	0.08	1.57	1.57
			1.43			2015	0.03		0.09	1.52	1.52
			1.39			2020	0.03		0.09	1.48	1.48
			1.37			2025	0.03		0.09	1.45	1.45
			1.37			2030	0.03		0.09	1.45	1.45
	Sparta[5]										
	Glencoe[6]										

Note: Table assumes Northern Improvements complete to allow KRS2 to serve Northern System
1 - Storage Capacity does not include tanks out of service as noted in Table 6-3
2 - Storage from other zone includes the total storage capacity less needed equalization for that zone. Assumes fire in
3 - Central System based on 8,000 gpm for 4 hours (1.92 MG), assumed 500 gpm for 2 hours (0.06 MG) in Northern
4 - Storage provided by the new 0.3 MG storage tank.
5 - Demands from Sparta is included in Bromley. Bromley Tank provides storage to Sparta Zone
6 - Storage and supplies are provided by Gallatin County Water Department.

Table 6-6

Central Emergency Supply Analysis

Source	Treatment Plant		# of pumps	Total pumping Capacity (MGD)	Reliable Pump Capacity (MGD)	Standby Pump Capacity (MGD)
Raw Water Facilities						
Pool 9	KRS1		6	74.4	62.0	0.0
Jacobson Reservoir	RRS		3	22.8	9.0	13.4
Lake Ellerslie			2	10.0	4.0	6.0
Pool 3	KRS2		4	34.0	27.0	10.0
	Total		**15**	**141.2**	**102**	**29.4**

Finished Water Facilities

Source		Clearwell Total Volume (MG)	# of pumps	Total pumping Capacity (MGD)	Reliable Pump Capacity (MGD)	Standby Pump Capacity (MGD)
KRS1		2.97	6	51.6	41.7	10.0
RRS		1.05	6	37.0	27.0	16.5
KRS2 [1]		1.60	4	34.0	27.0	10.0
Total		**5.62**	**16**	**122.6**	**95.7**	**36.5**

Central Storage

Source	Storage Capacity (MG)[1]	Pumped Stroage	# of pumps	Total pumping Capacity (MGD)	Reliable Pump Capacity (MGD)	Standby Pump Capacity (MGD)
Clay Mills 1	3.00	Yes	2	9.0	3.0	6.0
Clay Mills 2	3.00					
Parkers Mill	3.00	Yes	2	9.0	3.0	3.0
York Street	1.00	Yes	1	2.5	0.0	0.0
Mercer Road	2.00	Yes	1	5.0	0.0	0.0
Hume Road	3.00	Yes	3	12.0	6.0	9.0
Cox Street	1.00	Yes	1	3.0	0.0	0.0
Cox Street	1.00	Yes	1	2.5	0.0	0.0
Russell Cave	1.00	Yes	3	7.0	4.0	3.0
Woodlake	3.00	Yes	3	30.0	20.0	10.0
Eastland	2.00	No	N/A	N/A	N/A	N/A
Tates Creek	0.50	No	N/A	N/A	N/A	N/A
Briar Hill	0.75	No	N/A	N/A	N/A	N/A
Muddy Ford	0.75	No	N/A	N/A	N/A	N/A
Hall	0.20	Yes	2	1.2	0.6	0.0
Sadieville	0.38	No	N/A	N/A	N/A	N/A
Total	**25.58**			**81.2**	**36.6**	**31.0**

1 - KRS2 provides service to the Central System (8 mgd) and the Northern System (2 mgd)

Table 6-7

Northern Emergency Supply Analysis

Zone	Source	Treatment Plant		# of pumps	Total pumping Capacity (MGD)	Reliable Pump Capacity (MGD)	Standby Pump Capacity (MGD)
Raw Water Facilities							
	Pool 3	KRS2		4.0	34.0	27.0	10.0
Total				**4.0**	**34.0**	**27.0**	**10.0**
Finished Water Facilities							
Zone	**Source**		**Clearwell Total Volume (MG)**	**# of pumps**	**Total pumping Capacity (MGD)**	**Reliable Pump Capacity (MGD)**	**Standby Pump Capacity (MGD)**
	KRS2 [1]		1.6	4.0	34.0	27.0	10.0
Total			**1.6**	**4.0**	**34.0**	**27.0**	**10.0**
Northern Storage							
Zone	**Source**	**Storage Capacity (MG)**	**Pumped Stroage**	**# of pumps**	**Total pumping Capacity (MGD)**	**Reliable Pump Capacity (MGD)**	**Standby Pump Capacity (MGD)**
Owenton/Rockdale/ New Columbus	Hesler	0.24	No	N/A	N/A	N/A	N/A
	Perry Street	0.10	No	N/A	N/A	N/A	N/A
	Fairgrounds	0.40	No	N/A	N/A	N/A	N/A
	Elk Lake	0.10	No	N/A	N/A	N/A	N/A
	New Columbus	0.23	No	N/A	N/A	N/A	N/A
	Long Ridge	0.10	No	N/A	N/A	N/A	N/A
	New 300,000 Tank	0.30	Yes	3.0	3.0	2.0	2.0
	New 600,000 Tank	0.60	No	N/A	N/A	N/A	N/A
Monterey	Monterey	0.12	No	N/A	N/A	N/A	N/A
Wheately	Wheatley	0.13	No	N/A	N/A	N/A	N/A
Bromley	Bromley	0.18	No	N/A	N/A	N/A	N/A
Sparta	Sparta	0.05	No	N/A	N/A	N/A	N/A
Glencoe	Glencoe	0.10	No	N/A	N/A	N/A	N/A
Total		**2.64**		**3.0**	**3.0**	**2.0**	**2.0**

1 - KRS2 provides service to the Central System (8 mgd) and the Northern System (2 mgd)

Table 6-8

Central Emergency Supply Calculations

Year	Average Day Demand (MGD)	Total Standby Source Capacity (MGD) [1]	Emergency Supply Requirement @ 50% of ADD (MGD)	Emergency Storage Surplus or Deficit (MGD)
Central Treatment Facilities				
2010	40.73	24.50	20.37	4.14
2015	41.22	24.50	20.61	3.89
2020	43.19	24.50	21.60	2.91
2025	44.68	24.50	22.34	2.16
2030	46.19	24.50	23.10	1.41
Central Storage Facilities				
Year	**Average Day Demand (MGD)**	**Total Storage Capacity (MGD)**	**Emergency Storage Requirement @ 50% of ADD (MGD)**	**Emergency Storage Surplus or Deficit (MGD)**
2010	40.73	25.58	20.37	5.22
2015	41.22	25.58	20.61	4.97
2020	43.19	25.58	21.60	3.99
2025	44.68	25.58	22.34	3.24
2030	46.19	25.58	23.10	2.49

1 - Total Standy Source Capacity includes RRS and KRS2 finished water facilites minus 2.0 MGD for KRS2 providing service to the Northern System.

Table 6-9

Northern Emergency Supply Calculations[1]

Year	Average Day Demand (MGD)	Total Standby Source Capacity (MGD)	Emergency Supply Requirement @ 50% of ADD (MGD)	Emergency Storage Surplus or Deficit (MGD)
Owenton/Rockdale/New Columbus/Monterey/Bromley/Sparta [2]				
2010	0.89	2.00	0.45	1.56
2015	0.98	2.00	0.49	1.51
2020	1.07	2.00	0.53	1.47
2025	1.16	2.00	0.58	1.42
2030	1.16	2.00	0.58	1.42

Owenton/Rockdale/New Columbus/Monterey/Bromley/Sparta

Year	Average Day Demand (MGD)	Total Standby Storage Capacity (MGD)	Emergency Storage Requirement @ 50% of ADD (MGD)	Emergency Storage Surplus or Deficit (MGD)
2010	0.89	2.41	0.45	1.97
2015	0.98	2.41	0.49	1.92
2020	1.07	2.41	0.53	1.88
2025	1.16	2.41	0.58	1.83
2030	1.16	2.41	0.58	1.83

Carrol Co./Wheatley

Year	Average Day Demand (MGD)	Total Storage Capacity (MGD)	Emergency Storage Requirement @ 100% of ADD (MGD)	Emergency Storage Surplus or Deficit (MGD)
2010	0.08	0.13	0.08	0.05
2015	0.09	0.13	0.09	0.05
2020	0.10	0.13	0.10	0.04
2025	0.10	0.13	0.10	0.03
2030	0.10	0.13	0.10	0.03

Glencoe

Year	Average Day Demand (MGD)	Total Storage Capacity (MGD)	Emergency Storage Requirement @ 100% of ADD (MGD)	Emergency Storage Surplus or Deficit (MGD)
2010	0.03	0.10	0.03	0.07
2015	0.03	0.10	0.03	0.07
2020	0.04	0.10	0.04	0.06
2025	0.04	0.10	0.04	0.06
2030	0.04	0.10	0.04	0.06

1- It is assumed that the pressure zones in the Northern System that are served by KRS2 (Owenton/Rockdale/New Columbus, Monterey, Bromley, and Sparta) must meet the reduced storage requirement of 50% of the average demand. While Carrol Co./Wheatley and Glencoe pressure zones, because they are physically disconnected from the Northern System, will be required to have adequate storage to meet 100% of the average day demands in an emergency situation.

2- Total Supply is only 2.0 MGD due to standby puming capacity of the New Northern Booster Station

Table 6-10

American Water Power Outage Analysis

System	Pressure Zone	Year	Average Day Demand	Effective Storage Available (MG) [1]	Volume Available from Storage During a Power Outage (MG) [2]	Emergency Powered Supply Facilities (MGD) [3]	Total Capacity Available During a Power Outage	Percentage of Average Day Demand Supplied
Central System	Central	2010	40.73	17.04	10.93	24.50	35.43	87%
		2015	41.22	17.04	10.86	24.50	35.36	86%
		2020	43.19	17.04	10.56	24.50	35.06	81%
		2025	44.68	17.04	10.34	24.50	34.84	78%
		2030	46.19	17.04	10.11	24.50	34.61	75%
Northern System	Owenton/Rockdale/ New Columbus	2010	0.77	1.66	1.54	1.97	3.51	>100%
		2015	0.85	1.66	1.53	1.97	3.50	>100%
		2020	0.92	1.66	1.52	1.96	3.49	>100%
		2025	1.00	1.66	1.51	1.96	3.47	>100%
		2030	1.00	1.66	1.51	1.96	3.47	>100%
	Monterey North/South	2010	0.03	0.42	0.42	2.00	2.42	>100%
		2015	0.03	0.42	0.42	2.00	2.42	>100%
		2020	0.04	0.42	0.41	2.00	2.41	>100%
		2025	0.04	0.42	0.41	2.00	2.41	>100%
		2030	0.04	0.42	0.41	2.00	2.41	>100%
	Carroll Co./Wheatley	2010	0.08	0.08	0.065	0.00	0.06	81%
		2015	0.09	0.08	0.063	0.00	0.06	72%
		2020	0.10	0.08	0.062	0.00	0.06	65%
		2025	0.10	0.08	0.061	0.00	0.06	59%
		2030	0.10	0.08	0.061	0.00	0.06	59%
	Bromley/Sparta[4]	2010	0.09	0.10	0.08	2.74	2.83	>100%
		2015	0.10	0.10	0.08	2.65	2.73	>100%
		2020	0.11	0.10	0.08	2.56	2.64	>100%
		2025	0.12	0.10	0.08	2.47	2.55	>100%
		2030	0.12	0.10	0.08	2.47	2.55	>100%
	Glencoe [5]							

1 - Includes tanks at adequate elevation to serve customers and pumped storage tanks that have backup power to operate pump(s)
2 - Assumes Equalization Percentage per zone (15% of ADD) is not available.
3 - Does not include any pumped storage pump stations. Emergency Powered pumping facility for Central System includes 16.5 mgd from RRS and 8 mgd from KRS2.
4 - Surplus from Owenton/Rockdale/New Columbus Zone is Emergency Powered Puming avaialbe to Bromley/Sparta Zone.
5 - Storage and supplies are provided by Gallatin County Water Department.

SECTION 7
ENERGY EVALUATION AND OPTIMIZATION

Under separate cover, KAW's system was evaluated with the purpose of optimizing the delivery of water by saving energy and energy costs (*Kentucky American Water Energy Optimization, January 2011*). Highlights from the document are summarized below.

From an energy perspective, the costs were divided into two categories: 1) energy demand (kW), and 2) energy use (kWh). While the two share some overlap, they are largely exclusive of each other and can be treated as two distinct areas of savings. KAW has over 100 accounts, which were lumped into the four categories of "Major Pumps", "RRS Buildings", "Distribution Pumps," and "Remaining Facilities". An assessment of total billings from July 2011 through June 2012 indicates that approximately 86% of the total billings (energy plus demand) is from low and high service pumping (and transfer pumps).

The period from July 2011 through June 2012 was evaluated, inclusive of KRS-2 operations. The analysis indicated that KAW expended a statewide total of 46,866,367 kWh of energy during that time period. Total electric billings, including energy plus demand, rolled up to $3.7M, of which approximately 85% is due to "major pumps" (raw water pumps, finished water pumps and transfer pumps). Energy costs for the major pumps from July 2011 through June 2012 as shown in **Table 7-1**.

Table 7-1 KAW "Major Pumps" By Facility				
Facility	**Energy Use ($)**	**Energy Demand ($)**	**Remaining Charges ($)**	**Total Electric Costs**
KRS-1	$959,604	$525,062	$142,151	$1,676,818
KRS-1 Delivery(1)	$959,604	$525,062	$142,151	**$1,672,667**
RRS	$126,700	$93,478	$15,455	$235,633
Jacobson	$67,876	$39,118	$13,150	$120,144
Transfer	$60,297	$33,016	$50,000(1)	$143,313
RRS Delivery	$254,873	$165,613	$78,604	**$499,090**
KRS-2	$364,889	$101,585	$137,475	$603,949
Woodlake	$106,364	$74,743	$12,193	$193,300
KRS-2 Delivery	$471,253	$176,328	$149,668	**$797,249**
Owenton	$47,940	$4,691	$541	$53,172
Severn	$63,728	$10,633	$12,268	$86,629
North Delivery	$111,668	$15,324	$12,809	**$139,801**

1. A portion of the KRS-1 billings is from pumping to RRS.

Energy Demand Costs- Energy demand is the amount charged to KAW, and is based on the peak amount of power that was utilized, during any 15-minute interval, within the monthly billing period. Energy demand is computed differently among the three plants. KRS-1, Jacobson Pumps and Woodlake Pump Station are serviced by Kentucky Utilities (KU) and are all on Time of Day Service ("TODS"), which divides the day into the three periods of base demand, intermediate demand and peak demand, with peak being almost three times more expensive than base. RRS is also serviced by KU but energy demand is calculated using the single "Power Service" rate which varies between winter and summer. KRS-2 is serviced by Owen Electric and is calculated using a single "Large Industrial Rate", which is also time of day and varies with season.

Demand costs at all facilities were based on the average of instantaneous demands during the highest 15-minute interval of each billing period. The actual demand cost for that billing period was the highest of either 1) a percentage of the demand set by an elevated demand from the previous 11 months, or 2) the actual demand for that month. In most cases, demand costs for each month were set by the actual demand for that month, and not by the preceding billing periods. There are some instances where a demand “floor” has been set by a previous billing period and the water company is paying for unused demand. The greatest potential for energy demand cost savings could be incurred by shifting peak loads to less costly demand periods. There appears to be enough “stray” 15-minute intervals to consider this, however operations personnel would need to assess how this might impact the delivery of water.

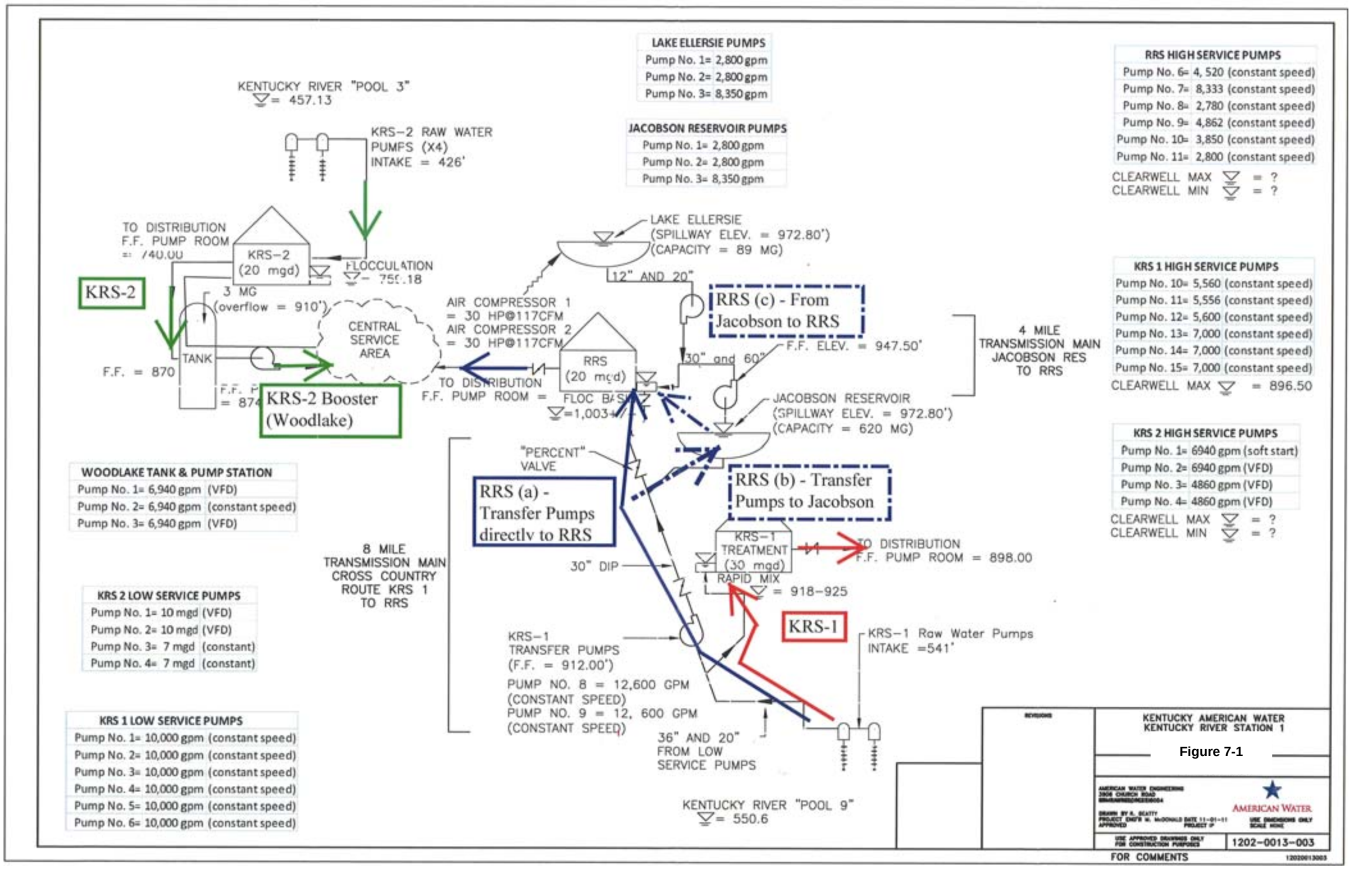
An analysis evaluated several methods for potentially decreasing energy demand at all of the plants, primarily through load shifting. Load shifting has the potential to reduce overall energy costs (i.e., total billed costs, including energy use, demand and “other costs) between 3% to 5% per year. These estimates are very preliminary and largely depend upon close coordination with operations personnel (i.e., load shifting, staggered equipment run time, real time SCADA monitoring). Note also, that there may be an overall energy use savings as well, however load shifting at the same kW to a less costly demand period but still at the same kW, will not reduce energy use.

Energy Use Savings - Energy use is the total energy used for the entire monthly billing period and is primarily determined by the efficiency of system operations (ie; hydraulic efficiency) and the efficiency of the mechanical equipment. Mechanical efficiency is primarily measured by pump efficiency. Wire to water pump testing has identified several pumps with efficiencies in the 50% to 60% range versus typical efficiencies of 75 to 85% for highly efficient pumps of this type. Recommendations have been made to rehabilitate or replace nine pumps (five at KRS-1 and four at RRS) for a potential system-wide efficiency gain of approximately 5%, or, based on actual billings, approximately 3% per year. A cost calculation indicates a potential composite “simple” payback of 7 to 10 years, depending on the frequency of run time for the rehabilitated pumps.

Hydraulic efficiency was evaluated using a hydraulic model of the system. Hydraulic modeling indicated that by stopping throttling of the transfer pump (and adding a VFD), a 1% to 2% per year overall energy savings could be achieved. Utilizing this model, the energy efficiency for the major pumps was calculated for each of the facilities by apportioning the pumping energy use required to deliver water for each treatment plant based on the three existing routes of raw and treated water delivery, as shown below. Route 1 is Pool #9 to KRS-1 to the system. Route 2 is Pool #9 to Transfer Pumps to RRS to the system (with variations to Jacobson), and Route 3 is

Pool #3 to KRS-2 to Woodlake pump station to the system. The three routes of delivery are shown on **Exhibit 7-1**.

Exhibit 7-1
KAW Hydraulic Flow Split



Note that there are three possible routes of raw water delivery to RRS. Raw water can either be delivered directly from the Kentucky River via the KRS-1 raw water pumps, or directly from the Jacobson Reservoir via the Jacobson pumps, or a combination of the two (i.e., double pumped). The least expensive route to RRS is directly from Jacobson and the most expensive is double pumping from the Kentucky River to Jacobson and from Jacobson to RRS.

Total gallons pumped from July 2011 through April 2012 was calculated from KAW's pumpage reports and total kWh consumed during the same period was obtained from KAW's electrical bills. A cost per million gallons (kWh/MG) was then calculated for each of the three routes, as listed in **Table 7-2** below. Note that the billings for RRS represent an average of the three routes, and will vary depending on how much water is obtained from the Jacobson Reservoir versus the other flow routes.

Table 7-2 Actual kWh/MG Consumed (Jul '11 through Apr '12)			
	kWh	**MG Treated**	**kWh/MG**
KRS-1[(1)]	19,667,878	5,746	3,300 - 3,500
RRS[(2)]	7,247,122	3,436	2,109
KRS-2[(3)]	9,233,296	2,036	4,534

1. "KRS-1" billings only (range is due to an estimate of how much energy should be allocated to RRS pumping.
2. "Transfer + Jacobson + RRS" billings.
3. "KRS-2 + Woodlake" billings.

Jacobson Reservoir - The reliability of the Jacobson Reservoir was evaluated from a pure average rainfall and average day withdrawal perspective. A simple withdrawal rate versus refill rate of the reservoir indicates that, in an average rainfall year (assumed to be spread evenly throughout the year) the Jacobson Reservoir would be capable of sustaining approximately 8 mgd over 30 days (approximately 250,000 gallons/month). Note that this is not the same as a Safe Yield analysis, which would take drought and excessive rainfall years into account and result in less than the 8 mgd during dry periods and more than 8 mgd during wet periods. The analysis does, however, provide an estimate supporting the viability of natural recharge.

Chemical Costs - Chemical costs were obtained from the Monthly Operating Reports ("MORs"). The maximum cost from each month was summarized, then each monthly maximum was averaged for the year. Results are shown below in **Table 7-3**.

Table 7-3 Chemical Costs			
MONTH	**KRS-1**	**KRS-2**	**RRS**
July	$97.65	$141.62	$161.39
Aug	$84.56	$161.04	$112.24
Sep	$96.07	$149.21	$142.82
Oct	$83.85	$92.85	$159.44
Nov	$108.33	$105.09	$144.51
Dec	$101.37	$111.32	$162.25
Jan	$94.65	$105.76	$132.14
Feb	$73.33	$71.34	$142.00
Mar	$99.63	$97.93	$141.99
Apr	$82.18	$68.79	$131.70
May	$94.59	$100.09	$112.96
Jun	$71.10	$107.48	$140.33
Jul	$85.95	$139.87	$97.02
AVG	**$90.25**	**$111.72**	**$136.98**

The cost per million gallons for chemical use ranges from $90.25 (KRS-1) to $136.98 (RRS) for the study period.

<u>Carbon Footprint</u> - Scope 1 and Scope 2 greenhouse gas direct emissions were evaluated. The Greenhouse Gas ("GHG") Protocol defines direct and indirect emissions as follows:

- Direct GHG emissions are emissions from sources that are owned or controlled by the reporting entity.
- Indirect GHG emissions are emissions that are a consequence of the activities of the reporting entity, but occur at sources owned or controlled by another entity.

The GHG Protocol further categorizes these direct and indirect emissions into three broad scopes:

- Scope 1: All direct GHG emissions.
- Scope 2: Indirect GHG emissions from consumption of purchased electricity, heat or steam.
- Scope 3: Other indirect emissions, such as the extraction and production of purchased materials and fuels, transport-related activities in vehicles not owned or controlled by the reporting entity, electricity-related activities (e.g. T&D losses) not covered in Scope 2, outsourced activities, waste disposal, etc.

The analysis indicates that KAW emitted approximately 32,630 tonnes of CO_2 between July 2011 and June 2012, which equates to about 4.74 lbs/kgal delivered.

Table 7-4
Carbon Emissions

Emissions Source	Emissions (metric tons CO_2e)
Direct Emissions	
Stationary combustion	73
Mobile sources	23
Process/fugitive	0
Refrigerant	4
Subtotal	**99**
Indirect Emissions	
Electricity	32,531
Purchased steam	0
Purchased chilled water	0
Subtotal	**32,531**

Alternative Energy Evaluation - In addition to load shifting and energy efficiency opportunities, alternative energy sources provide another option to address peak loading. Three of the most common currently being utilized are solar, wind and fuel cells. Another potential source of energy are micro-turbines. The cost of energy at KAW is currently $0.0330/kWh at RRS, $0.03522/kWh at KRS-1 and $0.04993/kWh at KRS-2. These prices are very reasonable with any source available on the market today, thus any form of alternative energy would have a difficult time competing with these prices. Energy subsidies and carbon offset credits can not be discounted however, and can often provide enough assistance to render an otherwise unfeasible project to be cost-competitive. Within that context the four technologies were evaluated from a technical perspective, keeping in mind that pure CapEx costs would not be competitive with current electric costs, however grants and credits may offset costs enough to provide a reasonable payback. This would entail research at the State and Federal level.

- Wind - Based on wind measurements conducted at the Bluegrass International Airport, wind speeds do not appear to be sufficient to support the desired capacity at KRS-1. If the option is ever considered in the future, however, a more detailed wind analysis should be conducted over the course of a year. To generate enough substantial load, an average wind speed of approximately 22 to 24 mph at the height of the turbine (> 200') would be required. Wind monitoring at the Blue Grass International Airport in Lexington, KY indicates that over the course of the year typical wind speeds vary from 0 mph to 16 mph (calm to moderate breeze), rarely exceeding 23 mph (fresh breeze).

- Fuel Cells - Fuel cells are a technically feasible option. Assuming the KRS-1 site currently has gas service, installation is simple and involves a water and gas connection along with the electrical connectivity work on site. The space requirement is small and the carbon footprint is greatly reduced. While technically feasible, this technology is still the most expensive.

- Solar Photovoltaics - and fuel cells are technically feasible assuming enough area is available (initially appears to be approximately 10,000 square feet).

- Microturbines - are technically feasible as long as there is sufficient flow and head, and the location is close to a receiving entity (ie; close to the treatment plant). The cost feasibility would depend upon to what extent these parameters are met and how much energy could be generated. An initial evaluation of the system did not reveal any viable locations.

Summary - An analysis of KAW's energy use was conducted for the 12 month period from July 2011 through June 2012. The analysis indicated several opportunities for cost savings and energy savings through operational adjustments (kW) and mechanical retrofits (kWh). It is recommended that KAW consider incorporating these measures as indicated in **Table 7-5**. Recommendations containing capital costs are included in **Section 1** of this CPS.

Table 7-5
Energy and Cost Conservation Opportunities

Measure	Description	Current Status
Projects Recommended or Started		
Energy Efficiency	VFD on Transfer Pump	CPS Project A-8
Energy Demand	Load Shifting	Program Underway[(1)]
Lighting	See Study	Implemented in 2011
Pump Rehabilitation	Increase Pump Efficiency	CPS Projects A-1 and A-8
Projects for Consideration		
HVAC Controls[(2)]	HVAC efficiency	Recommendation
Jacobson on/off aeration[(3)]	Off during peak demands	
Generator Testing[(4)]	Shave peak during monthly exercising	Recommendation
Stagger Equipment Loadings[(5)]	Reduce Peak Demands	Recommendation

1. Operational adjustments require input of operations staff. Meetings were held in June 2013 to kick off program.
2. A more detailed evaluation of a single line diagram is required.
3. Perform in parallel with Jacobson Reservoir Study.
4. Exercise generators during peak demand periods.
5. Discuss alternatives with KAW staff to decrease demand charges.

APPENDIX A

PLANNING CRITERIA AND REGULATION

TABLE OF CONTENTS

1.0 OVERVIEW

The purpose of the Comprehensive Planning Study is to recommend capital improvements that enable American Water to:

- continue to provide safe, adequate and reliable service to customers in its service territory
- meet domestic, commercial and industrial customer demand, and
- enhance fire protection capability.

The engineering criteria used to evaluate various system components are detailed in the following subsections. Note that all American Water systems are unique and some of the following sections may not be applicable to certain water systems. For example, surface water system criteria are not applicable for systems supplied solely by groundwater.

2.0 ENGINEERING CRITERIA

In planning the needed water facilities, accepted engineering standards and practices are utilized to evaluate facilities. Using these standards and practices, an assessment is made to determine if adequate capacity and an appropriate level of reliability are present for domestic, commercial, industrial usage, and fire protection needs.

Specific details regarding the planning criteria utilized are provided in the following subsections. Recommendations included in this Comprehensive Planning Study address improvements that work towards meeting the planning criteria described above. In addition, recommendations are included in this report where structural or mechanical problems with existing facilities are evident.

It is beyond the scope of this Comprehensive Planning Study to attempt to identify the end of the useful life of each piece of American Water's equipment; for example, the many miles of pipeline within a distribution system. Also, capital expenditures will occur over time due to normal aging and operational wear on existing equipment, and to enhance system security. For this and various other reasons, it is expected that American Water may encounter additional capital expenditures beyond those identified in this Comprehensive Planning Study.

2.1 Customer & Demand Projection Methodology

Projections of the total number of customers and their associated demands are reviewed for the water system over a fifteen-year planning horizon for this study. Since each water system is unique, the specific techniques used to project both customers and demands vary as appropriate. In general, the projections are developed based on a review of population trends, historic customer and demand data, and local planning commission forecasts. Large customers may be interviewed by telephone, or are asked to complete surveys of current and potential water consumption. Discussions are held with water system personnel, either in conjunction with field visits to the system and/or via telephone. More specific methods used to develop both customer and demand projections are discussed below.

Long term per customer water use is anticipated to be impacted water efficiency trends resulting from the passage of the federal Energy Policy Act in 1992 (EPAct) and the Energy Independence and Security Act of 2007. In 1992, it was anticipated that over the next 20-25 years, water utilities in the U.S. would realize demand reductions as a result of the national water efficiency requirements that were set by the EPAct. The efficiency requirements set maximum use levels for toilets (1.6 gallons per flush (gpf), urinals (1 gpf), showerheads (2.5 gallons per minute (gpm) and faucets (2.5 gpm). These efficiency standards applied to plumbing fixtures in new and renovated residential and non-residential facilities. The EPAct standards will have a cumulative and long term impact on lowering future indoor water usage as existing fixtures are gradually replaced.

AW has already seen savings on indoor residential consumption in the range of 10-17% over the last 12 years due to the Energy Policy Act requirements. Based on a thorough analysis of trends across AW, a 1% annual reduction in existing residential consumption is expected to result from the Energy Policy Act and The Energy Independence and Security Act of 2007 mandating lower flow plumbing fixtures and appliances in new homes and renovations, see **Table A-1**. Based on an assumed appliance lifetime and the years the new regulations on dishwashers and clothes washers take effect, the effects could be noticeable for another 10-15 years. Therefore this reduction should be considered (at a minimum) in planning analyses over the next 10-15 years before leveling off.

TABLE A-1

Flow rates from typical fixtures and appliances before and after Federal Standards

Type of Use	Pre-Regulatory Flow*	New Standard (maximum)	Federal Standard	Year Effective	WaterSense / ENERGY STAR Current Specification+ (maximum)
Toilets	3.5 gpf	1.6 gpf	**U.S. Energy Policy Act**	1994	**1.28 gpf**
Clothes washers**	41 gpl (14.6 WF)	Estimated 26.6 gpl (9.5 WF)	**Energy Independence & Security Act of 2007**	2011	**Estimated 16.8 gpl (6.0 WF)**
Showers	2.75 gpm	2.5 gpm	**U.S. Energy Policy Act**	1994	**2.0 gpm**
Faucets***	2.75 gpm	2.5 gpm (1.5 gpm)	**U.S. Energy Policy Act**	1994	**1.5 gpm at 60 psi**
Dishwashers	14.0 gpc	6.5 gpc for standard; 4.5 gpc for compact	**Energy Independence & Security Act of 2007**	2010	**4.25 gpc for standard; 3.5 gpc for compact**
Commercial Pre Rinse Spray Valves	1.8 to 6 gpm	1.6 gpm	**U.S. Energy Policy Act of 2005**	2006	**Under development**

* Source: *Handbook of Water Use and Conservation*, Amy Vickers, May 2001

** Average estimated gallons per load and water factor (see calculations)

*** Regulation maximum of 2.5 gpm at 80 psi, but lavatory faucets available at 1.5 gpm maximum (see calculations)

+Source: http://www.epa.gov/watersense/ and http://www.energystar.gov websites

ABBREVIATIONS USED	
gpcd	gallons per capita per day
gpf	gallons per flush
gpl	gallons per load
gpm	gallons per minute
gpc	gallons per cycle
WF	water factor, or gallons per cycle per cubic feet capacity of the washer (the smaller the water factor, the more water efficient the clothes washer)

Trends in Residential Consumption

Per customer residential usage in gallons per customer per day (gpcd) is projected based upon historic use patterns, consideration of the impacts of both existing and future water conservation efforts, and any potential changes in the number of persons per household.

Trends in Commercial Consumption

Projections of commercial customers and water demand are based primarily on historic trends. Growth in commercial water demand generally follows residential growth trends, as commercial development typically goes hand-in-hand with residential growth. One parameter that is considered in projecting commercial usage is the historic relationship between residential customers and commercial customers. Where confirmed major changes in commercial activity are identified (e.g., a large office complex or shopping center), appropriate figures are incorporated into the projections.

Trends in Industrial Consumption

As in the commercial category, industrial water demand projections are also dependent on historic usage trends. However, since there are typically far fewer industrial customers than commercial customers, it is easier to identify changes in water demands by the major industries, and thus forecast industrial water demand. The projected water usage for key industrial customers is based in part on information obtained through interviews conducted by water system personnel familiar with the service area.

Trends in Other Public Authority Consumption

Additional water efficiency is anticipated in the federal government sector with the introduction of the following executive order. Executive Order (EO) 13514, “Federal Leadership in Environmental, Energy, and Economic Performance,” was signed on October 5, 2009. EO 13514 introduces new greenhouse gas (GHG) emissions management requirements, expands water reduction requirements for federal agencies, and addresses waste diversion, local planning, sustainable buildings, environmental management, and electronics stewardship. EO 13514 enhances EO 13423, which requires agencies to reduce energy and water intensity and achieve other sustainability goals. This EO also defines specific water conservation criteria to reduce potable water consumption intensity 26 percent by FY 2020, compared to an FY 2007 baseline (This extends the water consumption intensity reduction requirement of EO 13423 by five years.) and reduce industrial, landscaping, and agricultural water use 20 percent by FY 2020, compared to an FY 2010 baseline. Based on an analysis of trends across AW, a 2.5% annual reduction in existing other public authority consumption occurred over the last 10 years presumably resulting from the Executive Orders discussed above in addition to the Energy Policy and Energy Independence and Security Acts see **Table A-1**.

Non-revenue and Unaccounted-for Water

Non-revenue water is projected based on historic annual data and discussions with water system personnel regarding future activities in these classifications. Non-Revenue water is defined as the difference between the total system delivery and the sum of all billed authorized (metered) consumption. It includes water for fire fighting, street cleaning, main flushing, and identifiable leakage or unbilled authorized consumption as well as water losses. Unbilled Authorized consumption includes usage such as: fire fighting, street cleaning, main flushing, and other beneficial uses that are not typically metered, but can be estimated. Water losses are defined as the difference between the total system delivery and the sum of all metered sales, and unbilled authorized consumption.

Maximum Day to Average Day Demand Ratios

The average day demand projections are determined from a summation of forecasts for the individual classifications. Future maximum day to average day demand ratios are estimated using a statistical analysis of historic data. Both a point estimate and an interval estimate of this ratio are determined. The point estimate is the median value of the ratio over the chosen historic period, and represents a value for which past ratios were above this value 50% of the time, and at or below this value 50% of the time. While this level may be adequate to estimate annual operational parameters, the level is not adequate on which to base long-term capital planning decisions. Rather, American Water's long-range forecasts utilize the criteria that facility planning should be based upon meeting projected maximum day customer demands with a 95% confidence level, where supply availability and affordability constraints don't dictate a lower confidence level. The confidence level value of 95% represents a level that is not expected to be exceeded more than once in 20 years. Planning facilities for a higher confidence level (e.g., in 20 of 20 years) would result in higher capital costs for small incremental gains in reliability.

To define the maximum day to average day demand ratio that will not be exceeded in a given number of years, an interval estimate around the mean value of this ratio is determined. The interval estimate defines the interval of values that the maximum to average day ratio will fall within for a certain degree of confidence. Several confidence intervals, namely the 99%, 95%, 50% and 5% intervals, are evaluated to illustrate the probable variation in maximum day demands that will likely be experienced during the planning period. Each confidence interval is calculated based upon multiplying the mean value (plus or minus the standard deviation) by a reliability coefficient.

2.2 Source of Supply Analysis Methodology

American Water's sources of supply should have the necessary quantity of water to meet the projected system demand, and be of good enough quality to provide finished water after treatment that complies with all Federal and State regulations. Cost effective water efficiency and demand side management options should be maximized prior to developing new sources of water.

The quality of the water from source of supply is regularly monitored for routine wet chemistry parameters such as pH, turbidity, alkalinity, parasites, microbes, etc., as well as for potential chemical contaminants in order to optimize the chemical treatment process.

State and American Water's standards are applied (whichever is more stringent) when evaluating the adequacy of supply. Sources of Supply should have the necessary quantity of water to supply the system's needs and be of good enough quality to provide, through treatment, finished water that meets all Federal and State regulations.

River supplies are considered adequate when the low flow of record is greater than or equal to the maximum day demand plus required passing flows. Surface reservoirs or lakes should have a safe yield sufficient to meet the average day demand during the critical drought period, based on an event with a recurrence interval of no less than one in fifty years. Groundwater supplies should have a safe yield sufficient to meet the average day demand during the critical drought period without overdrafting the supplying aquifer, based on an event with a recurrence interval of no less than one in fifty years. Sources of supply should also have sufficient allocation rights to permit average and maximum demands to be met.

2.3 Source Water Quality and Watershed Protection

The quality of surface water is affected by the amount and types of activity in watersheds that feed surface water sources. Runoff from farmland and urbanized area storm water, discharges from sewage treatment plants and industrial plants, and accidental spills in the water body can adversely impact raw water quality.

A source water monitoring program is maintained to ensure the quality of the finished water, and to control the costs associated with treating the water supply. The program is designed to define the potential for water quality impacts from both point and non-point sources. Watersheds are actively monitored through routine sampling of various raw water quality parameters. For surface supplies, monitoring activities are coordinated with local, state and federal authorities, and communication procedures have been established in the event of a contamination incident.

Raw water testing is performed by American Water's laboratory in Belleville, Illinois. Selected contaminants of concern include: inorganics, metals, minerals, pesticides, priority pollutants, synthetic and volatile organic chemicals, and microbiological and radiological parameters. A brief summary of some key parameters is provided in the Source of Supply and Production Section of this report to provide an indication of general water quality.

2.4 Treatment Facility Evaluation Criteria

Production facilities are defined as those used in raw water acquisition, transmission, treatment and pumping. Recommendations for capital improvements were developed after evaluating American Water's ability to provide a reliable and high quality water supply, to ensure continued compliance with existing and anticipated federal and state water quality and environmental regulations, and to meet projected customer demands through the planning period.

The goal of American Water is to continue to produce high quality water that meets or surpasses federal and state water quality standards. However, the characteristics of each individual source of supply require a diversity of treatment techniques including: disinfection, corrosion control, pH adjustment, and complete clarification/filtration. Other, more sophisticated treatment techniques are applied, as necessary, on a case-by-case basis.

The adequacy of production (treatment) facilities is evaluated based on the ability to provide an adequate, reliable finished water supply that will satisfy present and future demands, be aesthetically satisfactory to customers, and meet all federal and state regulations. Treatment plants are evaluated to assure that loading rates for all components are sustainable under maximum demand conditions without compromising water quality. Plant hydraulics are evaluated to ensure that adequate volumes of water can flow through the various components.

Each treatment process and chemical feed system at the plant is analyzed both as a separate entity and in conjunction with the facility's overall operations. Monitoring and control equipment should meet regulatory requirements and American Water's standards. Chemical feed and storage systems should be adequately sized to meet the full range of production rates while conforming to American Water's standards for safety, reliability, and construction. These issues are discussed further in this subsection.

Consideration is also given to providing adequate redundancy of treatment plant components to ensure reliability of service during scheduled or unscheduled maintenance and during emergencies. Adequate auxiliary power and/or dual utility power feeds should be provided to enable the plant to produce 100% of the average daily demand.

2.4.1 Drinking Water Regulations

Using the authorities granted under the Safe Drinking Water Act (SDWA) and state statutes, USEPA and state regulatory agencies have proposed and promulgated numerous drinking water regulations that will impact the treatment process and operation of Water Company facilities both now and in the future. In particular, Congress passed the 1996 Amendments to the Safe Drinking Water Act to reaffirm prior rules, establish new requirements for selecting contaminants to be regulated, allow for the analysis of health risk reduction, costs and benefits, and permit competing risks to be weighed. Currently, USEPA has standards set for almost 100 contaminants.

Current federal regulations are explained in more detail in the following subsections and a summary of the time frame for proposal, promulgation, and enforcement of recently promulgated and future regulations is shown in **Table A-2**.

Table A-2

Target Dates for Current and Future Drinking Water Regulations

Rule	Proposal Date(1)	Promulgation Date(1)	Compliance Date(1,2)
IESWTR	November 1997	December 16, 1998	January 2002
Stage 1 D/DBPR	November 1997	December 16, 1998	January 2002
LT1 ESWTR	April 2000	January 14, 2002	January 2005
Arsenic Rule	June 2000	January 22, 2001	2004 (new sources) 2006 (existing sources)
Filter Backwash Recycle Rule	April 2000	June 8, 2001	2004 (2006 if capital improvements required)
Radionuclides	April 2000	December 7, 2000	By Dec. 31, 2007
Stage 2 D/DBPR	August 2003	January 4, 2006	April 2012(3)
LT2 ESWTR	August 2003	January 5, 2006	April 2012(3)
Ground Water Rule	May 2000	November 8, 2006	December 1, 2009
Revisions to Lead and Copper Rule	July 2006	October 10, 2007	April 10, 2008(4)
Revised Total Coliform Rule	July 14, 2010	January 2013	April 1. 2016
Radon	November 1999	TBD	TBD

Notes:

(1) Dates for regulations that have not yet been promulgated are best estimates based on latest information.

(2) Compliance (effective) dates are normally 3 years after promulgation date. Many rules use a staggered implementation schedule with larger systems (e.g., systems serving larger populations) beginning implementation at the compliance date and smaller systems complying at a later date. Systems making major capital improvements may be allowed two additional years to achieve compliance, depending on the rule.

(3) The implementation schedule for the Stage 2 D/DBPR and Long Term 2 ESWTR will use a 6-month phase-in approach as follows: systems serving over 100,000 people; systems serving 50,000 – 99,999; systems serving 10,000 – 49,999; and systems serving less than 10,000.

(4) The revisions to the Lead and Copper Rule became effective upon State adoption of primacy for the revised rule (April 10, 2008 for States that adopt rules by reference).

Disinfection Byproduct Regulations

Disinfection of drinking water helps protect against microbial contamination. However, the disinfectants themselves can react with naturally-occurring materials in the water to form unintended organic and inorganic byproducts which may pose health risks. In order to address cancer concerns related to high disinfection byproduct (DBP) levels, USEPA has promulgated a number of regulations to limit DBP levels in the distribution system. The most recent sets of DBP regulations were developed in conjunction with new regulations to control microbial contaminants in an effort to balance microbial protection and DBP formation. Current disinfectant and disinfection byproduct limits are provided in **Table A-3**.

Previous DBP Rules – Through the Total Trihalomethanes Rule (TTHM Rule) (1979) and the Stage 1 Disinfectants and Disinfection Byproducts Rule (Stage 1 DBPR) (1998), USEPA established monitoring requirements and limits for TTHMs (0.1 mg/L under the TTHM Rule; reduced to 0.080 mg/L under the Stage 1 DBPR) and HAA5s (0.060 mg/L under the Stage 1 DBPR). Compliance was determined by calculating a running annual average based on a system-wide average of quarterly monitoring. The number of samples required was based on the number of treatment plants in the system. Only systems disinfecting the water with a chemical disinfectant were required to monitor and meet the Maximum Contaminant Levels (MCLs) (although some State drinking water programs extended the Stage 1 DBPR requirements to consecutive systems that purchase disinfected water).

The Stage 1 DBPR also set enforceable maximum residual disinfectant levels for chlorine, chloramines, and chlorine dioxide; MCLs for bromate and chlorite; and set requirements for removal of total organic carbon (TOC) in conventional treatment plants.

Stage 2 Disinfectants and Disinfection Byproducts Rule (Stage 2 DBPR) – The Stage 2 DBPR, promulgated in January 2006 in conjunction with the Long Term 2 Enhanced Surface Water Treatment Rule, is designed to further reduce cancer risks (and potential reproductive / developmental risks) and ensure that all customers are equally protected. The rule applies to all water systems that add a chemical disinfectant or deliver water that has been treated with a chemical disinfectant (e.g., consecutive systems). Although the TTHM and HAA5 levels have not changed (0.080 mg/L and 0.060 mg/L, respectively), compliance is now based on the Running Annual Average (RAA) at each location (Locational Running Annual Average or LRAA) and the number of samples required is based on the population served. Further, to ensure that monitoring captures both maximum TTHM levels as well as maximum HAA5 levels, systems are required to conduct an Initial Distribution System Evaluation to identify Stage 2 compliance monitoring locations. The rule also includes requirements for systems to investigate any “high DBP levels” under an Operational Evaluation.

Table A-3

Disinfectant and Disinfection Byproduct Limits

Disinfectant	MRDLG (mg/L)	MRDL (mg/L)	Comment
Free chlorine	4.0	4.0	as Cl_2
Chloramines (Total chlorine)	4.0	4.0	as Cl_2
Chlorine dioxide	0.8	0.8	as ClO_2
Contaminant	MCLG (mg/L)	MCL (mg/L)	Comment
Total trihalomethanes	0.0	0.080	Converting from RAA to LRAA
Total haloacetic acids (HAA5)	0.0	0.060	Converting from RAA to LRAA
Bromate	0.0	0.01	for systems with ozone
Chlorite	0.8	1.0	for systems with ClO_2

MRDLG - Maximum Residual Disinfectant Level Goal
MRDL - Maximum Residual Disinfectant Level
MCLG - Maximum Contaminant Level Goal
MCL - Maximum Contaminant Level

Surface Water Regulations

<u>Background</u>

The various Surface Water Treatment Rules govern water supplies whose source of drinking water is surface water, which it defines as "all water which is open to the atmosphere and subject to surface runoff" such as rivers, lakes, and reservoirs. Surface water is particularly susceptible to microbial contamination from sewage treatment plant discharges and runoff from storm water and snow melt. These sources often contain high levels of fecal microbes that originated in livestock wastes or septic systems. The Surface Water Treatment Rules set forth requirements for removal and / or inactivation of these contaminants.

<u>Previous Surface Water Treatment Rules</u>

The original Surface Water Treatment Rule (1979) sets non-enforceable health goals, or Maximum Contaminant Level Goals (MCLGs), for *Legionella*, *Giardia*, and viruses at zero because any amount of exposure to these contaminants represents some health risk. Since measuring disease-causing microbes in drinking water is not considered to be feasible, USEPA established a treatment technique in this rule rather than an MCL. Under the rule, all systems must filter and disinfect their water to provide a minimum of 99.9 percent combined removal and inactivation of *Giardia* and 99.99 percent of viruses. The adequacy of the filtration process is established by measuring turbidity (a measure of the amount of particles) in the treated water and determining if it meets USEPA's performance standard. Further, to assure adequate microbial protection in the distribution system, water systems are also required to provide continuous disinfection of the drinking water entering the distribution system and to maintain a detectable disinfectant level within the distribution system.

<u>Interim Enhanced Surface Water Treatment Rule</u>

The Interim Enhanced Surface Water Treatment Rule (1997) applies to systems using surface water or ground water under the direct influence of surface water that serve 10,000 or more persons. The rule also includes provisions for states to conduct sanitary surveys for surface water systems regardless of system size. The rule builds upon the treatment technique requirements of the original Surface Water Treatment Rule with the following key additions and modifications:

- MCLG of zero for *Cryptosporidium*
- 2-log *Cryptosporidium* removal requirements for systems that filter
- Strengthened combined filter effluent turbidity performance standards
- Individual filter turbidity monitoring provisions
- Disinfection profiling and benchmarking provisions
- Systems using ground water under the direct influence of surface water now subject to the new rules dealing with *Cryptosporidium*
- Inclusion of *Cryptosporidium* in the watershed control requirements for unfiltered public water systems
- Requirements for covers on new finished water reservoirs
- Sanitary surveys, conducted by states, for all surface water systems regardless of size

<u>Long Term 1 Enhanced Surface Water Treatment Rule (LT1 SWTR)</u>

While the Interim Enhanced Surface Water Treatment Rule only applies to systems serving 10,000 or more people, the Long Term 1 Enhanced Surface Water Treatment Rule (2002) is designed to strengthen microbial controls for systems serving fewer than 10,000 people. The rule will also prevent significant increase in microbial risk where small systems take steps to implement the Stage 1 Disinfectants and Disinfection Byproducts Rule. The Long Term 1 Rule generally tracks the approaches used in the Interim Enhanced Surface Water Treatment Rule for improved turbidity control, including individual filter monitoring and reporting.

<u>Long Term 2 Enhanced Surface Water Treatment Rule (LT2 SWTR)</u>

The purpose of the LT2 SWTR is to reduce disease incidence associated with *Cryptosporidium* and other pathogenic microorganisms in drinking water. The rule applies to all public water systems that use surface water or ground water that is under the direct influence of surface water regardless of the number of people served. The rule bolsters existing regulations and provides a higher level of protection of drinking water supplies by:

- Targeting additional *Cryptosporidium* treatment requirements to higher risk systems
- Requiring provisions to reduce risks from uncovered finished water storage facilities
- Providing provisions to ensure that systems maintain microbial protection as they take steps to reduce the formation of disinfection byproducts

Systems initially monitor their water sources to determine treatment requirements. This monitoring involves two years of monthly sampling for *Cryptosporidium*. To reduce monitoring costs, small filtered water systems will first monitor for *E. coli*–a bacterium that is less expensive to analyze than *Cryptosporidium*–and will monitor for *Cryptosporidium* only if their *E. coli* results exceed specified concentration levels.

Filtered water systems will be classified in one of four treatment categories (bins) based on their monitoring results. Most systems are expected to be classified in the lowest bin and will face no additional requirements. Systems classified in higher bins must provide additional water treatment to further reduce *Cryptosporidium* levels by 90 to 99.7 percent (1.0 to 2.5-log), depending on the bin. Systems will select from different treatment and management options in a "microbial toolbox" to meet their additional treatment requirements. All unfiltered water systems must provide at least 99 or 99.9 percent (2 or 3-log) inactivation of *Cryptosporidium*, depending on the results of their monitoring.

Additionally, systems that store treated water in open reservoirs must either cover the reservoir or treat the reservoir discharge to inactivate 4-log virus, 3-log *Giardia lamblia*, and 2-log *Cryptosporidium*. These requirements are necessary to protect against the contamination of water that occurs in open reservoirs.

Finally, systems must review their current level of microbial treatment before making a significant change in their disinfection practice. This review will assist systems in maintaining protection against microbial pathogens as they take steps to reduce the formation of disinfection byproducts under the Stage 2 DBPR, which USEPA is finalizing along with the LT2 ESWTR.

Arsenic Rule

In January 2001, USEPA reduced the MCL for arsenic from 0.05 mg/L to 0.010 mg/L based on new health data.

Radionuclides Rule

In 2000, USEPA revised the existing radionuclides regulation, which had been in effect since 1977, by requiring new monitoring provisions that will ensure that all customers of community water systems will receive water that meets the appropriate limits for radionuclides in drinking water. This included a standard for uranium as required by the 1986 Amendments to the Safe Drinking Water Act. The current standards are:

- Combined radium 226/228 standard of 5 pCi/L
- Gross alpha standard for all alphas of 15 pCi/L (not including radon and uranium)
- Combined standard of 4 mrem/year for beta emitters

- Uranium standard of 30 µg/L

Lead and Copper Rule Revisions

In 1991, USEPA published the Lead and Copper Rule in an attempt to control lead and copper in drinking water. The rule aimed to minimize lead and copper in drinking water, primarily by reducing water corrosivity. Lead and copper enter drinking water primarily through plumbing materials. Exposure to lead and copper may cause health problems ranging from stomach distress to brain damage.

In the Lead and Copper Rule, USEPA established "Action Levels" for lead and copper. Based on first-draw samples collected at taps within the distribution system, lead and copper concentrations must be less than 0.015 mg/L and 1.3 mg/L, respectively, in ninety percent of the samples. Selected sample sites must consist of single-family residences which contain copper pipes with lead solder installed after 1982, which contain lead pipes, or which are served by a lead service line. Following implementation of state-specified "optimal" treatment to minimize lead and copper concentrations at consumer taps, annual follow-up monitoring is required. If the results of follow-up monitoring indicated that the system is consistently in compliance with the lead and copper Action Levels, the state may elect to reduce the annual monitoring requirements. Should follow-up monitoring indicate noncompliance, the utility is required to initiate a public education program, collect additional water quality samples, and possibly begin a program of replacing lead service lines.

In 2000, USEPA published minor revisions to the Lead and Copper Rule. These revisions streamline and reduce monitoring and reporting burden, and address implementation problems and issues arising from legal challenges. The minor revisions addressed implementation problems and issues arising from legal challenges to the 1991 rule.

USEPA issued Short-Term Revisions to the Lead and Copper Rule in late 2007 that covered a number of issues:

- The revisions addressed confusion about sample collection by clarifying language that speaks to the number of samples required and the number of sites from which samples should be collected. It also modified definitions for monitoring and compliance periods to make it clear that all samples must be taken within the same calendar year. Finally, the revisions added a new reduced monitoring requirement, which prevents water systems above the lead action level to remain on a reduced monitoring schedule.

- The revisions require water systems to provide advanced notification and gain the approval of the primacy agency for intended changes in treatment or source water that could increase corrosion of lead.

- The revisions require that all utilities provide notification of tap water monitoring results for lead to owners and/or occupants of homes and buildings who consume water from the taps that are part of the utility's sampling program.

- The revisions add a requirement for utilities to reconsider previously "tested-out" lines when resuming lead service line replacement programs.

- The revisions change the content of the message to be provided to consumers in the event of a lead action level exceedance, changes how the materials are delivered to consumers, and the timeframe in which materials must be delivered.

Ground Water Rule

USEPA published the final Ground Water Rule (GWR) in the Federal Register on November 8, 2006. The purpose of the rule is to provide for increased protection against microbial pathogens in public water systems that use ground water sources. USEPA is particularly concerned about ground water systems that are susceptible to fecal contamination since disease-causing pathogens may be found in fecal contamination including *E. coli*, *enterococci*, and coliphage.

The Ground Water Rule applies to public water systems that serve ground water. The rule also applies to any system that mixes surface and ground water, where the ground water is not treated in by the surface water treatment process. The risk-targeting strategy incorporated into the Ground Water Rule provides for:

- Regular sanitary surveys of public water systems to look for significant deficiencies in key operational areas;

- Triggered source water monitoring when a system that does not sufficiently disinfect drinking water identifies a positive sample during its Total Coliform Rule monitoring and assessment monitoring (at the option of the state) targeted at high-risk systems;

- Implementation of corrective actions by ground water systems with a significant deficiency or evidence of source water fecal contamination to reduce the risk of contamination; and,

- Compliance monitoring for systems that are sufficiently disinfecting drinking water to ensure that the treatment is effective at removing pathogens.

The compliance date for triggered monitoring (and associated corrective actions) and compliance monitoring was December 1, 2009. There are no timeframes associated with the assessment monitoring because it is at the option of state. States must complete their initial round of sanitary surveys by December 31, 2012 for most community water systems. States will have until December 31, 2014 to complete the initial sanitary survey for community water systems that are identified by the state as outstanding performers and non-community water systems.

Revised Total Coliform Rule (RTCR)

The USEPA has revised the 1989 Total Coliform Rule (TCR), a national primary drinking water regulation (NPDWR). The purpose of the 1989 TCR is to protect public health by ensuring the integrity of the drinking water distribution system and monitoring for the presence of microbial contamination. USEPA anticipates greater public health protection under the revised requirements, which are based on recommendations by a federal advisory committee and the agency's consideration of public comments.

The final RTCR establishes a Maximum Contaminant Level Goal (MCLG) and an MCL for E. coli and eliminates the MCLG and MCL for total coliforms, replacing it with a treatment technique for coliforms that requires assessment and corrective action. Specifically:

- o The revised rule establishes an MCLG of 0 for E. coli, a more specific indicator of fecal contamination and potential harmful pathogens than total coliform. EPA has removed the 1989 MCLG and MCL for total coliform. Many of the organisms detected by total coliform methods are not of fecal origin and do not have any direct public health implication. The "acute" total coliform MCL violation under the 1989 TCR has been maintained as the MCL for E. coli under the RTCR.
- o Under the new treatment technique for coliforms, total coliforms serve as an indicator of a potential pathway of contamination into the distribution system. A PWS that exceeds a specified frequency of total coliform occurrence must conduct an assessment to determine if any sanitary defects exist and, if found, correct them. In addition, under the new treatment technique requirements, a PWS that incurs an E. coli MCL violation must conduct an assessment and correct any sanitary defects found.

Radon

The 1991 proposed standard for radon was withdrawn under the 1996 SDWA Amendments. Under the new SDWA Amendments, the USEPA prepared a risk assessment study for radon in drinking water using the best available science. In addition, USEPA directed an assessment of the health risk reduction benefits that are associated with reducing radon concentrations in indoor air. The USEPA published a health risk reduction and cost analysis in February 1999, for exposure to radon in drinking water and air. This included a discussion on the costs and benefits of multimedia mitigation programs. The MCLG and MCL for radon were proposed in November 1999 at 0 pCi/L and 300 pCi/L, respectively. An alternative MCL was proposed at 4,000 pCi/L with a Multimedia Mitigation program (MMM) to address radon risks in indoor air. The State or Community Water System (serving over 10,000 persons) can develop a MMM program. Most Community Water Supplies (CWSs) serving 10,000 or less are expected to meet the Alternative Maximum Contaminant Level (AMCL) and to participate in a State MMM. The USEPA is strongly encouraging States to take full advantage of the flexibility and risk reduction opportunities in the MMM program. The USEPA fact sheet on Radon states, "It is more cost-effective to reduce risk from radon exposure from indoor air, than from drinking water". Radon is generally not found in surface water at levels of concern, but is present at high levels in some groundwater sources.

States regulators have indicated that implementing an MCL / AMCL regulation would be difficult. However, since the regulatory construct was included as part of the 1996 Amendments to the Safe Drinking Water Act, USEPA would be hard pressed to pursue another approach. Therefore, it is unlikely that the rule will be promulgated as proposed and USEPA has not indicated an expected promulgation date.

Regulating New Contaminants

The 1996 SDWA Amendments include a process that USEPA must follow to identify new contaminants which may require Federal regulation in the future. Specifically, USEPA must periodically release a Contaminant Candidate List (CCL), which is a list of unregulated contaminants that it uses to prioritize research and data collection efforts to help make a determination whether to regulate a specific contaminant. USEPA must make "regulatory determinations" on at least five contaminants every five years – this could include a "determination" to regulate, to not regulate, to issue a health advisory, or that no action is necessary. When making a "determination" to regulate, the law requires that USEPA consider three areas: 1) projected adverse health effects from the contaminant; 2) the extent of occurrence of the contaminant in drinking water; and 3) whether regulation of the contaminant would present a "meaningful opportunity" for reducing risks to health. More information on the CCL process is available at http://water.epa.gov/scitech/drinkingwater/dws/ccl/index.cfm

Related to the CCL process is the Unregulated Contaminant Monitoring Rule (UCMR) program. Under this program, monitoring is required at selected water systems to determine the occurrence levels for contaminants that may occur in drinking water and may potentially have an adverse health impact, but are not yet regulated at the Federal level. Most contaminants that are monitored under the UCMR are listed on the CCL. More information on the UCMR, including unregulated contaminants currently included in the rule, is available at http://www.epa.gov/safewater/ucmr/index.html

Once USEPA decides to regulate a contaminant (which could be through the CCL process, the Six Year Review of current regulations required under the SDWA, or by a separate decision), the regulatory development process is a slow, deliberative process. From the time that USEPA makes a decision to regulate, the process of developing a proposed rule, taking comment, responding to comment, and finalizing the rule can take anywhere from 3-5 years or even longer. Water systems then have 3 years from the date of promulgation to prepare to comply with the rule, so in essence, water systems have at least 6-8 years until they would have to be in compliance with any new regulations that USEPA would begin working on today.

2.4.2 Design and Construction Standards

Many states have adopted regulations governing water quality that are identical to federal regulations. However, in several instances, states have established regulations and standards that are more stringent than federal requirements, sometimes by a significant amount. For a state to be granted primacy by the USEPA, that state's adopted regulations must be at least as stringent as the federal regulations.

The Recommended Standards for Water Works has been used by many States to form the basis of standards for the design and construction of public water supply systems. It should be noted that the actual design of facilities may vary from these standards, and will be subject to review by each state. State-specific regulations are discussed in the Production Section of this report. Some of the major provisions of the Recommended Standards for Water Works are summarized in **Table A-4**.

Table A-4

Major Provisions of the Recommended Standards for Water Works

Treatment Process	Summary of Standard
Mixing	• The detention period should be no less than 30 seconds.
Flocculation	• The detention period should be no less than 30 minutes. • Duplicate facilities should be provided.
Sedimentation	• The detention period for conventional basins should be no less than 4 hours.

	• At least two units should be provided for redundancy. • Inlet and outlet devices should be provided to provide uniform settling velocities and to minimize short-circuiting. • Mechanical solids collecting equipment should be provided.
Filtration	• At least two units should be provided for redundancy, and provisions should be made to assure continuity of service with (1) filter removed from operation. • The normal filtration rate is 2 gpm/ft^2, but rates can be increased to 3 gpm/ft^2 for greensand media and 4 gpm/ft^2 for dual media. • Indicating rate-of-flow controllers, loss-of-head gauges and filter-to-waste piping should be provided for each filter. • Provisions should be made to backwash filters at a rate between 15 and 20 gpm/ft^2 for a period not less than 15 minutes. Rate-of-flow control should also be provided. • Filter media should have a total depth between 24 and 30 inches.
Disinfection	• Standby equipment shall be provided to replace the largest unit during shutdowns.

2.5 Underground Storage Tank Management

Federal regulations call for upgrading existing underground fuel and chemical storage tanks to provide leak detection, corrosion protection and spill/overflow protection.

2.6 Electrical Service and Standby Power

In order to provide an acceptable degree of reliability, the ability to produce 100% of the projected average day demand is desirable in each distribution system pressure zone. In some instances, the availability of a sufficient volume of finished storage water or interconnections helps to meet this guideline. Emergency generators, engine driven pumps and/or dual utility power feeds can also be used to provide temporary power to the plant during an outage.

2.7 Partnership for Safe Drinking Water

The Partnership for Safe Drinking Water is a voluntary cooperative effort between USEPA, American Water Works Association (AWWA), and other drinking water organizations to help ensure the safety of America's drinking water. According to AWWA, "The Partnership provides a new measure of safety by implementing prevention programs where legislation or regulation do not exist. The preventative measures are based around optimizing plant performance and thus increasing protection against microbial contamination in America's drinking water supply."

The Partnership agreement requires participating utilities to attempt to reach certain performance goals and to perform a self-assessment of surface water plant performance. Performance criteria include the following targets:

- Clarified turbidities less than 3.0 NTU in 95% of samples,
- Filter effluent turbidities less than 0.1 NTU in 100% of samples,
- Filter effluent turbidities less than 0.3 NTU for less than 15 minutes following filter backwash.

American Water has targeted these performance criteria as treatment goals, and has implemented operational changes at various treatment plants where necessary and practical to consistently meet the targets.

2.8 AW Guidelines for Chemical Feed, Storage and Containment

Compressed gas and liquid treatment chemicals used in the water industry are generally stored and fed in a concentrated form with many being strong acids and bases. While these chemicals are necessary to provide safe, potable water, proper management of the chemicals is necessary to protect the consumer, American Water's personnel, and the environment. In addition, many of these chemicals can damage American Water's facilities if the proper equipment and safeguards are not provided.

Chemical feed and storage facilities at American Water treatment plants are evaluated to determine adequacy compared to the AW Standard for Liquid Chemical Storage, Feed, and Containment. These guidelines go beyond the minimum requirements of the Recommended Standards for Water Works by providing increased protection to consumers, Company personnel and facilities, and the environment.

Feed equipment is considered adequate if sufficient capacity is available to treat the water while considering maximum flow and feed rates with the largest chemical feeder or pump out of service. Chemical storage is considered adequate if 31 days storage is available while considering maximum flow and feed rates, and provisions for containment. Primary containment is defined as the container holding the chemical. Secondary containment is a structure designated to hold spillage or leakage. The minimum secondary containment volume is considered to be 110 percent of the largest storage tank volume within the containment area.

Facilities to house compressed gas feed systems are required to provide safety for the operator and local population, and to ensure adequate containment in the event of a gas leak. Individual feed and storage rooms are recommended for all installations. The storage room should contain all elements of the feed system which are under pressure, and be sized for a minimum 30 day supply.

More specific guidance for liquid and compressed gas feed systems is provided in the AW Engineering Standards: Liquid Chemical Storage, Feed and Containment, and Compressed Gas Feed Systems and Storage Facilities. An analysis of chemical facilities can be found in the Production Section of this report.

3.0 DISTRIBUTION PIPING, PUMP AND STORAGE EVALUATION CRITERIA

The ability of distribution system facilities to provide safe, adequate and reliable service to customers is analyzed based on forecasted customer demands and fire protection requirements. Computer modeling of the distribution system is utilized as a tool in the analysis to determine system deficiencies and evaluate the effectiveness of proposed improvements under future demand conditions. Published reports from the Insurance Service Office (ISO) are used as a guideline in analyzing the ability of various system components to deliver fire protection. The ISO is a major source of information, products, and services related to property and liability risk; and one of their important services is to evaluate the fire suppression delivery systems of jurisdictions around the country. The result of those reviews is a classification number that ISO distributes to insurers, who then use the Public Protection Classification (PPC) information to help establish fair premiums for fire insurance. Generally, communities with better fire protection are offered lower insurance premiums.

3.1 Distribution System Analysis Methodology

The analysis of American Water's facilities includes pipelines, storage tanks, booster stations and emergency power provisions. Under peak demand conditions, a number of minimum standards should be met for each of these facilities. These standards are described below.

- **Pipelines** - Distribution system mains are considered adequate if they can meet customer demand at a minimum system pressure of 20 psi. Fire protection requirements should be met while maintaining a minimum pressure of 20 psi in the distribution system. (Note: State and local guidelines may require that higher pressures be maintained.)

- **Distribution Storage** - Storage facilities are considered adequate if the effective volume of the facility, or groups of facilities acting together, provide sufficient volume to meet equalization needs and a fire protection reserve (if necessary) during maximum day demand events. In addition, State regulations are also considered as they relate to a particular distribution system.

The effective volume of storage is that quantity which can be used from the tank while maintaining adequate system pressures under the domestic and fire flow conditions outlined above for distribution mains. The ideal equalization volume is that quantity of water needed to allow the production plant or booster station output rates to be constant and equal to the daily demand on the maximum day of the year. The actual use of equalization storage enables a reasonably constant rate of treatment plant or booster station operation, and thereby promotes overall system efficiency and economy.

Existing storage capacity was also analyzed to determine its contribution to overall system reliability. Where appropriate, recommendations are made if additional storage will significantly improve system reliability (e.g., ability of the system to maintain service to customers during an emergency, such as a power outage, a chemical or fuel spill impacting the source of supply, or a large main break).

Other factors considered when determining storage reserves are the fire protection ratings published by ISO. Storage reserves for a given pressure zone are calculated on the basis of the highest published ISO Needed Flow and duration. The impact of storage volume on water quality is also considered when sizing proposed storage facilities. ISO's municipal fire protection testing may identify sites with needed fire flows greater than 3,500 gpm for a duration of three hours. In many pressure zones, particularly in residential areas, the identified maximum is less than 3,500 gpm. Where individual structures are assigned ISO Needed Flows above 3,500 gpm, it is assumed that fire protection needs in excess of 3,500 gpm at these sites will be satisfied through the development of individual customer-owned fire suppression systems.

- **Distribution System Booster Stations** - Booster pumping facilities are considered adequate if the capacity of the pump stations, with the largest pumping unit out of service, is sufficient to meet the maximum daily demand projected to occur within each pressure zone. When storage facilities are not present in a pressure zone, the booster station pumps should be able to meet peak instantaneous demands at adequate pressure. In pressure zones without storage, the booster pumps may also provide the only source of fire protection.

- **Emergency Power** - The ability to provide continuous service during a power outage is critical to a system's reliability and depends on several factors including: the nature of the electrical service (i.e., service from one vs. two substations), the presence of any floating storage within the pressure zone, standby electrical generating capacity, and the availability of pumps which can be driven by diesel fuel or natural gas.

During a power outage, the demand is assumed to be 100 percent of the average day demand. Analysis of outages in other systems has shown this to be a reasonable estimate of customer usage under these conditions.

The facilities within a pressure zone are considered adequate if 100 percent of the projected average day demand can be met from emergency powered pumping facilities, or if floating storage facilities are available, to provide the needed demand for more than 24 hours.

A number of distribution system improvement projects are recommended in this report with specific justification such as assuring safe, adequate, and reliable general service, while others are primarily to improve water transmission, provide redundancy, and to enhance fire protection. Each type of project has multiple benefits that may result in general improvement of the system in terms of increased pressures, flows, reliability, and more stable water quality.

3.2 Distribution System Computer Modeling

The computer model has become a valuable tool for developing future distribution system improvement programs. A computer model is developed for the distribution system using the WATERCAD software program. Data relating to pipe diameter, length, material, age and connection points are obtained from distribution maps and records supplied by the Water Company. Pipe friction coefficients (or C-factors) are determined for selected pipelines utilizing available flow test data, or standard values based on the age, material and size of pipeline. These results are then used to estimate C-factors for other pipelines of similar diameter, age and material.

Customer demands are modeled by applying meter route data at the appropriate pipeline junctions in the computer model to simulate customer demands. Large customers are considered individually in order to apply specific peaking factors to metered consumption. After any newly installed pipes, tanks, or booster stations are added to the model, the output data are compared with known pressures, flows, and water levels obtained from data recorders at key locations in the actual system. Consumption data or pipeline data are then adjusted to achieve the best possible correlation between actual and modeled parameters. Three demand scenarios are generally considered:

- The peak hour demand on the maximum day.
- The minimum hour demand on the maximum day (during night time storage refill conditions).
- The maximum day flow for use in evaluating fire protection.

After calibration, future demands are allocated throughout the system to sectors of projected growth for the individual scenarios. Successive computer runs are then made to test various alternatives of distribution system improvements and their success in solving system problems. Final selection of distribution system improvement projects is based in part on computer simulations of water system hydraulics under these various present and future demand scenarios.

3.3 Property Sizing for Distribution System Storage Tanks

Where projects are recommended involving the construction of distribution storage tanks to meet equalization and fire protection storage needs in the system, preliminary sites are chosen for planning purposes. In the preliminary design phase of such projects, the final site selection and purchase of appropriate property for the tank is undertaken. The American Water guideline "Property Sizing for Steel Tank Construction" includes the following considerations for lot sizing:

- Obtain a lot large enough to provide an adequate layout area for steel plates, columns, etc., during tank erection.

- The size of the lot should be sufficient to provide reasonable isolation from existing or possible future residential or industrial building sites or parking facilities.

- The size of the lot should also be sufficient to minimize airborne migration during blast cleaning and painting operations (additional containment procedures may be required by regulatory agencies).

Sufficient property should be purchased to provide a minimum of 100 feet from the tank sidewall. In cases where elevated tanks are involved, the 100-foot dimension would be from the sidewall of the bowl. Additionally, in the case of an elevated tank, the length of each side of the lot should be at least twice the height of the tank. The size of the lot for an elevated tank would be the greater of the two criteria. In addition, specific regulatory requirements regarding site screening may increase lot size requirements.

Another factor for consideration when purchasing a tank site is the handling of the water produced during an accidental overflow event. These flows typically involve high volumes for short durations. If an adequately sized conduit or pond, which can be permitted to receive intermittent flows of chlorinated water, is not easily accessible from the tank site, sufficient property should be purchased to allow construction of an overflow retention pond at the site. The general rule for estimating the size of the retention pond is based on the assumption that it may take about 30 minutes for American Water's operating personnel to valve off the tank to stop an overflow event.

Prior to purchasing property for a tank site, the need and potential for constructing multiple tanks at the site should be considered. This may be either a present or future need. If additional tanks may be located at the site in the future, a preliminary plan of the future site layout should be completed to define the appropriate lot size.

3.4 Distribution System Main Replacement Programs

Many water distribution networks operated throughout American Water have been developed over many years. In the past, distribution mains have been acquired or installed using then-current design standards that, in some cases, do not conform to present day engineering design practice. Some mains that were installed under these historically acceptable practices are now unable to satisfy current requirements. Many of American Water's operating companies have an ongoing main replacement program to address these deficiencies.

Mains in need of replacement typically include pipes that are 4-inch diameter or smaller, unlined, cast iron, or galvanized iron pipe. Priority under the main replacement programs is given to those pipes which have become maintenance problems. It is recommended that Investment Projects continue to be developed with projects prioritized on an annual basis. The Investment Projects should be revised annually as mains are replaced and newer priorities are added.

The design of main replacements and extensions is normally based upon projected system demands and the maximum needed fire flow, but the following general criteria should also be followed:

- Mains should not be less than 8-inches in diameter, except where the main does not serve a fire hydrant and there is no possible further extension of the facilities beyond 500 feet, or where proper engineering justification for a smaller main can be made.

- Major transmission mains or mains which potentially can serve as major transmission mains should not be less than 12-inches in diameter.

Many pipelines that will be constructed in the future to reinforce an existing system will be installed parallel to smaller, older pipelines. In most instances, it is recommended that the old main be retired and that all fire hydrants and customer services be connected to the new main. As part of American Water's policy, any lead services encountered during water main installation are generally replaced as part of that construction project.

3.5 Tank Maintenance Programs

Each operating system has developed a tank maintenance program to schedule routine inspections, evaluate the condition, and identify needed improvements. Any deficiencies are then budgeted for improvements. A tank inventory should be maintained of all steel tanks utilized in American Water including distribution storage, washwater, sedimentation and wastewater holding tanks.

3.6 Geographic Information Systems

GIS (Geographic Information Systems) software provides an association between graphic data from maps and drawings and textual data in a database. Facility data such as hydrant records, tap orders, and maintenance history can be linked to American Water's distribution system maps, providing a geographic reference for managed facilities.

GIS systems replace the need to use Computer-Aided Drafting (CAD) to maintain distribution system maps, thus eliminating the need for maintaining two separate graphics software systems: one to perform CAD operations and one to perform facility maintenance tracking of distribution system infrastructure and GIS operations. This is particularly advantageous to utility companies since CAD software is commonly used to maintain distribution system maps.

GIS software provides a means to perform analyses on geographic areas. GIS would be a valuable tool for engineering or internal accounting purposes, and for obtaining data for use by outside entities. Capabilities of GIS systems include:

- **Distribution System Mapping** - eliminates need of manual drafting for map updates; consolidates data in one location.
- **Facilities Management** - provides computerized inventory and maintenance programs for distribution system facilities; allows link between facilities and maintenance data; improves data collection and reporting.
- **Engineering and Operations Queries and Reports** - integrates data so that information retrieval is an automated process; provides the ability to query more than one source of data within single or multiple geographic areas for the purpose of developing maintenance programs.
- **Other Features** – can provide a link between customer information, facilities management data and water company maps for geographic analyses; furnishes a potential link to existing Distributed Control Systems (DCS), distribution system computer models, and water quality analyses.

With the increasing use of information systems to collect and manage data, and the higher performance of newer computing equipment, consideration should be given to implementation of or updating to a GIS system as existing data systems become outdated or obsolete. Time and labor needed to manually maintain, update and query multiple disparate sources of information should be reduced, while facility maintenance can be better managed and system analysis for determining and prioritizing capital improvement needs can be significantly enhanced.

4.0 AMERICAN WATER'S ROLE IN REGIONALIZATION

Regionalization opportunities are evaluated to determine if a consolidated solution to water supply problems in a particular area is feasible or if management services opportunities are viable. Regionalization of water systems can often provide economies of scale, avoid duplication of facilities, and provide more effective service to customers. Water systems within a specific geographic area can regionalize to benefit from shared treatment facilities or pumping facilities. Interconnections between water systems can improve reliability and enhance the fire protection system. In the case of management services, expertise within American Water can be utilized to improve other area water supplies and benefit the State's residents.

American Water's technical capability and financial resources have led to acquisition and regionalization opportunities. In general, activities have involved acquiring water systems near an existing American Water service area, and physically consolidating the new system's distribution network into the existing American Water distribution system. In the case of remote water systems, they are operated as satellite service areas, but with management from the American Water's corporate office.

5.0 WATER RESOURCE MANAGEMENT

Water resource management has become an important part of the planning process. Water resource management refers to those activities and programs designed to protect, maintain and monitor efficient use of water resources. These measures include managing water resources from both the supply and demand side. Such activities include: meter maintenance and replacement programs, leak detection and repair, scheduled water main replacement, and drought management.

Metering provides an accurate accounting of water flowing through the system, thereby helping to determine where losses and excess usage may occur. American Water policy is to meter all customers. In some cases, commercial meters on apartment buildings or other multi-tenant facilities may have been changed over to individual meters. Also, fire services may be equipped with flow indicators. Residential service meters are replaced after a predetermined interval, based on State guidelines. Larger meters should be tested on a routine basis. On the supply side, all source of supply meters should be tested and calibrated regularly.

Water resource management through leak detection and repair results in reduced non revenue water by reducing water losses. Reducing the volume of unaccounted-for water can improve system hydraulics, reduce costs for water treatment and pumping, and in some cases can delay capacity-oriented construction. In situations where water demand exceeds supply, reducing non revenue water can result in the availability of more water for customer consumption.

Leak detection surveys should be performed on an ongoing basis. In addition, valves should be sounded for leaks as part of a valve exercise program. Hydrants should be inspected on a regular basis and tested for leaks. Customer meters should be sounded on all service calls, and whenever a curb box is relocated or raised for paving.

Replacement of aged facilities can conserve water through controlling system losses. For instance, unlined pipelines can be a source of leakage. In addition to the major main replacements recommended in this report, mains that have known leakage problems or require frequent maintenance are given priority under ongoing main replacement programs. The program concentrates on mains which are 6-inches in diameter or smaller. These mains are frequently constructed from unlined, cast, or galvanized iron.

All of these measures are aimed at a water resource management program that controls water losses, protects the sources of supply, and maintains efficient and economical delivery and usage of water resources. Continuation of these practices will assist in providing high quality service to the customer.

APPENDIX B

DETAILED COST ESTIMATES

Kentucky American Water
2013 CPS

Project A-1 Jacobson Reservoir Pump Station Improvements

Item	Description	Qty	Unit	Unit Cost	Cost
1	**General Conditions**				
1.1	General Conditions	1	LS	$118,672.64	$ 118,000
2	**Earthwork**				
2.1	Finish Grading & Seeding	1	LS	$ 3,542.88	$ 3,543
2.2	Demolition & Upgrades	1	LS	$ 60,298.80	$ 60,299
2.3	Sheeting & Shoring	1	LS	$ 110,000.00	$ 110,000
2.4	Dewatering	1	LS	$ 5,000.00	$ 5,000
2.5	Structural Excavation	1	LS	$ 5,280.00	$ 5,280
2.6	Structural Subgrade	1	LS	$ 4,924.00	$ 4,924
2.7	Structural Backfill	1	LS	$ 6,542.50	$ 6,543
2.8	Access Road	1	LS	$ 4,900.00	$ 4,900
3	**Concrete**				
3.1	Reinforcing Steel	1	LS	$ 18,000.00	$ 18,000
3.2	Concrete	1	LS	$ 12,901.00	$ 12,901
3.3	Formwork	1	LS	$ 40,782.00	$ 40,782
3.4	Finish/Patch/Cure	1	LS	$ 9,123.00	$ 9,123
3.5	Winter Head Protection	1	LS	$ 5,000.00	$ 5,000
4	**Metals**				
4.1	Miscellaneous Metals	1	LS	$ 16,532.64	$ 16,533
5	**Woods & Plastics**				
5.1	Rough Carpentry	1	LS	$ 7,031.20	$ 7,031
6	**Thermal & Moisture Protection**				
6.1	Foundation Coating	1	LS	$ 1,075.44	$ 1,075
6.2	Drywall	1	LS	$ 9,000.00	$ 9,000
6.3	Insulation	1	LS	$ 3,000.00	$ 3,000
7	**Doors & Windows**				
7.1	One New and One Replacement	1	LS	$ 8,300.00	$ 8,300
8	**Finishes**				
7.1	Paints and Coatings	1	LS	$ 11,500.00	$ 11,500
9	**Specialties**				
9.1	Signage	1	LS	$ 706.36	$ 706
10	**Equipment**				
10.1	Horizontal Pumps	1	LS	$ 74,693.76	$ 74,694
10.2	Chemical Feed/Transfer Pumps	1	LS	$ 46,903.36	$ 46,903
10.3	Chemical Tanks	1	LS	$ 29,920.00	$ 29,920
10.4	Commissioning and Startup	1	LS	$ 17,179.68	$ 17,180
11	**Special Construction**				
11.1	Instrumentation & Control	1	LS	$ 100,000.00	$ 100,000
11.2	Field Instruments	1	LS	$ 48,504.60	$ 48,505
11.3	Elecrical Control Building	1	LS	$ 61,832.64	$ 61,833
12	**Mechanical**				
12.1	Process Pipe	1	LS	$ 59,029.20	$ 59,029
12.2	Valves	1	LS	$ 33,901.00	$ 33,901
12.3	Line Stop & Valve	1	LS	$ 29,259.76	$ 29,260
12.4	Flange Packs	1	LS	$ 6,706.36	$ 6,706
12.5	Chemical Feed/Flowmeter Manhole	1	LS	$ 17,839.84	$ 17,840
12.6	Fire Suppression & HVAC Mods for Existing Pump	1	LS	$ 55,000.00	$ 55,000
12.7	HVAC	1	LS	$ 42,371.00	$ 42,371
13	**Electrical**	1	LS	$ 995,000.00	$ 995,000
13.1	Electrical Equipment	1	LS	$ 1,680.96	$ 1,681
13.2	Ductbank	0	LS	$ 210,307.86	$ -
13.3	Standby Generator (including Foundation & Earthwork)		LS	$ 200,000.00	$ 150,000
				SUBTOTAL	**$ 2,231,262**
6	**Other** (Legal, eng, admin, AFUDC, OH, permitting, etc.)			30%	$ 669,378.59
7	**Contingencies**			20%	$ 446,252.40
				TOTAL	**$ 3,300,000**

Notes:
All costs taken from construction cost estimate from Gannett Fleming/Layne Proposal Dated 9/6/12

Assumptions:

Kentucky American Water
2012 CPS
Project A-3 Richmond Road Filter Building
Replacement of existing facility

Item	Description		Qty	Unit	Unit Cost	Cost
1	**General Conditions - Replacement Filter Building**					
	1.1	General Conditions	10% of 2.1 to 2.2			$ 823,865
2	**Replacement Filter Building**					
	2.1	Construct new filter building	1	LS	$ 7,469,706.00	$ 7,469,706
	2.2	Construct filter masonry enclosed structure	1	LS	$ 768,940.00	$ 768,940
					SUBTOTAL	**$ 9,062,511**
3	**Other** (Legal, eng, admin, AFUDC, OH, permitting, etc.)				30%	$ 2,718,753
4	**Contingencies - Replacement Building**				20%	$ 2,356,253
					TOTAL	**$ 14,100,000**

Kentucky American Water
2013 CPS

Project A-4 Midway Interconnection (Leestown)

Item			Qty	Unit	Unit Cost	Cost
12" DIP, Fittings and Valves			12,600	LF	$ 40.00	$504,000
Installation			12,600	LF	$ 60.00	$756,000
Easement Acquisition/Development			-	LF	$ 7.00	$0
					Subtotal	**$1,260,000**
Other (Legal, eng, admin, AFUDC, OH, permitting, etc.) - **30%**						$378,000
Contingencies - 20%						$245,700
					TOTAL	**$1,900,000**

Kentucky American Water
2013 CPS

Project A-5 Kentucky River Station 1 - Residuals Improvements
Installation of tube settlers, TSS monitors, piping and valve modifications

Item	Description		Qty	Unit	Unit Cost	Cost
1	**General Conditions**					
	1.1	General Conditions	10% of 2.1 to 5.2			$ 493,670
2	**Aldrich Units - Retrofit w/ Tube Settlers (Residuals A)**					
	2.1	Install tube settlers in Aldrich units	10	EA	$ 452,730.00	$ 4,527,300
3	**Aldrich Units - TSS Monitors (Residuals B)**					
	3.1	Install turbidimeter with transmitter	3	EA	$ 5,847.00	$ 17,541
4	**Sludge Lagoon - Influent Piping (Residuals C)**					
	4.1	Install 30" dia surge rellief valve	1	EA	$ 10,000.00	$ 10,000
5	**Aldrich Units - Influent Valving (Residuals D)**					
	5.1	Replace existing 16" butterfly valves	10	EA	$ 31,000.00	$ 310,000
	5.2	Replace existing valve actuators	10	EA	$ 7,186.00	$ 71,860
					SUBTOTAL	**$ 7,000,000**
6	**Other** (Legal, eng, admin, AFUDC, OH, permitting, etc.)				30%	$ 2,100,000
7	**Contingencies**				15%	$ 1,365,000
					TOTAL	**$ 10,500,000**

Kentucky American Water
2013 CPS

Project A-6 Storage Tank and System Nitrification

Item	Description		Qty	Unit	Unit Cost	Cost
1	**Modeling**					
			1	EA	$ 100,000.00	$ 15,000
2	**Distribution System Flushing**					
		Aggressive Distribution System Flushing	1	EA	$ 200,000.00	$ 150,000
3	**Mixing**					
		Tank mixing technologies	1	EA	$ 100,000.00	$ 100,000
		Total:				$ 265,000
4	**Other** (Legal, eng, admin, AFUDC, OH, permitting, etc.)				30%	$ 79,500
5	**Contingencies**				15%	$ 39,750
					TOTAL	**$ 384,250**

$ 400,000

Kentucky American Water
2013 CPS

Project A-7 Kentucky River Station 1 - Chemical Storage & Feed Improvements
OPTION 1 - Installation of PAC bag system

Item	Description		Qty	Unit	Unit Cost	Cost
1	**General Conditions**					
	1.1	General Conditions	10% of 2.1 to 4.2			$ 16,798
2	**Chemical Feed Line Redundancy (Chemical A)**					
	2.1	Install 1" dia chemical line	370	LF	$ 46.42	$ 17,175
	2.2	Metering pump and appurtenances	1	EA	$ 20,805.00	$ 20,805
3	**PAC System - Bag Option (Chemical B.1)**					
	3.1	Install PAC equipment in existing facility	0	LS	$ 62,796.00	$ -
	3.2	Install platform and handrail for PAC	0	LS	$ 19,799.00	$ -
	3.3	Lift for PAC bags	0	EA	$ 6,870.00	$ -
4	**Intermediate Chlorine Feed Improvements**					
	4.1	Replace existing panels	10	EA	$ 5,000.00	$ 50,000
	4.2	Replace existing piping and rotameters	1	LS	$ 80,000.00	$ 80,000
5		**Piloting n/c**				
6		**Coagulant Storage**				
	6.1	Bldg addition w structural floor, HVAC, Lighting, Arch	1	SF	$ 500,000.00	$ 500,000
	6.2	New tank	1	EA	$ 100,000.00	$ 100,000
					SUBTOTAL	**$ 784,778**
5	**Other** (Legal, eng, admin, AFUDC, OH, permitting, etc.)				30%	$ 235,434
6	**Contingencies**				20%	$ 204,042
					TOTAL	**$ 1,200,000**

Kentucky American Water
2013 CPS

Project A-8 Kentucky River Station 1 - Pump Replacements
Replace 3 existing pumps and install new VFD's on existing pumps

Item	Description		Qty	Unit	Unit Cost	Cost
1	**General Conditions**					
	1.1	General Conditions	10% of 2.1 to 5.2			$ 134,008
2	**High Service Pump Replacement**					
	2.1	Replace #10: vertical turbine - 8 MGD, 700 HP	1	EA	$ 435,000.00	$ 435,000
	2.2	Replace #11: vertical turbine - 8 MGD, 700 HP	1	EA	$ 435,000.00	$ 435,000
	2.3	Replace #15: vertical turbine - 10 MGD, 900 HP	1	EA	$ 435,000.00	$ 435,000
3	**Variable Frequency Drives**					
	3.1	Install VFD on raw water vertical turbine pumps	3	EA	$ 5,847.00	$ 17,541
	3.2	Install VFD on high service vertical turbine pumps	3	EA	$ 5,847.00	$ 17,541
	3.3	Install VFD on transfer pump	1	EA	$ 5,847.00	$ 5,847
					SUBTOTAL	**$ 1,479,937**
6	**Other** (Legal, eng, admin, AFUDC, OH, permitting, etc.)				30%	$ 443,981
7	**Contingencies**				20%	$ 384,784
					TOTAL	**$ 2,300,000**

Kentucky American Water
2013 CPS

Project **A-9** **Georgetown Bypass and US 25 Area**
12-Inch on Lisle Road

Item			**Qty**	**Unit**	**Unit Cost**	**Cost**
12" DIP			15,700	LF	$ 30.00	$471,000
Installation			15,700	LF	$ 70.00	$1,177,500
Easement Acquisition/Development			-	LF	$ 12.00	$0
					Subtotal	**$1,648,500**
Other (Legal, eng, admin, AFUDC, OH, permitting, etc.) - **30%**						$494,550
Contingencies - 20%						$321,458
					TOTAL	**$2,500,000**

Kentucky American Water
2013 CPS

Project B-1 Evaluation of Jacobson Reservoir Safe Yield

Item	Description		Qty	Unit	Unit Cost	Cost
1	Element					
			1	Ea	$ -	$ -
			1	Ea	$ -	$ -
				Equip Cost	$ -	$ -
			25%	Equip Cost	$ -	$ -
					SUBTOTAL	$ -
2	Engineering, legal, administration, construction management and overhead					$ -
3	Contingencies					$ -
					TOTAL	**$ -**
						$ 65,000

(1) Based on treating half of the flow through the IX unit and blending to achieve compliance

Kentucky American Water
2013 CPS

Project **B-2** **Briar Hill Tank Area**

Item	Qty	Unit	Unit Cost	Cost
12" DIP	38,900	LF	$ 30.00	$1,167,000
8" DIP	4,000	LF	$ 25.00	$100,000
6" DIP	8,300	LF	$ 20.00	$166,000
Installation	51,200	LF	$ 50.00	$2,560,000
Easement Acquisition/Development	-	LF	$ 12.00	$0
			Subtotal	$3,993,000
Other (Legal, eng, admin, AFUDC, OH, permitting, etc.) - **30%**				$1,197,900
Contingencies - 20%				$778,635
			TOTAL	**$6,000,000**

Kentucky American Water
2013 CPS

Project **B-3** **North of Sadieville**

Item	**Qty**	**Unit**	**Unit Cost**	**Cost**
8" DIP	19,000	LF	$ 25.00	$475,000
Installation	19,000	LF	$ 50.00	$950,000
Easement Acquisition/Development	-	LF	$ 12.00	$0
			Subtotal	$1,425,000
Other (Legal, eng, admin, AFUDC, OH, permitting, etc.) - **30%**				$427,500
Contingencies - 20%				$277,875
			TOTAL	**$2,100,000**

Kentucky American Water
2013 CPS

Project B-4 Areas Along KY-22

Item	Qty	Unit	Unit Cost	Cost
6" DIP	19,150	LF	$ 20.00	$383,000
Installation	19,150	LF	$ 50.00	$957,500
Easement Acquisition/Development	-	LF	$ 12.00	$0
			Subtotal	$1,340,500
Other (Legal, eng, admin, AFUDC, OH, permitting, etc.) - **30%**				$402,150
Contingencies - 20%				$261,398
			TOTAL	**$2,000,000**

Kentucky American Water
2013 CPS

Project **B-5** **State Highway Upgrades**
Georgetown Road and Newtown Pike

Item	**Qty**	**Unit**	**Unit Cost**	**Cost**
24" DIP	29,600	LF	$ 70.00	$2,072,000
Installation	29,600	LF	$ 125.00	$3,700,000
Easement Acquisition/Development	-	LF	$ 12.00	$0
			Subtotal	$5,772,000
Other (Legal, eng, admin, AFUDC, OH, permitting, etc.) - **30%**				$1,731,600
Contingencies - 20%				$1,125,540
			TOTAL	**$8,600,000**

Kentucky American Water
2013 CPS

Recurring Project
Increase Replacement Rate of Annual Main Replacement Program
Design and Permitting: On-going
Construction: On-going

Item	Description		Qty	Unit	Unit Cost	Cost
1	**Piping**					
		1% main replaced per year	1	LS	$7,336,659	$7,337,000
					SUBTOTAL	$7,337,000
		Other (legal, engineering, etc.) @			20%	$1,468,000
		Contingencies @			15%	$1,101,000
					TOTAL	**$9,910,000**

APPENDIX C

SITE AND PLANT PHOTOS

Distribution Pump

24” Raw Water Ball Valve

24” Raw Water Ball Valve


24” Raw Water Ball Valve


Ammonia

Ammonia Calibrator

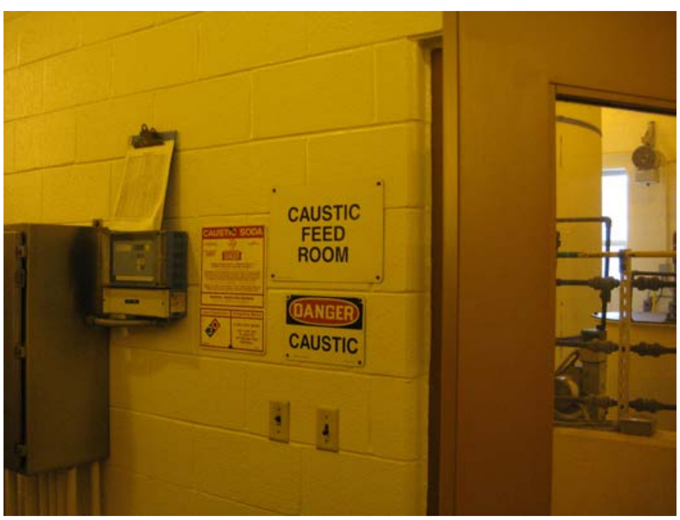
Ammoniator Room

Ammoniator Room

Backwash Tanks (2)


Ball Valve Center Ring


Ball Valve Center Ring



Caustic

Caustic

Chemical Building

Chemical Feed Trench

Chlorinators

Chlorine

Chlorine

Chlorine

Chlorine

Chlorine Room Vacuum


Clearwell (outside)


Finished Water

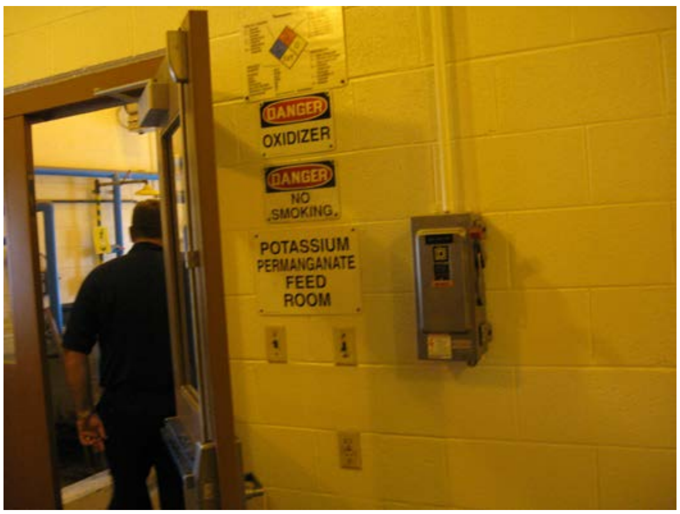

From Raw Water Pump Platform

High Service Pumps


High Service Pumps


Hydrotreator

Hydrotreator



$KMnO_4$

$KMnO_4$

$KMnO_4$

$KMnO_4$

PAC

PAC Feed

PACl “Cedar Floc”

Rapid Mix


Rapid Mix


Rapid Mix


Rapid Mix Air Relief

Raw Water Ball Valves


Raw Water Discharge Piping


Raw Water Lines


Raw Water Pump Motors


Raw Water Pump Shafts


Raw Water Pump Station Electric Building

Raw Water Switchgear

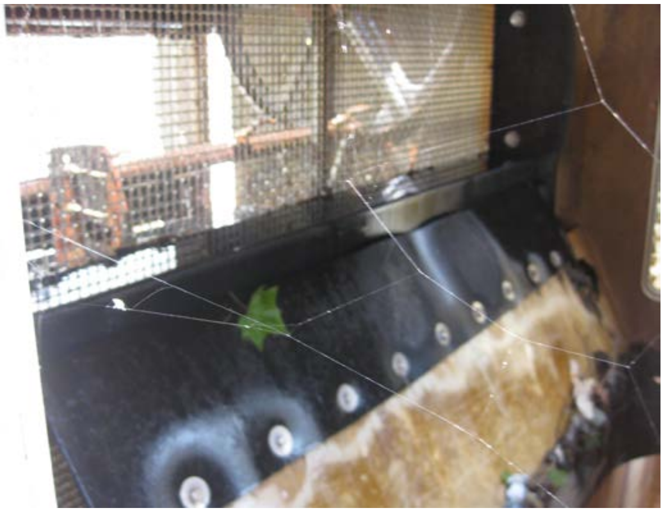

Raw Water Transfer Pumps

Raw Water Transfer Pumps

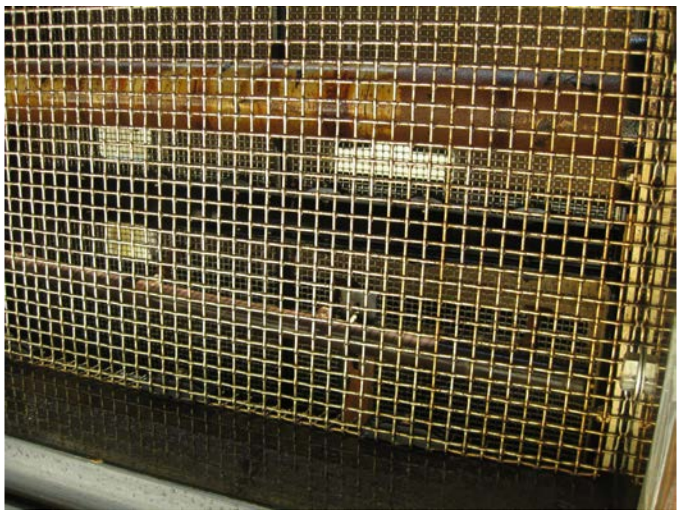
Raw Water Transfer Pumps

Raw Water Transfer Pumps

Raw Water Traveling Screen



Raw Water Traveling Screen



Raw Water Traveling Screen

Ring Foundation Hydrotreator


Ring Foundation Hydrotreator

Sodium Thiosulfate


Sodium Thiosulfate


Spare Motor


Top of Tram


Top of Tram

Tram and Intake

Tram going down

Tram going down

Valve House #5

Valve House #5


Valve House #5

KRS1

Valve Vault Clearwell to “tank over the hill”

Valve Vault Clearwell to “tank over the hill”

Valve Vault Rapid Mix to Hydrotreator

Valve Vault Rapid Mix to Hydrotreator

Valve Vault Rapid Mix to Hydrotreator

Vibration Monitoring


Waste Water Basins


Waste Water Basins & Sludge Lagoons


Waste Water Basins & Sludge Lagoons


Waste Water Basins & Sludge Lagoons

Zinc Orthophosphate

Zinc Orthophosphate


Distribution Pump


24” Raw Water Ball Valve


24” Raw Water Ball Valve

24” Raw Water Ball Valve

Ammonia

Ammonia Calibrator

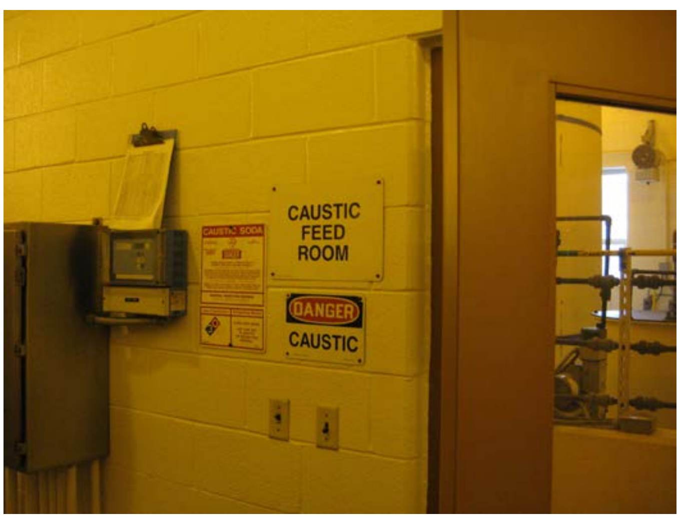

Ammoniator Room

Ammoniator Room


Backwash Tanks (2)

Ball Valve Center Ring

Ball Valve Center Ring



Caustic

Caustic

Chemical Building


Chemical Feed Trench

Chlorinators

Chlorine

Chlorine

Chlorine


Chlorine

Chlorine Room Vacuum

Clearwell (outside)

Finished Water

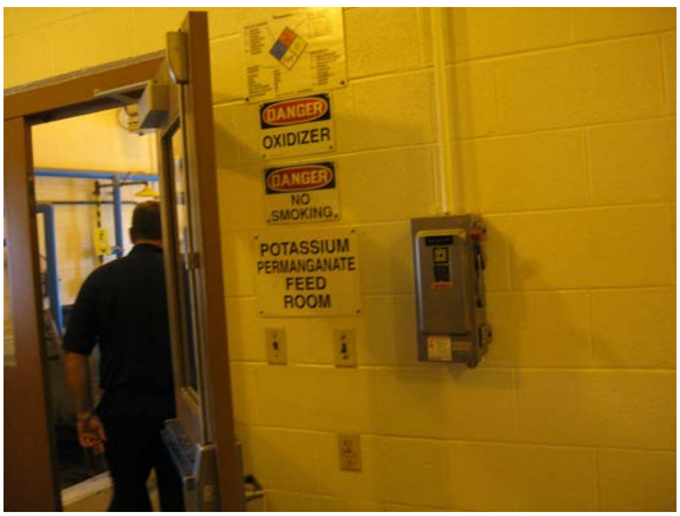
From Raw Water Pump Platform

High Service Pumps


High Service Pumps


Hydrotreator


Hydrotreator



$KMnO_4$


$KMnO_4$

$KMnO_4$

$KMnO_4$

PAC

KRS1

PAC Feed

PACl “Cedar Floc”

Rapid Mix

Rapid Mix

Rapid Mix

Rapid Mix Air Relief

Raw Water Ball Valves


Raw Water Discharge Piping


Raw Water Lines

Raw Water Pump Motors

Raw Water Pump Shafts

Raw Water Pump Station Electric Building

Raw Water Switchgear

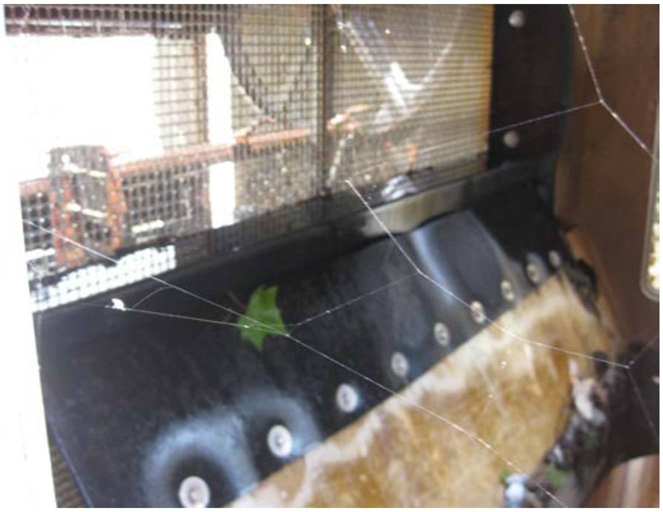

Raw Water Transfer Pumps

Raw Water Transfer Pumps

Raw Water Transfer Pumps

Raw Water Transfer Pumps

Raw Water Traveling Screen



Raw Water Traveling Screen

Raw Water Traveling Screen

Ring Foundation Hydrotreator


Ring Foundation Hydrotreator

Sodium Thiosulfate


Sodium Thiosulfate


Spare Motor


Top of Tram


Top of Tram

Tram and Intake

Tram going down

Tram going down

Valve House #5


Valve House #5


Valve House #5

Valve Vault Clearwell to "tank over the hill"

Valve Vault Clearwell to "tank over the hill"

Valve Vault Rapid Mix to Hydrotreator


Valve Vault Rapid Mix to Hydrotreator


Valve Vault Rapid Mix to Hydrotreator


Vibration Monitoring

Waste Water Basins

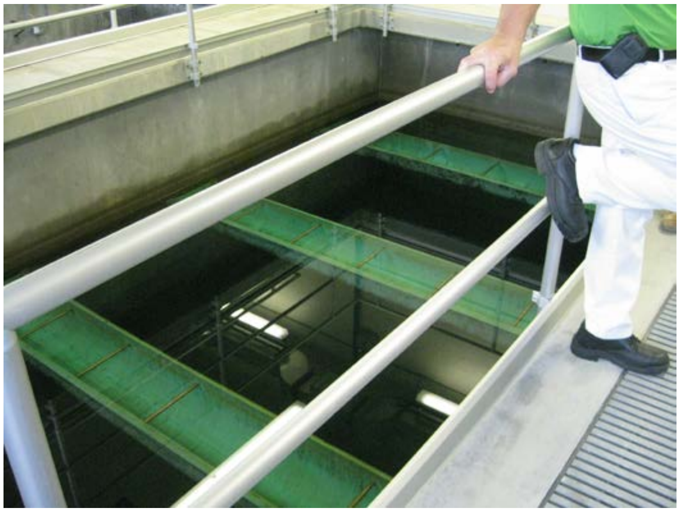
Waste Water Basins & Sludge Lagoons

Waste Water Basins & Sludge Lagoons


Waste Water Basins & Sludge Lagoons

Zinc Orthophosphate

Zinc Orthophosphate


Backwash Troughs


Chemical Feed Piping


Chlorine Scrubber

KRS2 Plant

Electrical Room


Electrical Room


Empty for future process (UV)


Filter Basins

Filter Gallery

Filter Gallery

KRS2 Plant

Filter Gallery

Filter Influent Valve

Floc Basin

KRS2 Plant


Floc Basin


Floc Empty


Floc Full

High Service Pumps


High Service Pumps


Main Control Room

KRS2 Plant

PACl Area

PACl Pipes

PACl Piping

KRS2 Plant


PACl Storage


Plate Settler Basin Sludge Rakes


Plate Settlers Empty

Plate Settlers Full

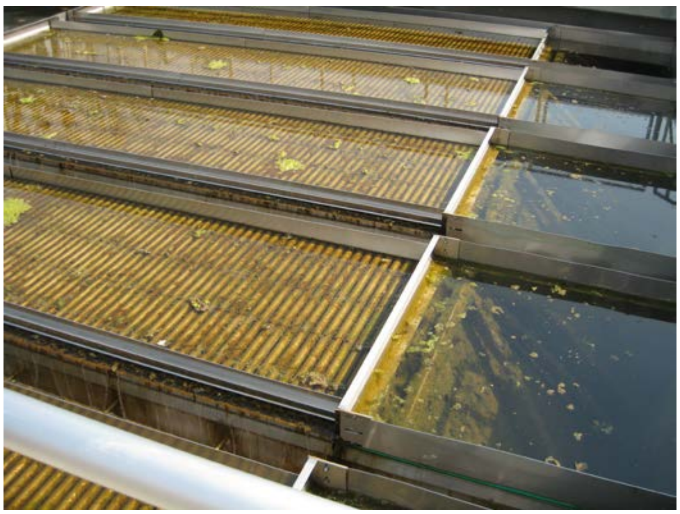
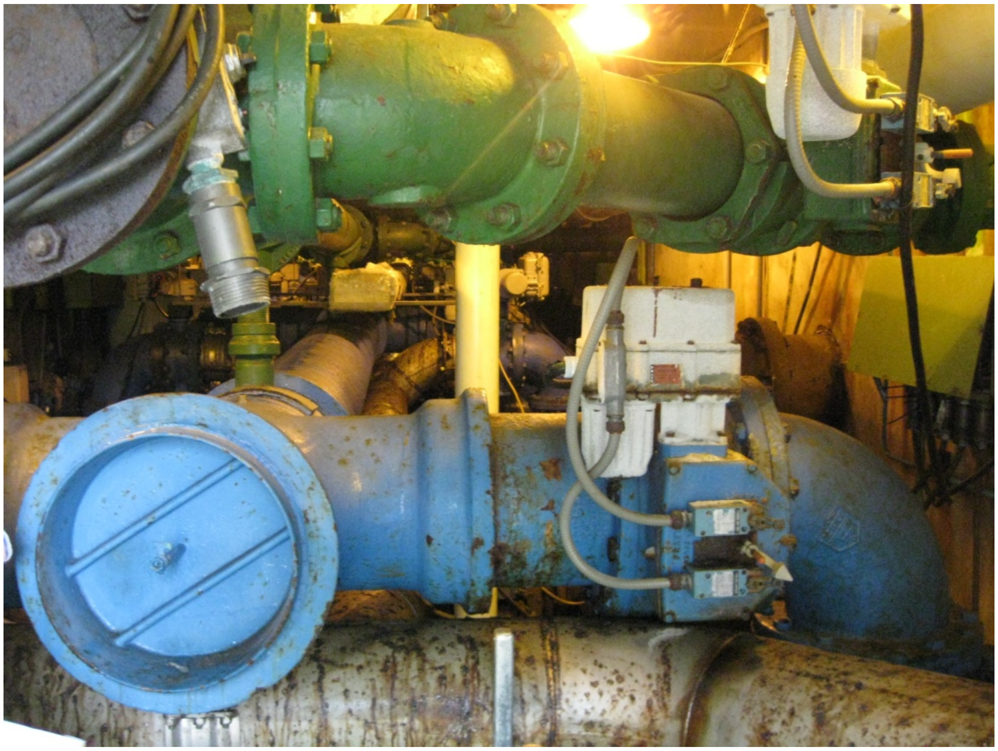


Plate Settlers Top

Raw Water Mag Meter


Supernatant Discharge


Supernatant Discharge



Richmond Road Station – Filter Piping Gallery

Richmond Road Station – Washwater Holding Tanks

Richmond Road Station - Filter Controls

Richmond Road Station – Filter Press Building

Richmond Road Station – Influent to Washwater Tank

Richmond Road Station – Sedimentation Basins

Richmond Road Station – Clearwell #2

Jacobson Reservoirs – Aeration System

Jacobson Reservoir – Pump Station to Richmond Road Station

APPENDIX D

PUMP EFFICIENCY AND RECOMMENDATIONS TECHNICAL MEMORDANDUM

AMERICAN WATER

MEMORANDUM

To: Lance Williams

From: Doug Potts

Copy: Michael Moler, Justin Sensabaugh, Mitzi Combs, Kevin Kruchinski, Jason Hurt, Wesley Jacobs, Peter Keenan, Mike McDonald

Date: May 11, 2012

Subject: Evaluation of Pump Efficiency Improvement Opportunities for KYAW

American Water has established goals for reducing its energy usage index (EUI) over the next several years. EUI is a measure of the amount of energy that is required to produce and deliver water to customers, and is strongly affected by the topography over which the water is distributed, as well as the condition and efficiency of the pumping equipment being used. Since the topography of a distribution system cannot be changed, the primary way to decrease EUI is by improving the condition and efficiency of the pumps.

Kentucky American Water (KAW) owns and operates three water treatment facilities that supply water to the City of Lexington and the surrounding region. KAW requested that American Water's COE Engineering group undertake an evaluation of the pumps at the Kentucky River Station No. 1 (KRS1) and Richmond Road Station (RRS) to identify the best options for increasing the efficiency of pumping equipment and lowering the EUI of these facilities. Kentucky River Station No. 2 (KRS2) was recently constructed and was therefore not included as part of this evaluation.

Table 1 provides a summary of relevant information about the existing major pumping equipment at the two facilities, including recent wire-to-water efficiency data and an estimate of the annual pump runtime and flow contribution for each pumping unit over recent years. Wire-to-water efficiency data are not yet available for the transfer pumps that convey raw water from the KRS1 raw water pump station (RWPS) to RRS, so those pumps were excluded from this evaluation.

As can be seen, the six raw water pumps at KRS1 all exhibit wire-to-water efficiencies between 69 and 74 percent, and each unit operated between about 20 and 50 percent of the time. In recent years, the highest efficiency unit has been operated slightly more than the other units, but runtime has been fairly well distributed among all six pumps. Based on the runtime hours and operational data, KAW typically operates two pumps continuously at the KRS1 RWPS, although up to five pumps may be required during peak demand periods.

At the KRS1 high service pump station (HSPS), between two and three pumps are normally in operation, including at least one 8 MGD unit and one 10 MGD unit. As can be seen, three of the high service pumps at KRS1 have markedly poorer wire-to-water efficiencies than the remaining three units, and all three lower efficiency units operated for significant periods of time over the past year.

Jacobsen Reservoir (JRPS) is equipped with three pumps, and runtime hours were fairly evenly distributed over the past year. Typically the reservoir pumps only operate about four months out of the year, but the efficiency of all three is generally poor.

At the RRS HSPS, two of four pumps typically operate. As shown in Table 1, the wire-to-water efficiency of Pumps 6 and 7 is significantly greater than Pumps 8 and 10, but these higher efficiency pumps operate far more frequently than the low efficiency units. In fact, the least efficient unit (Pump 10) was not operated at all last year.

Table 2 identifies which pumps are proposed for rehabilitation at each pump station, along with a budgetary estimate of the anticipated cost for rehabilitation. Consideration was given to the existing wire-to-water efficiency of each pump to develop this prioritized list for rehabilitation or replacement. In each case, the pumps with the lowest current wire-to-water efficiency rating are proposed for rehab because that will maximize overall efficiency and minimize the aggregate EUI for the station when the remaining pumps are also running. It was assumed that the wire-to-water efficiency that would be achieved by rehabilitating or replacing a pump would be about 82 percent. However, since the exact condition and repair needs cannot be known until the pumps are disassembled in the shop, cost allowances were based on the assumption that most of the pump elements would be replaced. In most cases, the motors will also be replaced, depending upon the age and condition. If actual repair requirements are relatively minor, the cost to rehabilitate the pumps should be substantially less than the dollar amounts budgeted.

Figure 1 shows the projected energy savings that would be expected as a result of rehabilitating two or three pumps in each of the pump stations. However, the energy savings will be proportional to the runtime hours or percent of the time that the rehabbed pumps are in service. Figure 1 shows that as the runtime for the rehabbed units increases, the total kilowatt-hours (kWh) saved also increases. This is because the higher efficiency rehabbed pumps will be responsible for more of the total production from the facility.

Figure 2 shows this same information expressed in terms of energy savings relative to the average annual power consumed by KAW each year. If KAW undertakes all nine pump rehab projects proposed in Table 2, a net energy savings of over 5 percent should be achieved even if the runtime on the pumps is evenly distributed between the rehabbed and non-rehabbed pumps. American Water has established a goal of reducing its EUI by 8 percent by 2016. In order for KAW to achieve this goal entirely from the proposed pump efficiency projects, production load borne by the non-rehabbed pumps would need to be reduced and the rehabbed pumps would have to operate about 75 percent of the time in the future.

Figure 3 shows how the proposed pump efficiency projects will impact KAW's overall EUI value, which averaged approximately 4.32 between 2008 and 2010. If all of the proposed pump improvements are made, the state-wide EUI would be projected to decline to a value between 4.08 and 3.93, depending on how much the rehabbed units operate. The percent energy savings exhibited in Figure 2 also apply to the EUI reductions shown in Figure 3. In other words, to achieve AW's 8 percent reduction goal for EUI, the rehabbed pumps would have to operate about 75 percent of the time in the future.

Rehabilitating pumping equipment often requires a significant capital investment. The greatest return on investment or "payback" is achieved by only rehabilitating the number of pumps that operate most frequently, and then maximizing their runtime. Figure 4 graphs the payback duration for each pump station as a function of the future runtime for the rehabbed pumps. Operating cost savings were based on an average power cost of 6.5 cents per kilowatt-hour, which combines both the variable (commodity) charges and the fixed demand charges. Presumably, demand charges will be reduced if pump efficiency improvements are successful.

As indicated earlier, maximizing the runtime of the rehabbed pumps will reduce the payback period. However, good operating practice generally dictates that pump runtimes be evenly distributed to avoid excessive wear on individual units. In addition, consideration should be given to the impact of the upgrades KAW's overall EUI reduction goals. Table 2 includes a prioritization ranking of the pumps that considers both the investment payback time, as well as the potential impact on energy reduction.

Figures 1 and 2 showed that rehabbing three of the pumps at the KRS1 HSPS would produce the greatest impact on energy reduction among the improvements at the four pump stations. In addition, Figure 4 shows that the rehabs would have the second lowest payback of the four pump stations evaluated. Therefore, the KRS1 HSPS pumps were ranked as the highest priority.

Rehabilitation of two of the KRS1 RWPS pumps also would have a significant impact on energy savings, but because of the size of the pumps and accessibility of the station, these pumps will be very costly to rehabilitate and payback would be considerably longer than the other stations. As a result, the KRS1 RWPS pumps received the lowest ranking. Several other factors were also considered in ranking these units lower than the other stations, including:

1. The efficiencies of these pumps are still relatively good (69% - 74%) as compared to "like new" efficiency of 82%

2. As part of this study and the Comprehensive Planning Study (CPS) currently being prepared, the installation of a VFD at the RWPS is being evaluated. The projected efficiency gains due to a VFD will likely far exceed any efficiency gains due to pump rehabilitation.

3. In lieu of a VFD, consideration is being given to downsizing one of the raw water pumps by trimming impellers or removing a pump bowl(s) to create more

incremental flow rate options. Should this option be selected, pump rehabilitation may be performed on the modified pump at that time.

The JRPS pumps operate less frequently and at a lower head and consume far less energy than the RRS HSPS pumps. However, rehabbing the JRPS pumps offers a greater opportunity for decreasing power consumption because the JRPS pumps are worn and the two RRS HSPS pumps that delivered most of the production capacity were relatively high efficiency units. Therefore, the JRPS pumps have a significantly faster payback are recommended as higher priority than the RRS HSPS.

CONCLUSION AND RECOMMENDATIONS

Based on the analysis included herein, it is recommended that KAW proceed with a program to rehabilitate or replace the three designated pumps at KRS1 HSPS. Depending on operational constraints, KAW may also want to proceed with rehabilitation of two of the JRPS units.

Finally, as you are aware, the discharge pressures from both the KRS1 and RRS plants are significantly lower than the historical average. The RRS discharge pressure has been reduced from 90 psi to 70 psi, and the KRS1 discharge pressure has been reduced from 160 psi to 120 psi. This is primarily due to distribution system improvements completed in recent years and to shifting of some of the system delivery load to the new KRS2 plant. Therefore, any pump rehab work should include revising the pump's design points by trimming the impellers, or, if the pump is to be replaced, selecting design points that are based on current system conditions.

TABLE 1
SUMMARY OF EXISTING PUMPING EQUIPMENT

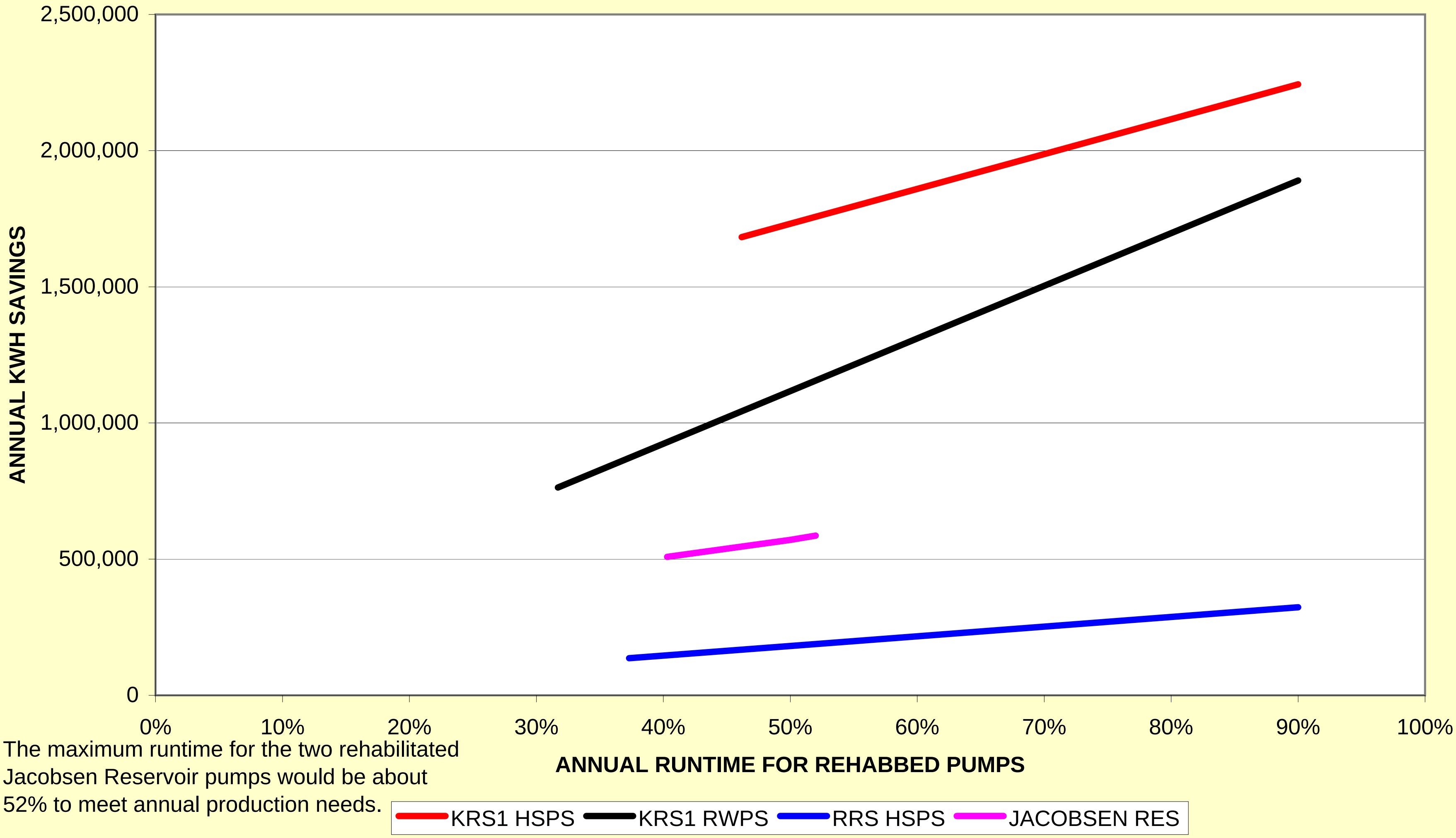
Pump No.	Pump Description	Pump Type	Design Flowrate (MGD)	Nameplate Motor Size (HP)	Wire-To-Water Efficiency	Historic Annual Runtime (Hrs)	Percent of Time in Service	Estimated Annual Pumpage (MGD)	Estimated % of Annual Volume Delivered	Estimated Current Pump EUI (MWH/MG)
KENTUCKY RIVER STATION NO. 1										
1	Raw Water	VTP	14.4	1,250	69	2,997	34%	4.9	17%	1.69
2	Raw Water	VTP	14.4	1,250	74	4,656	53%	8.2	29%	1.58
3	Raw Water	VTP	14.4	1,250	72	2,729	31%	4.7	16%	1.62
4	Raw Water	VTP	14.4	1,250	72	1,738	20%	3.0	10%	1.62
5	Raw Water	VTP	14.4	1,250	73	2,858	33%	5.0	18%	1.60
6	Raw Water	VTP	14.4	1,250	71	1,630	19%	2.8	10%	1.64
	SUBTOTAL		**86.4**	**7,500**	**N/A**	**16,608**	**N/A**	**28.5**	**100%**	**1.62**
8	Transfer	HSC	18.1	1,000	N/A	1,106	13%	N/A	N/A	N/A
9	Transfer	HSC	18.1	1,000	N/A	2,132	24%	N/A	N/A	N/A
	SUBTOTAL		**36.2**	**2,000**	**N/A**	**3,238**	**N/A**	**N/A**	**N/A**	**N/A**
10	High Service	VTP	8.0	700	60	3,838	44%	2.5	14%	2.09
11	High Service	VTP	8.0	700	61	5,617	64%	3.5	19%	2.06
12	High Service	HSC	8.1	700	70	4,258	49%	3.5	19%	1.79
13	High Service	HSC	10.0	800	73	2,599	30%	2.4	13%	1.72
14	High Service	VTP	10.0	800	75	5,861	67%	4.5	25%	1.67
15	High Service	VTP	10.0	900	56	2,611	30%	1.9	11%	2.24
	SUBTOTAL		**54.1**	**4,600**	**N/A**	**24,783**	**N/A**	**18.4**	**100%**	**1.89**
RICHMOND ROAD STATION										
1/C	Jacobson Resv.	HSC	4.0	100	63	3,100	35%	1.4	20%	0.30
2/H	Jacobson Resv.	HSC	4.0	100	58	3,300	38%	1.5	20%	0.34
3/F	Jacobson Resv.	HSC	12.0	400	57	3,800	43%	5.2	60%	0.46
	SUBTOTAL		**20.0**	**600**	**N/A**	**10,200**	**N/A**	**8.1**	**100%**	**0.40**
6	High Service	HSC	6.5	250	70	1,695	19%	1.2	12%	0.86
7	High Service	HSC	12.0	500	73	6,473	74%	8.7	85%	0.85
8	High Service	HSC	4.0	300	63	594	7%	0.3	3%	0.95
10	High Service	HSC	5.5	250	54	0	0%	0.0	0%	1.11
	SUBTOTAL		**28.0**	**1,300**	**N/A**	**8,762**	**N/A**	**10.2**	**100%**	**0.85**

TABLE 2
SUMMARY OF PROPOSED PUMPS TO BE REHABILITATED/REPLACED

Priority	Pump No.	Pump Description	Pump Type	Design Flowrate (MGD)	Nameplate Motor Size (HP)	Wire-To-Water Efficiency	Budget Rehab Cost	Minimum kWh Reduction	kWh Reduction @ 70% Runtime	Simple Payback Minimum	Simple Payback @ 70% Runtime
					KENTUCKY RIVER STATION NO. 1						
8	**1**	Raw Water	VTP	14.4	1,250	69	$370,000				
9	**6**	Raw Water	VTP	14.4	1,250	71	$370,000				
		SUBTOTAL		**28.8**	**2,500**	**N/A**	**$740,000**	**763,000**	**1,504,000**	**14.9**	**7.6**
1	**10**	High Service	VTP	8.0	700	60	$170,000				
2	**11**	High Service	VTP	8.0	700	61	$170,000				
3	**15**	High Service	VTP	10.0	900	56	$210,000				
		SUBTOTAL		**54.1**	**4,600**	**N/A**	**$550,000**	**1,682,000**	**1,987,000**	**5.0**	**4.3**
					RICHMOND ROAD STATION						
3	**2/H**	Jacobson Resv.	HSC	4.0	100	58	$30,000				
4	**3/F**	Jacobson Resv.	HSC	12.0	400	57	$60,000				
		SUBTOTAL		**16.0**	**500**	**N/A**	**$90,000**	**508,000**	**571,000**	**2.7**	**2.4**
5	**8**	High Service	HSC	4.0	300	63	$50,000				
6	**10**	High Service	HSC	5.5	250	54	$50,000				
		SUBTOTAL		**9.5**	**550**	**N/A**	**$100,000**	**136,000**	**252,000**	**11.3**	**6.1**
		STATEWIDE					**$1,480,000**	**3,089,000**	**4,314,000**	**7.4**	**5.3**

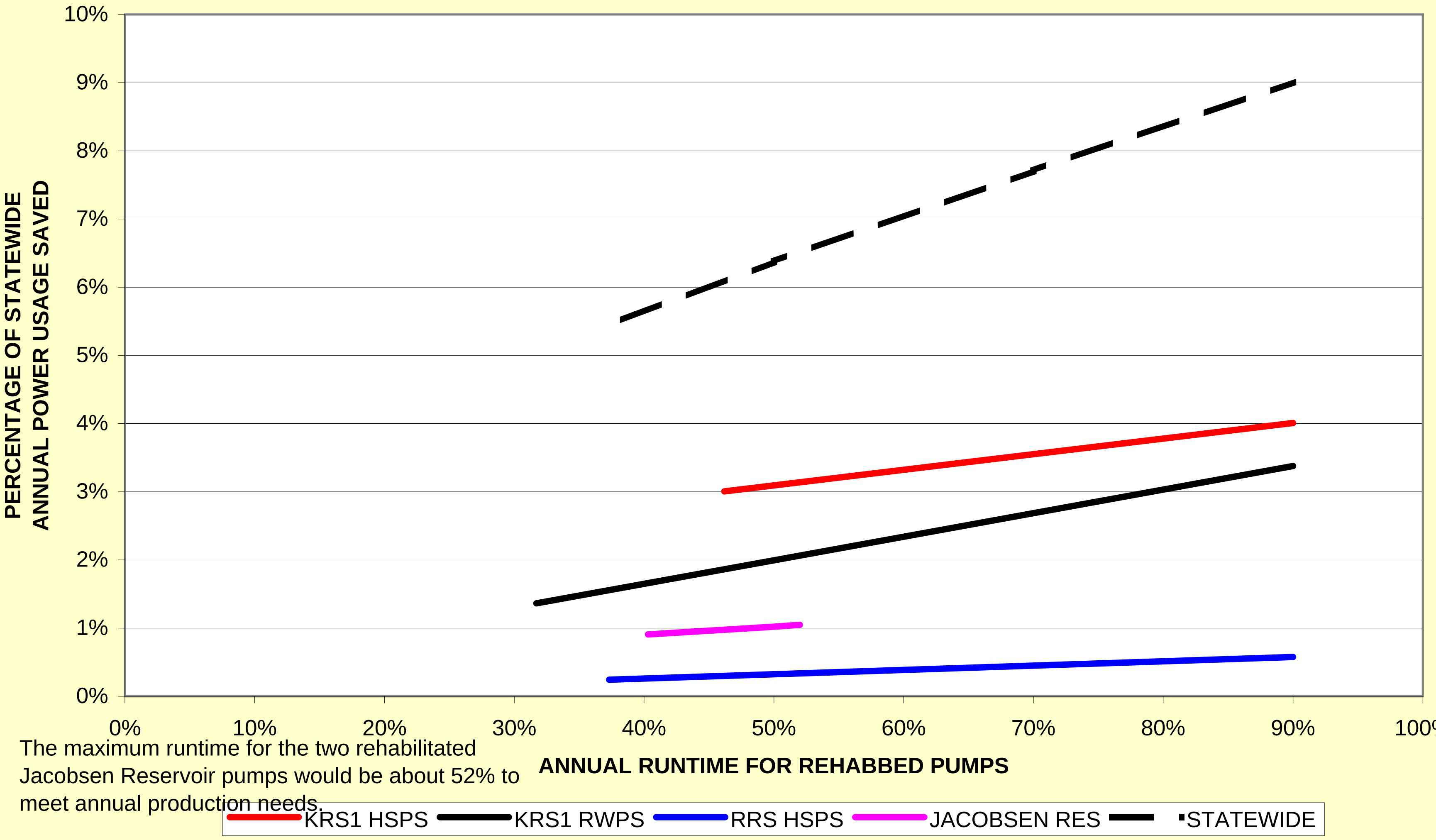
KAW Average Power Cost = $0.065 /kWh

FIGURE 1
KENTUCKY AMERICAN WATER
ANNUAL KWH REDUCTION BASED ON RUNTIME OF REHABBED PUMPS



The maximum runtime for the two rehabilitated Jacobsen Reservoir pumps would be about 52% to meet annual production needs.

FIGURE 2
KENTUCKY AMERICAN WATER
PERCENT OF ANNUAL STATEWIDE POWER SAVED



The maximum runtime for the two rehabilitated Jacobsen Reservoir pumps would be about 52% to meet annual production needs.

FIGURE 3
KENTUCKY AMERICAN WATER
EUI IMPACT FROM PROPOSED PUMP REHABILITATION

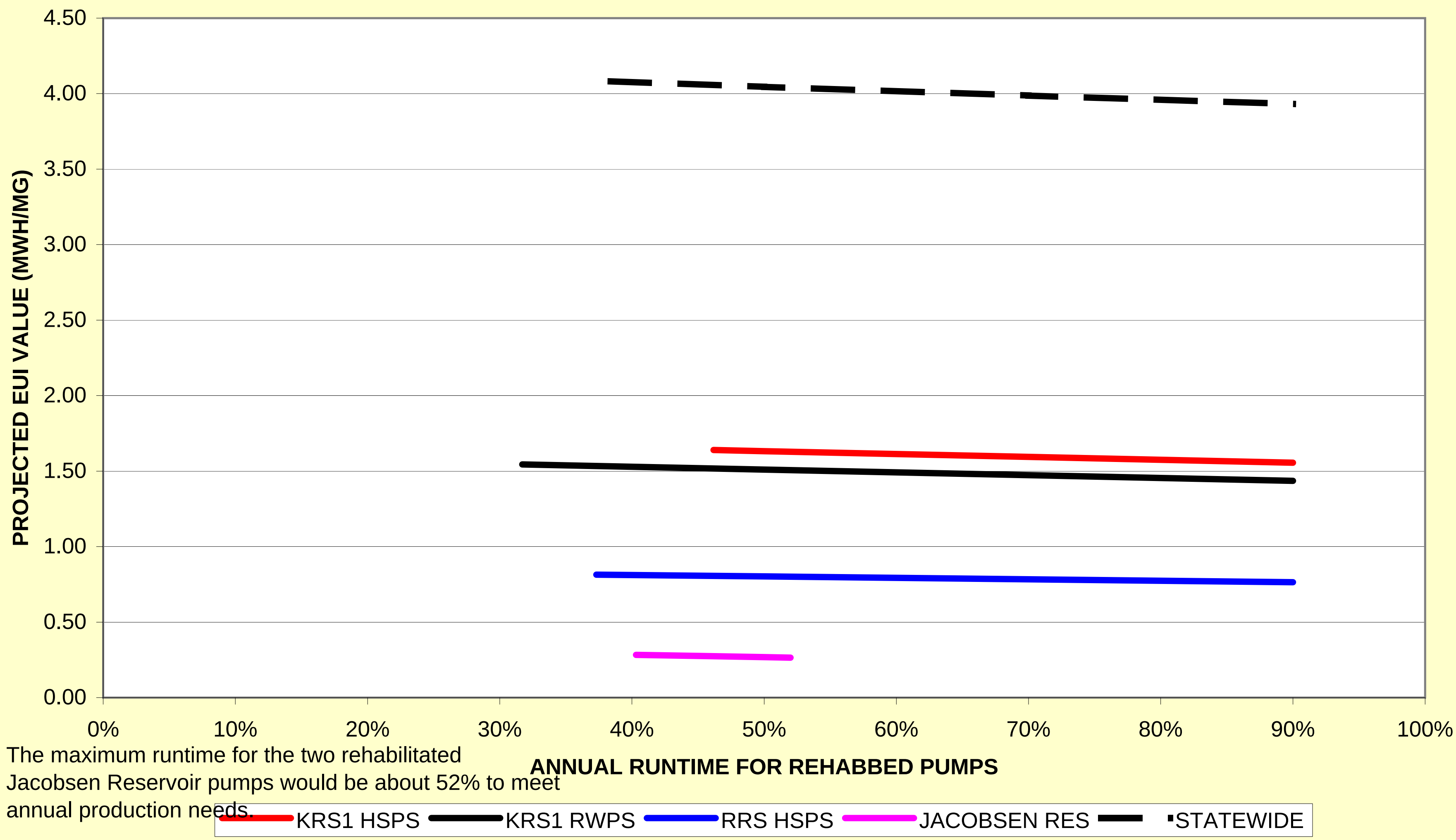
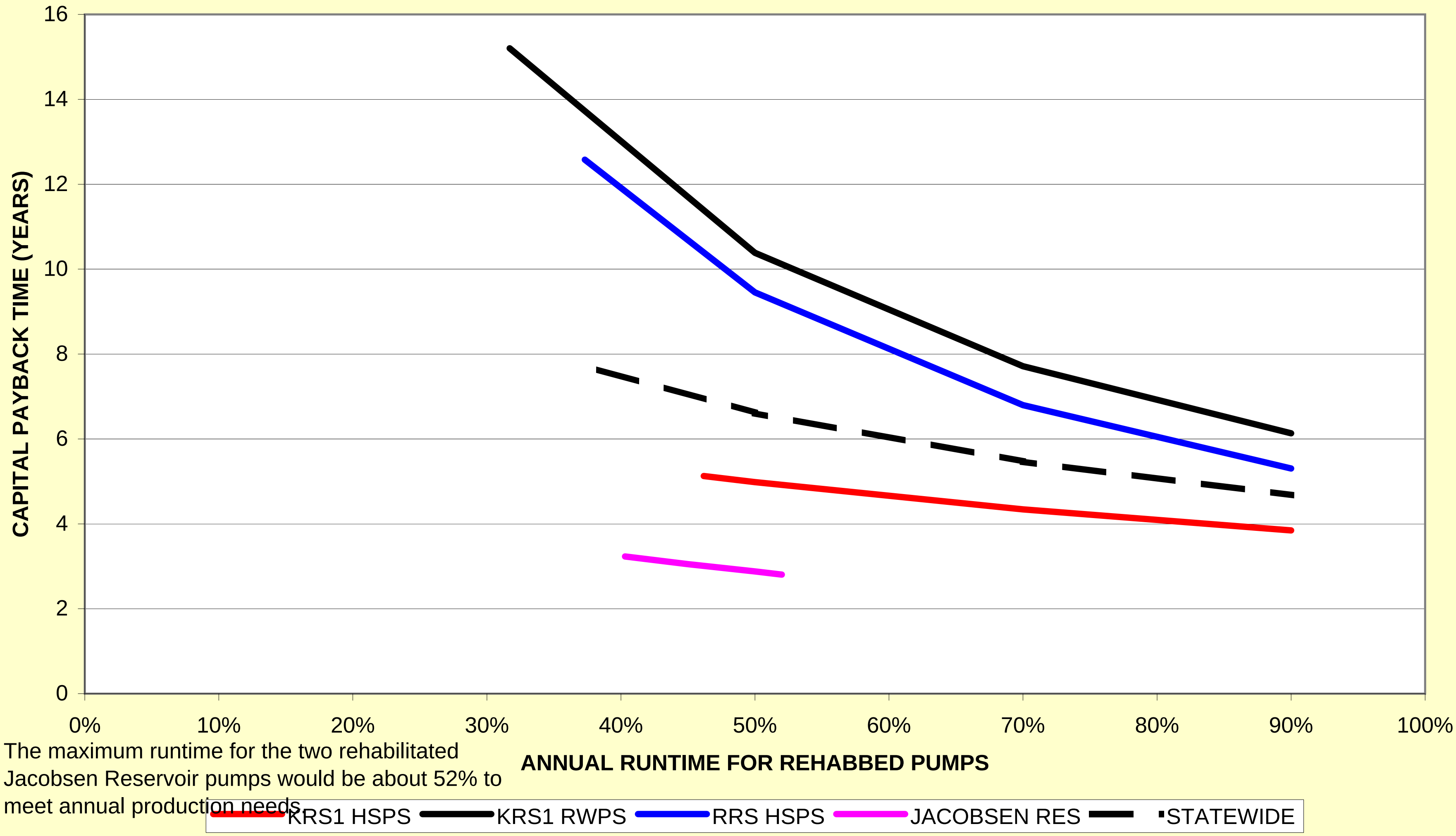


FIGURE 4
KENTUCKY AMERICAN WATER
PUMP REHABILITATION SIMPLE PAYBACK ANALYSIS



APPENDIX E

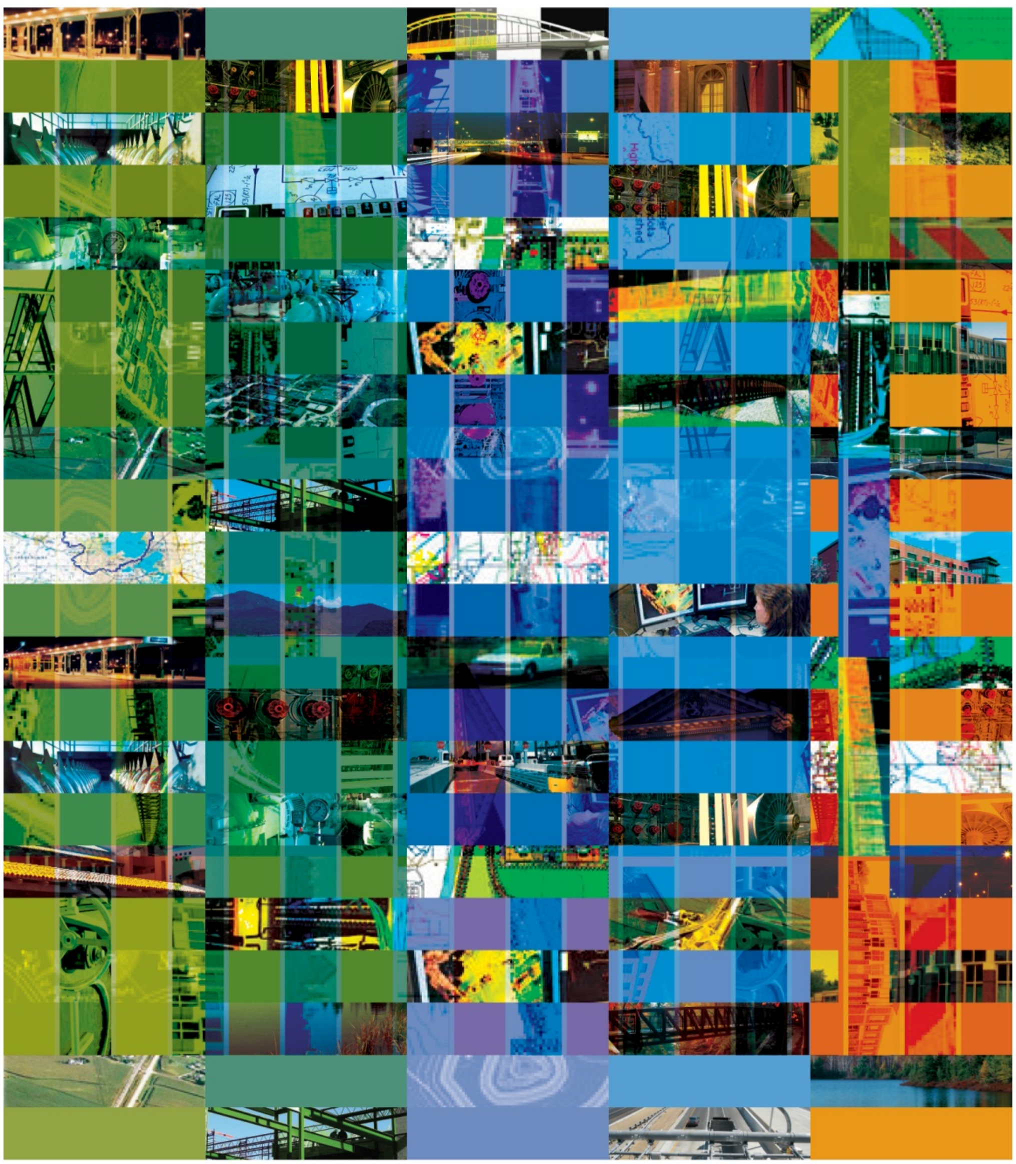
KAW HYDRAULIC ANALYSIS FOR COMPREHENSIVE PLANNING STUDY

Professional

Engineering

Services

Hydraulic Analysis for Comprehensive Planning Study



Report

Kentucky American Water

March 2012

STRAND ASSOCIATES®

STRAND ASSOCIATES®

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Waterfront Plaza
325 West Main Street, Suite 710
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March 19, 2012

Mr. Jason Hurt, P.E.
Kentucky American Water
2300 Richmond Road
Lexington, KY 40502

Re: Hydraulic Analysis for Comprehensive Planning Study

Dear Mr. Hurt:

Enclosed are three copies of the final Hydraulic Analysis for Comprehensive Planning Study.

Please call with any questions.

Sincerely,

STRAND ASSOCIATES, INC.®

Chris Keil, P.E.

Enclosure: Report

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Report for

Kentucky American Water

Hydraulic Analysis for Comprehensive Planning Study

STATE OF KENTUCKY
CHRISTOPHER J.
KEIL
23147
LICENSED
PROFESSIONAL ENGINEER

3/19/12

Prepared by:

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Louisville, KY 40202
www.strand.com

March 2012

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APPENDICES

SECTION 1
INTRODUCTION

1.01 PURPOSE

The purpose of this report is to summarize findings and provide recommendations for distribution system capital and operational improvements to Kentucky-American Water (KAW). Findings and recommendations will be made for KAW's Central Division (Central) located in Fayette County and surrounding areas and the Northern Division (Northern) located in Owen County and surrounding areas. Computerized distribution system models were utilized to simulate the current and projected future operating schemes for various demand scenarios and as a basis for recommended improvements.

1.02 SCOPE

The scope of this project included the following.

A. Model Updates

Update current available models for the Central and Northern Divisions based on key system improvements that have been implemented since the models' last update.

B. Model Calibration

The Central model was reviewed for calibration utilizing supervisory control and data acquisition (SCADA) information. An initial simulation was conducted of the updated Central model and results compared to SCADA data to determine if calibration was necessary. The model was refined further based on initial results and presented to KAW staff. Revised results indicated the Central model was trending well and was considered calibrated for the purpose of this study. See Section 2.04 for details.

The Northern model was calibrated using hydrant flow test data, SCADA information, and the hydraulic calibration engine of the Pipe2010 program.

C. Model Scenarios

Extended period simulations (EPS) were conducted to simulate various current year and one identified target future year demand conditions. The minimum demand day scenario was utilized as a basis for modeling improvements to water age, and the target year maximum day demand was used as a basis in modeling hydraulic improvements. Other current year and target year scenarios were also conducted. A separate fire flow analysis was also conducted.

D. Distribution Storage and System Improvements

After baseline model scenarios were completed, the results were analyzed to identify potential improvements. Recommendations were targeted to improve tank turnover, maintain adequate system storage and pressure, improve water quality by reducing water age, and comment on relative fire flow availability.

1.03 DEFINITIONS

CCWD No. 1	Carroll County Water District No. 1
CCWD No. 2	Carroll County Water District No. 2
Central	KAW Central Division
DBP	disinfection byproducts
EPS	extended period simulation
GCWD	Gallatin County Water District
HGL	hydraulic grade line
KAW	Kentucky American Water
KRS1	Kentucky River Station 1
KRS2	Kentucky River Station 2
KYTC	Kentucky Transportation Cabinet
MG	million gallons
MGD	million gallons per day
MM	Master meter
Northern	KAW Northern Division
psi	pounds per square inch
PS	pumping station
PRV	pressure reducing valves
PSV	pressure sustaining valves
RRS	Richmond Road Station
SCADA	supervisory control and data acquisition
Strand	Strand Associates, Inc.®
WTP	water treatment plant

SECTION 2
PRELIMINARY MODELING

2.01 MODEL BACKGROUND

A. Central Model

The Central model is a skeletonized model of the Fayette County area system including facilities as far north as the Sadieville tank in Scott County with a portion of the smaller mains not included. This model utilized the model updated by Strand Associates, Inc.® (Strand) for the July 2007 *Pool 3 3-MG Tank and Booster Station Modeling* report as a starting point.

B. Northern Model

The Northern model includes the entire distribution system in Owen County and surrounding areas including those served by the Owenton Water Treatment Plant (WTP) as well as those served from bulk purchases of adjoining water utilities. The Northern model was provided by KAW staff at the start of this project and was last updated in December 2008 by HDR Engineers. Because of the smaller size of the system, this model has not been skeletonized and represents Strand's understanding of the system in its current state.

2.02 MODEL UPDATES

A. Central Model Updates

The following updates were made to the Central model based on key water main replacements and extensions installed since the model's latest update.

1. Second Street–Added new 12-inch main between Jefferson Street and North Upper Street along Second Street. Actual main replacement was between Jefferson Street and North Limestone but no mains exist between North Upper Street and North Limestone in the skeletonized Central Division Model.

2. North Broadway–Added new 12-inch main between Church Street and Loudon Avenue on North Broadway.

3. South Limestone–Increased size of main on South Limestone between Virginia Ave and Montmullen Street to 12 inches. Added new 12-inch main between Montmullen Street and East Vine Street on South Limestone.

4. Oliver Lewis Way–Revised connectivity near Cox Street tanks. Added new 20-inch main crossing Oliver Lewis Way and new 12- and 16-inch mains on Pine Street Connecting to Manchester and Valley Avenue. Added new 8-inch mains on Madison Place and Meringo Street connecting West Maxwell and West High.

5. Newtown Pike–Added new 12-inch main connecting Aristides Boulevard to Newtown Pike.

6. Liberty Road–Added new 12-inch main from Grafton Mills Road to Todds Road along Liberty Road. Added new 12-inch main to complete loop from Liberty Road to Star Shoot Parkway.

7. New Circle Road–Added new 8- and 12-inch parallel mains from Floyd Drive to Bryan Avenue along East New Circle Road. Revised connectivity at intersections along East New Circle at Bryan Avenue, Floyd Drive, and Meadow Drive.

8. Jouett Drive and Sperling Drive–Added new 8- and 12-inch mains along Jouett Drive and Sperling Drive off Hayes Boulevard.

9. Spurr Road–Added new 8-inch main along Spurr Road west of Sadiesville Road.

10. Sir Barton Way–Added new 12-inch main from Carducci Street to Winchester Road along Sir Barton Way.

11. Polo Club Development–Added new 8-inch mains along Blackford Parkway, Cherry Meadow Park, Sunningdale Drive, Scottish Trace, Barrington Lane, and Walnut Grove Lane.

The following mains were added to the model to more accurately represent the existing piping layout in the Central system:

1. Royster Road–Added existing 6-, 8-, and 12-inch lines along Royster Road between North Cleveland and Briar Hill Road.

2. Todds Road–Added existing 3-, 4-, 6-, and 12-inch lines along Todds Road between North Cleveland Road and Deer Haven Lane.

3. North Cleveland Road–Added existing 6-inch line along North Cleveland Road between Todds Road and Athens Boonesboro Road.

4. Athens Boonesboro Road–Added existing 6- and 8-inch lines along Athens Boonesboro Road between Interstate 75 and Aphids Way.

5. Muddy Ford Road–Added existing 2-, 4-, and 6-inch lines along Muddy Ford Road between Barkley Road and Old US 62.

6. Winthrop Drive–Added existing 8-inch line on Winthrop Drive between Sunny Slope Trace and Wyndam Hills Drive.

7. Pine Needles Lane–Added existing 8-inch line along Pine Needles Land and Gulford Lane between Man O War Boulevard and Tupelo Lane.

8. Hume Bedford Pike–Added existing 8-inch line along Hume Bedford Pike between Russell Cave Road and Greenwich Pike

The KRS2 clearwell levels, KRS2 high service pump curves, Woodlake pump curves, and pumping configurations for KRS2 and Woodlake were updated in the model to represent their final constructed configurations.

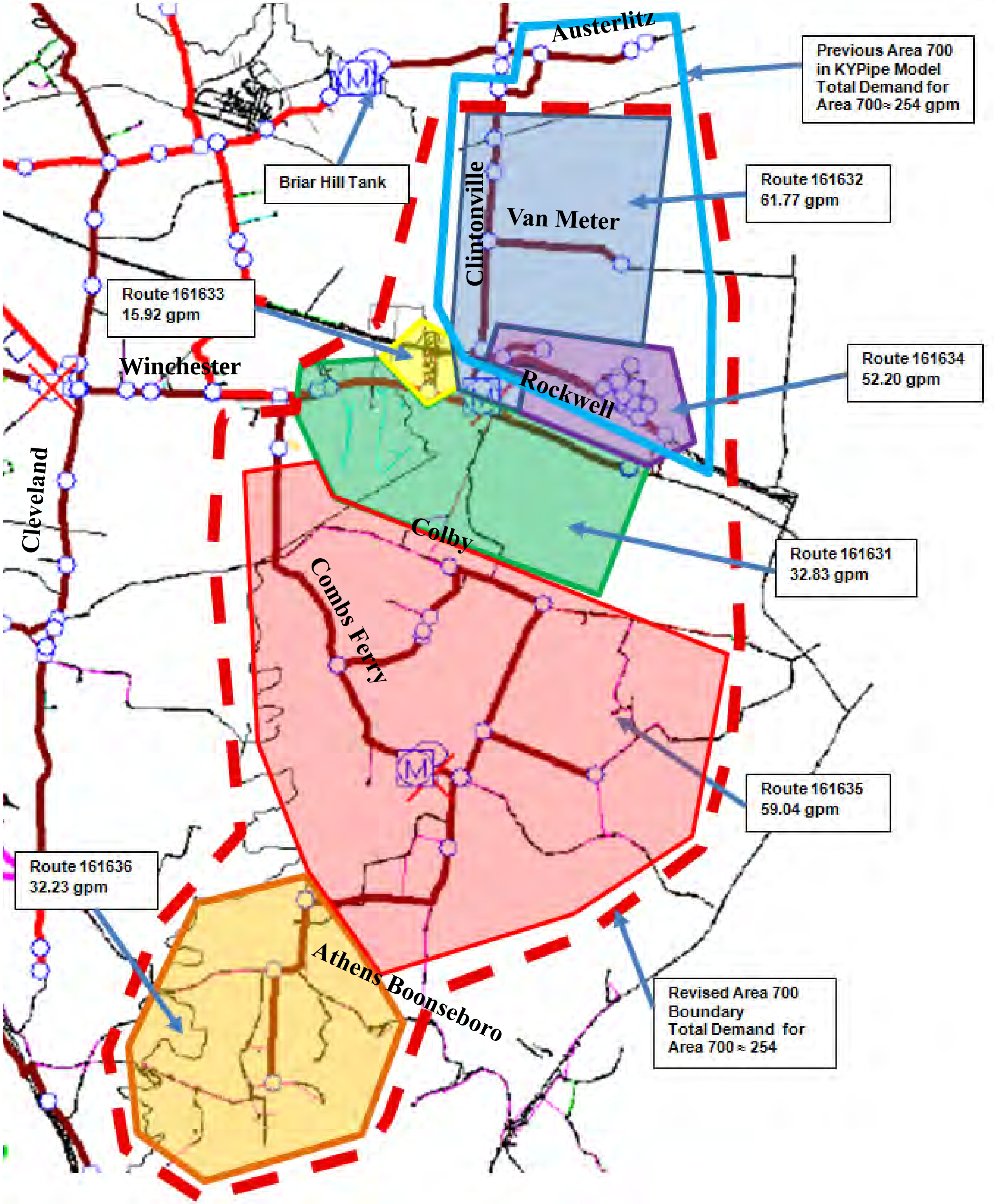
KAW partially closes a valve on the discharge side of KRS1 high service pump No. 12 and RRS high service pump No. 10. To represent this in the model, an additional K value of 30 was placed on the discharge pipes of each of these pumps.

Pressure sustaining valves (PSV) were added to the inlet pipes of the following ground storage tanks in the Central model based on confirmation of their installation from KAW staff. The valve pressures were set to achieve SCADA-reported fill rates of the ground storage tanks.

1. Clays Mill tank inlet: PSV set to 65 pounds per square inch (psi).
2. Mercer Road tank inlet: PSV set to 69 psi.
3. Parkers Mill tank inlet: PSV set to 62 psi.
4. York Street tank inlet: PSV set to 62 psi.

Pipe 12372 on the Briar Hill pumping station bypass was closed and the check valve removed based on comparison of preliminary modeling to SCADA information. The Hall tank inlet pipe was changed from 8 inches to 6 inches based on drawings provided by KAW.

A number of junction demands in the Briar Hill tank area were also redistributed to appropriately model demand conditions in the area. Initial modeling resulted in excessively low minimum pressures along Austerlitz Road, Van Meter Road, and Rockwell Road off Clintonville Road in the Briar Hill tank service area. KAW indicated that low pressures were typically not experienced in this area. Modeling results indicated low pressures were caused by excessive head loss in the water lines as a result of high flow conditions from nodes within distribution zone 700. A review of the model demands and demand information provided by KAW indicated that demands on Austerlitz Road, Van Meter Road, and Rockwell Road off Clintonville Road in distribution zone 700 were previously allocated based on a distribution zone map that is currently out of date. Demands along these roads were redistributed on a street-by-street basis from updated 2006 distribution zone and meter route customer water usage information supplied by KAW. Figure 2.02-1 shows how the demand in the Briar Hill area was redistributed. As shown in Figure 2.02-1, the previous base demand of distribution zone 700 was 254 gpm and was isolated to the area outlined in blue on the north side of the figure, which includes Austerlitz Road, Van Meter Road, Rockwell Road, and part of Clintonville Road. The base demand of 254 gpm was redistributed over a much larger area shown with the red dash outline shown in Figure 2.02-1. Shaded areas in Figure 2.02-1 also show how individual meter route demand was redistributed throughout the area based on the street-by-street review of customer usage in the area.



REVISED BRIAR HILL AREA
DEMAND ALLOCATION
HYDRAULIC ANALYSIS FOR
COMPREHENSIVE PLANNING STUDY
KENTUCKY AMERICAN WATER
LEXINGTON, KENTUCKY

STRAND ASSOCIATES®

FIGURE 2.02-1

5493.117

B. Northern Division Model Updates

The following updates were made to the Northern Division model based on water main replacements and extensions installed since the model's latest update.

1. KY-607–Added new 6-inch main from US-127 to Herman Greene Road along KY-607.

2. KY-2018–Added new 6-inch main from KY-1883 to KY-607 along KY-2018.

3. Big Twin Creek Road–Added new 4-inch main extension off KY-325 along Big Twin Creek Road.

4. KY-330–Added new 6-inch main along KY-330 to KY-845. Revised connectivity at intersection of KY-330 and KY-845 by connecting the new and existing 6-inch main on KY-330 to the existing 6-inch main on KY-845.

5. Duncan Hill Road–Added new 6-inch main along Duncan Hill Road between KY-845 and KY-330.

6. Herman Greene Road–Added approximately 8,000 LF of new 4-inch main southwest along Herman Greene Road from its intersection with Dividing Ridge Road.

Other Northern model updates include the following:

1. Revised inlet and outlet piping configuration for the Bromley Tank, Wheatley Tank, and the Monterey Tank based on conversations on their operation with KAW staff. Figures 2.02-2, 2.02-3, and 2.02-4 display the former and updated piping configurations for these tanks.

2. Monterey tank drain line increased from 6-inch to 8-inch based on record drawings of the Monterey area.

3. Owenton WTP reservoir grade was increased from 874 to 900 based on information provided by KAW.

4. Removed the 4-inch main along KY-845 between KY-22 and KY-330 based on information provided by KAW.

5. The Rockdale MM and Leaning Oak MM reservoir grades were increased to 1,095 based on pressures observed during hydrant flow tests.

6. A number of mains were closed in the southern portion of the model based on information provided by KAW to isolate the Rockdale MM, Leaning Oak MM, and New Columbus tank service areas. Figure 2.02-5 shows the Northern system infrastructure and pressure zone boundaries. Pressure zone boundaries in the model used closed

valves and pressure reducing valves (PRVs) to isolate pressure zones based on information provided by KAW.

7. The Owenton WTP pump curve was modified to decrease the flow. Modeled flow rates were consistently high compared to SCADA recorded flow rates. KAW staff also indicated the Owenton WTP high service pump was not operating on its curve. Table 2.02-1 shows the KYPipe Owenton WTP pump curve as it was provided in the 2008 HDR KYPipe model.

Original Pump Curve		Modified Pump Curve	
Head (ft)	**Flow (gpm)**	**Head (ft)**	**Flow (gpm)**
338	0	305	0
260	750	227	750
150	1,280	125	1,280

Table 2.02-1 Owenton WTP Pump Curve

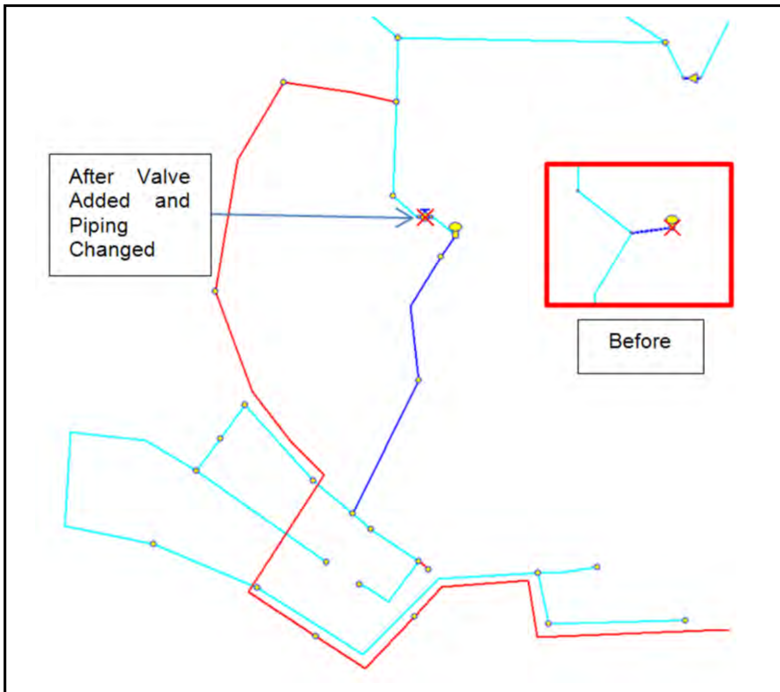


Figure 2.02-2 Monterey Tank Piping Configuration

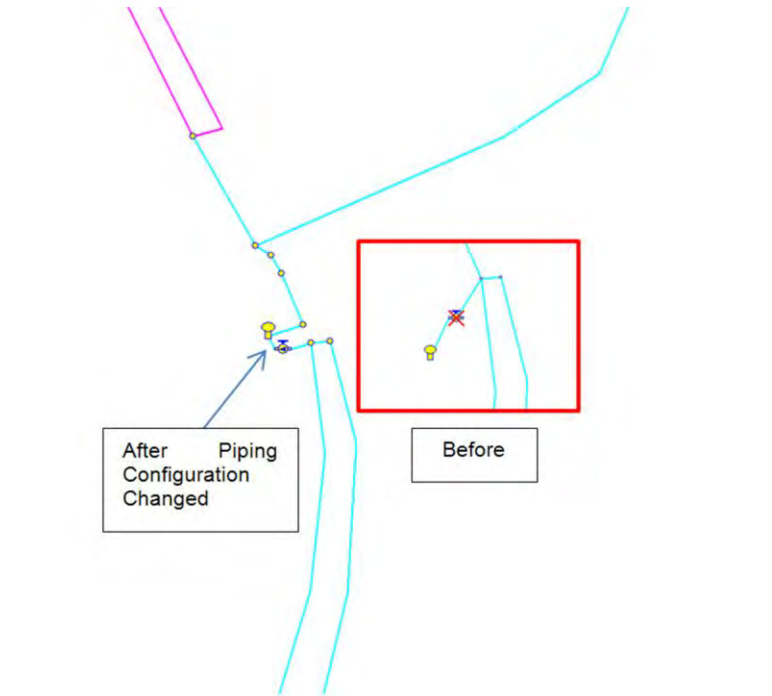


Figure 2.02-3 Bromley Tank Piping Configuration

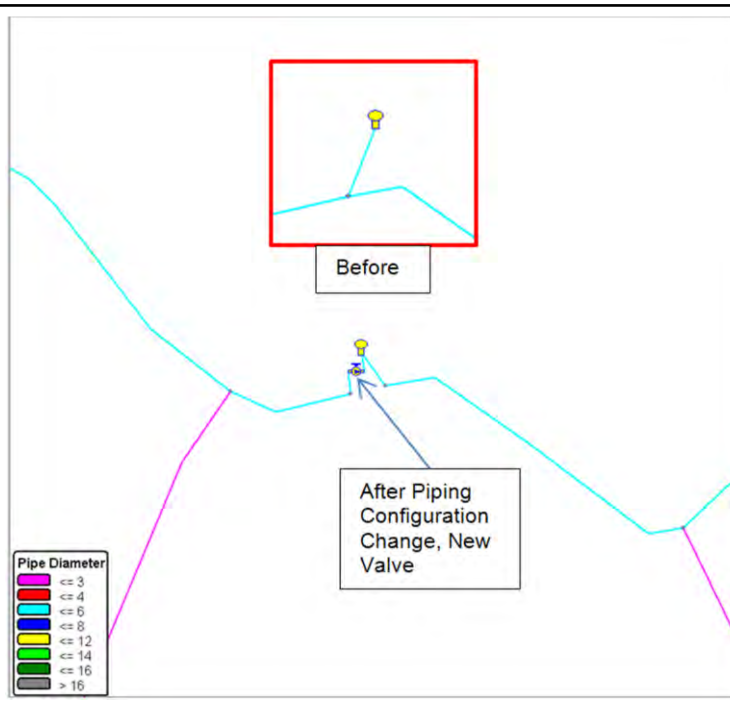
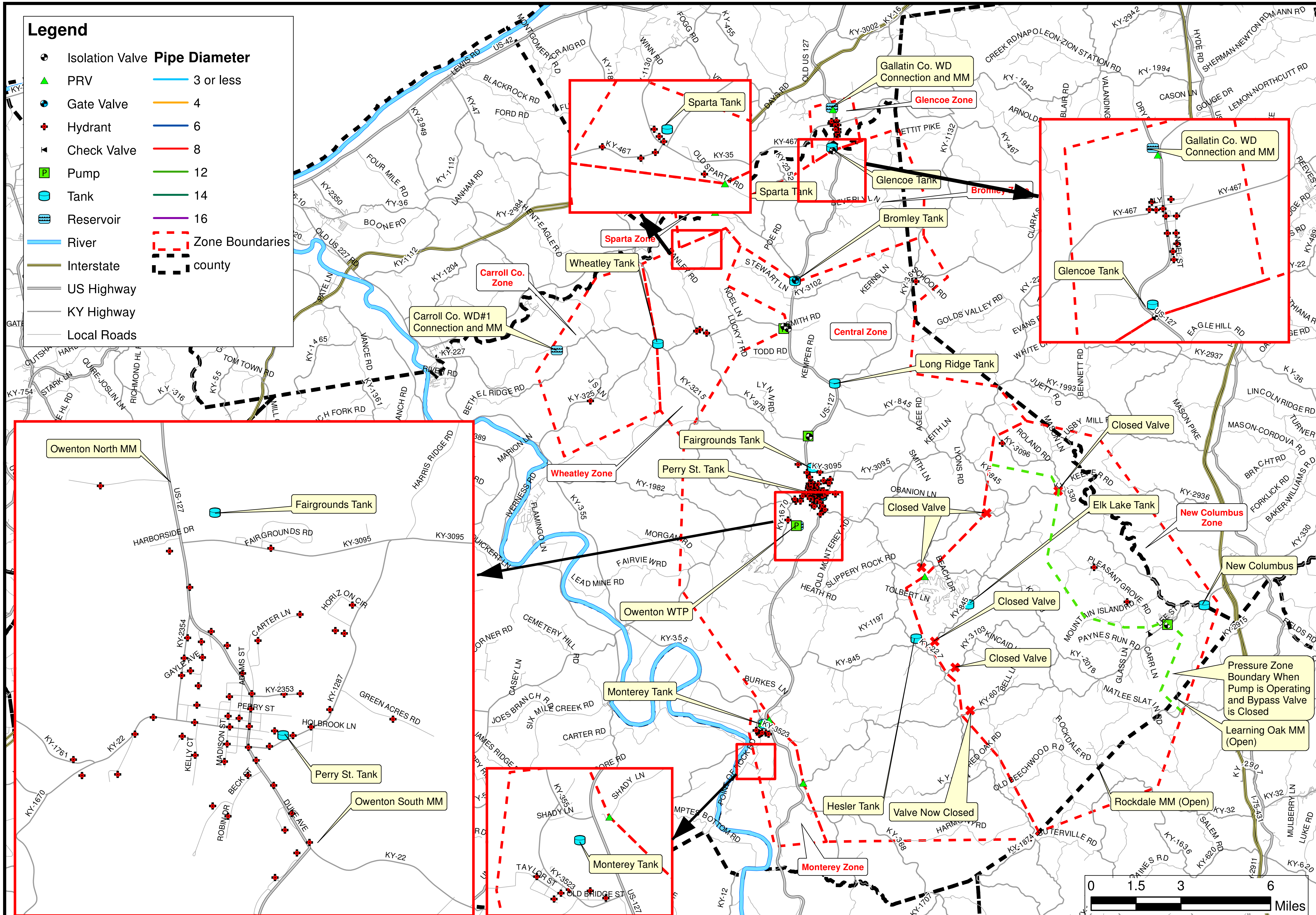


Figure 2.02-4 Wheatley Tank Piping Configuration



NORTHERN DIVISION PRESSURE ZONES

HYDRAULIC ANALYSIS FOR COMPREHENSIVE PLANNING STUDY
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FIGURE 2.02-5
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Path: S:\LOU\5400--5499\5493\117\Data\GIS\Report Figures\Figure 2.02-5 North Pressure Zones-New Template.mxd User: andrewe Date: 2/3/2012 Time: 2:07:45 PM

2.03 DEMAND ALLOCATION

A. Central Model

Base demand allocation was not adjusted in the Central model. Demand patterns were developed for the various scenarios based on SCADA recorded data to simulate the appropriate water demand in the Central model for each scenario. Additional demands were added to the Central model for future scenarios. See section 4.02 for details.

Peaking factors for the minimum and average demand scenarios were developed based on SCADA WTP production and tank levels. No SCADA was provided for a maximum demand day scenario, therefore the peaking factors from the previous KRS2 modeling efforts for a maximum demand scenario from the July 2007 *Pool 3 3-MG Tank and Booster Station Modeling* report were used and scaled to produce the appropriate demand in the Central model.

B. Northern Model

Base demands in the Northern model were allocated using customer and meter route usage data provided by KAW for the month of May 2011. The four largest users were identified in the usage data and demand for these users was individually placed on junctions in the Northern model. Large users and their representative junctions in the model are:

1. Water fill station north of Owenton on US127, KYPipe junction J-363.
2. Water fill station within Owenton, KYPipe junction J-695.
3. Itron Industries on the north side of Owenton, KYPipe junction J-798.
4. Owenton WWTP, KYPipe junction J-269.

After large users were individually allocated, the remaining water usage for each meter route was calculated, and the demand was evenly distributed to all junctions within each meter route. Figure 2.03-1 shows the model junction meter routes based on the meter route map provided by KAW.

Peaking factors were developed using SCADA data and field data provided by KAW. SCADA data was provided for WTP production, tank levels, and North and South Master Meter (MM) flows. Weekly flow volumes for two weeks in September 2011 were provided for the Leaning Oak and Rockdale MMs. Northern model demand types were separated by area and type of use. Table 2.03-1 summarizes the demand types used for the Northern model.

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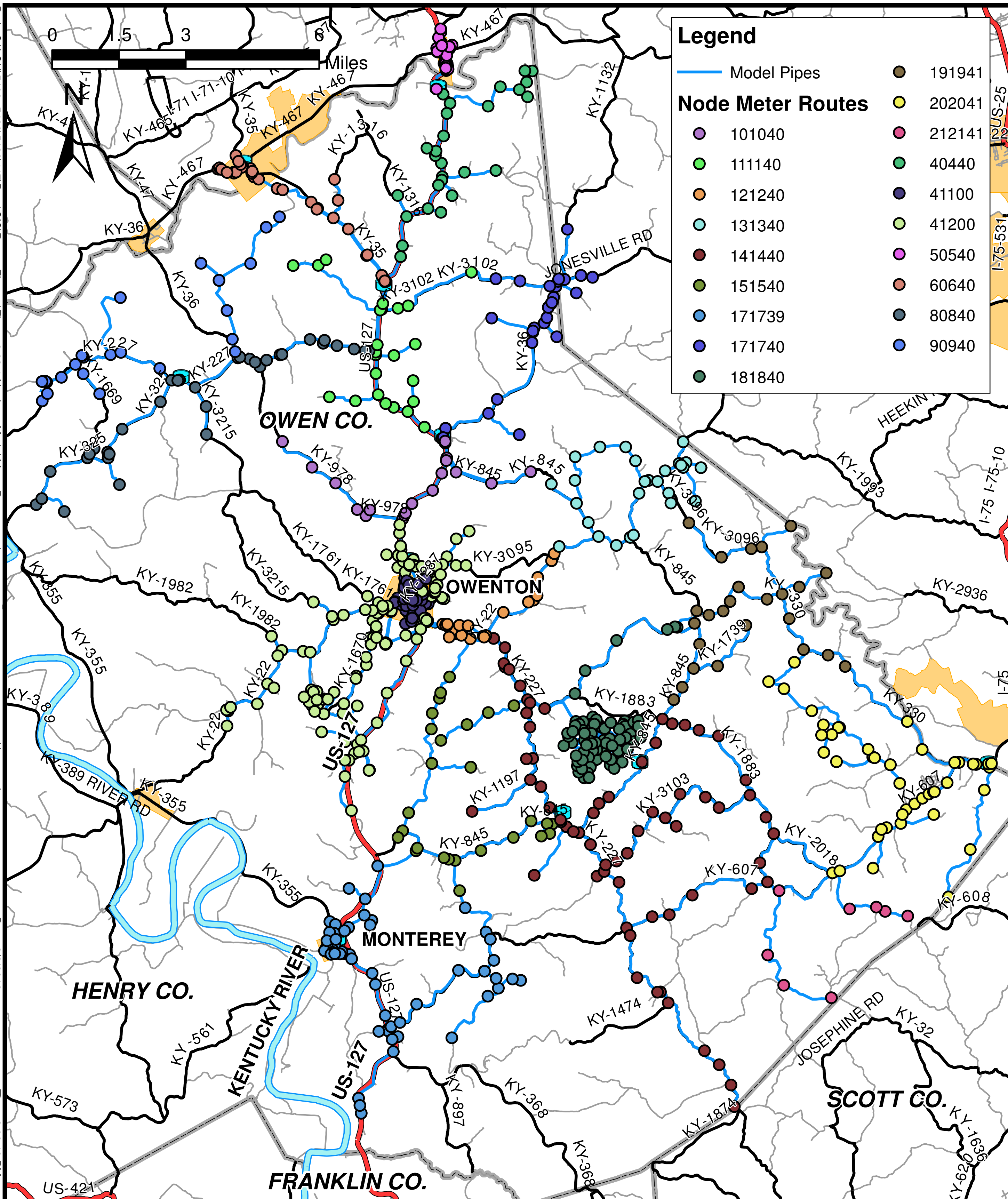

NORTHERN DIVISION MODEL

JUNCTION METER ROUTES

HYDRAULIC MODELING FOR THE

COMPREHENSIVE PLANNING STUDY

KENTUCKY AMERICAN WATER

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FIGURE 2.03-1

5493.117

Description	Demand Type
North MM Service Area	1
Owenton	2
Glencoe	3
Carroll Co./Wheatley Area	4
South MM Service Area	5
Rockdale MM, Leaning Oak, and New Columbus Service Area	6
Monterey	7
Large Users	8

Table 2.03-1 Northern Model Demand Types

2.04 MODEL CALIBRATION

A. <u>Central Model</u>

SCADA-recorded data provided by KAW for an average demand day was used as a basis for determining model calibration for the Central model. July 17, 2011, was selected by KAW as the average demand day.

A current average demand day scenario was simulated in the Central model after model updates were incorporated, and results were compared to the SCADA-recorded data. Initial result comparisons indicated a number of tanks were not filling at the appropriate rate. Changes were made to the model to better match SCADA-recorded tank fill rates. These changes include the incorporation of the PSV, the inlet pipe diameter change to the Hall tank, and the closing of the Briar Hill PS bypass which better reflect actual field conditions.

After the changes were incorporated and results were compiled, KAW staff reviewed the results of the simulation against the SCADA-recorded data and determined the Central model was trending well with the SCADA data. Therefore, the Central model was considered calibrated and no additional calibration efforts were necessary. Figures 2.04-1 through 2.04-14 show SCADA and modeled tank levels and WTP production for the current average day used to determine calibration.

Figure 2.04-3 shows only one SCADA tank level and one modeled tank level for the Clays Mill tanks. Although the existing system has two Clays Mill tanks, the tanks are identical in volume, shape, and elevation and are hydraulically linked to each other based on piping configuration and their proximity. KAW draws from both tanks equally when draining the tanks; therefore, the two Clays Mill tanks were modeled as one large tank with a total volume of both the Clays Mill tanks for simplicity.

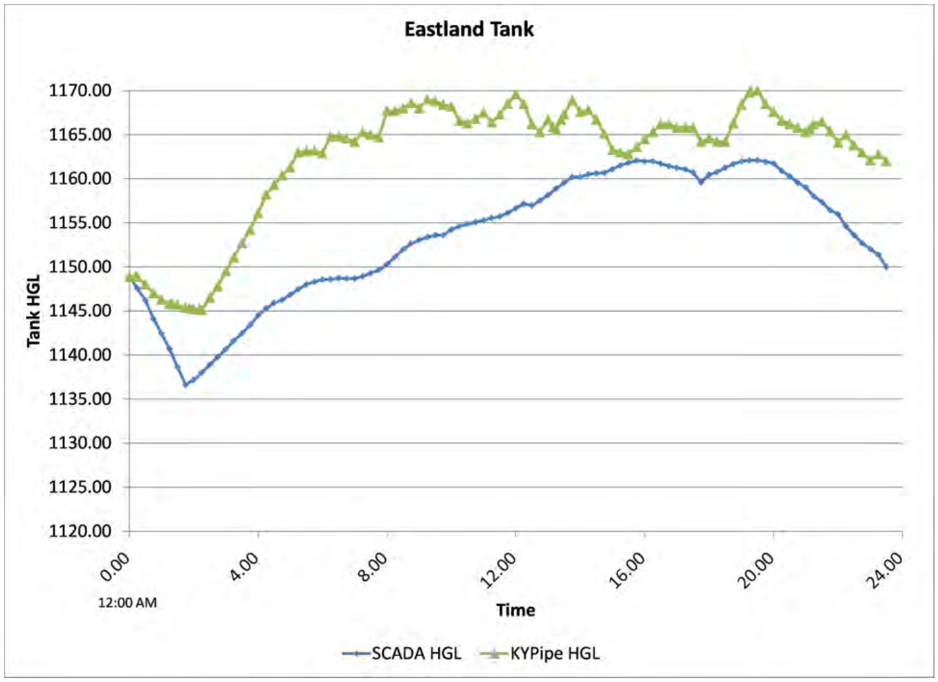


Figure 2.04-1 Eastland Tank SCADA Calibration Comparison

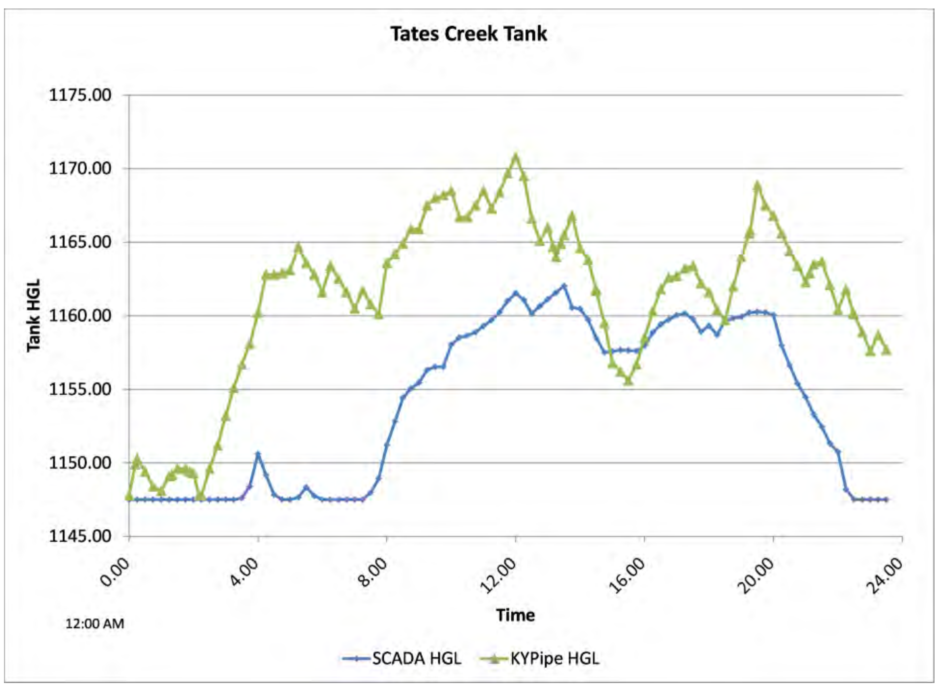


Figure 2.04-2 Tates Creek Tank SCADA Calibration Comparison

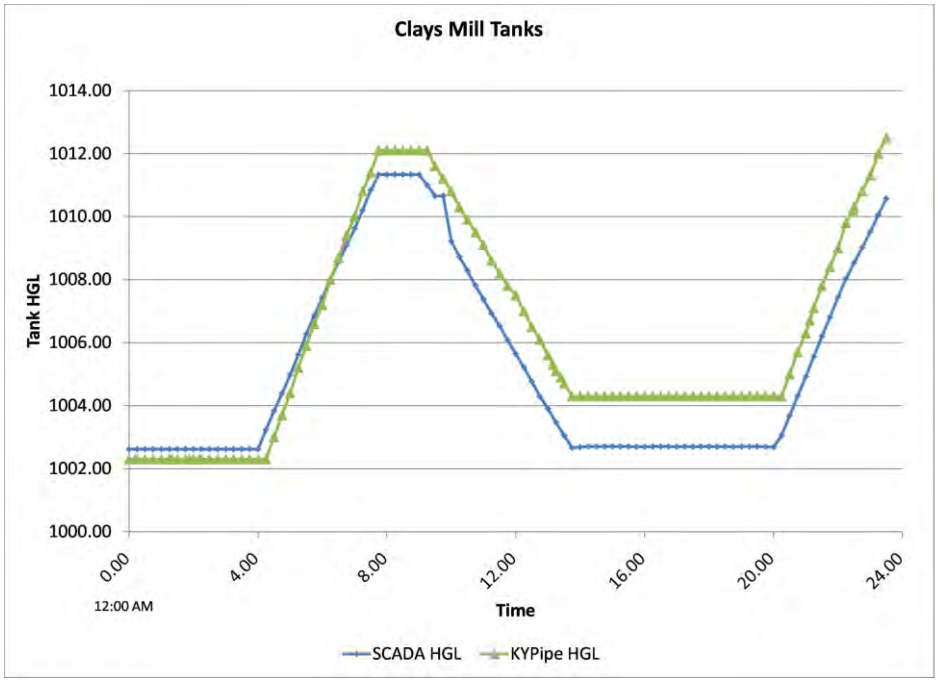


Figure 2.04-3 Clays Mill Tank SCADA Comparison

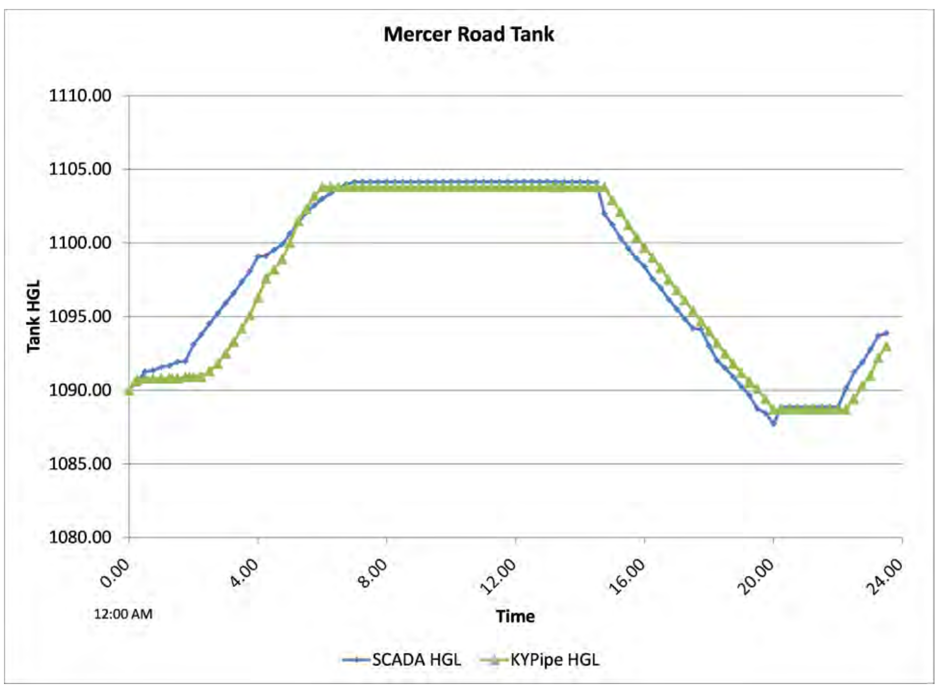


Figure 2.04-4 Mercer Road Tank SCADA Comparison

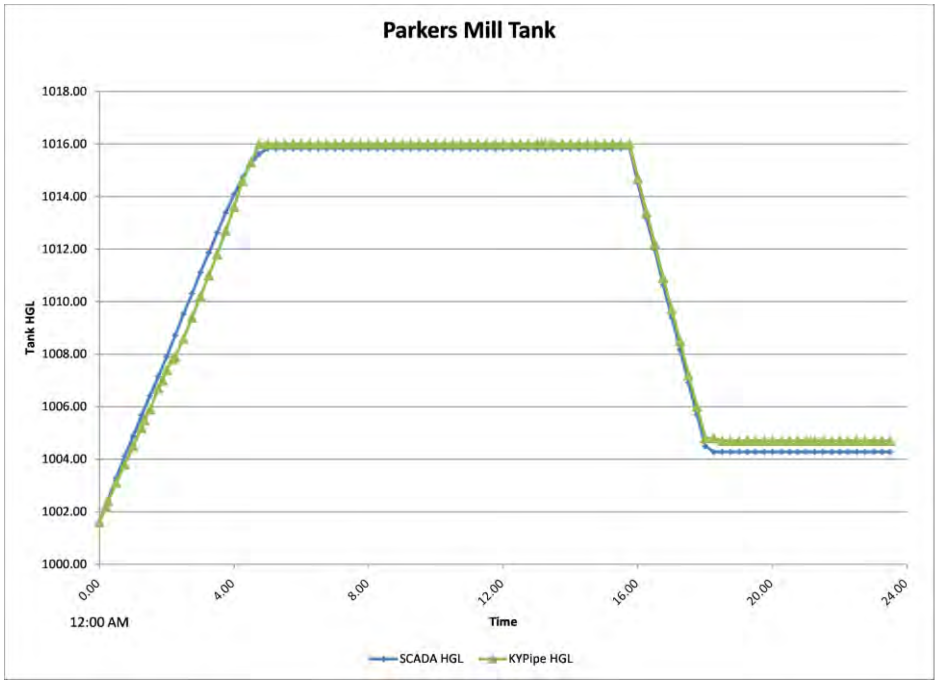


Figure 2.04-5 Parkers Mill Tank SCADA Comparison

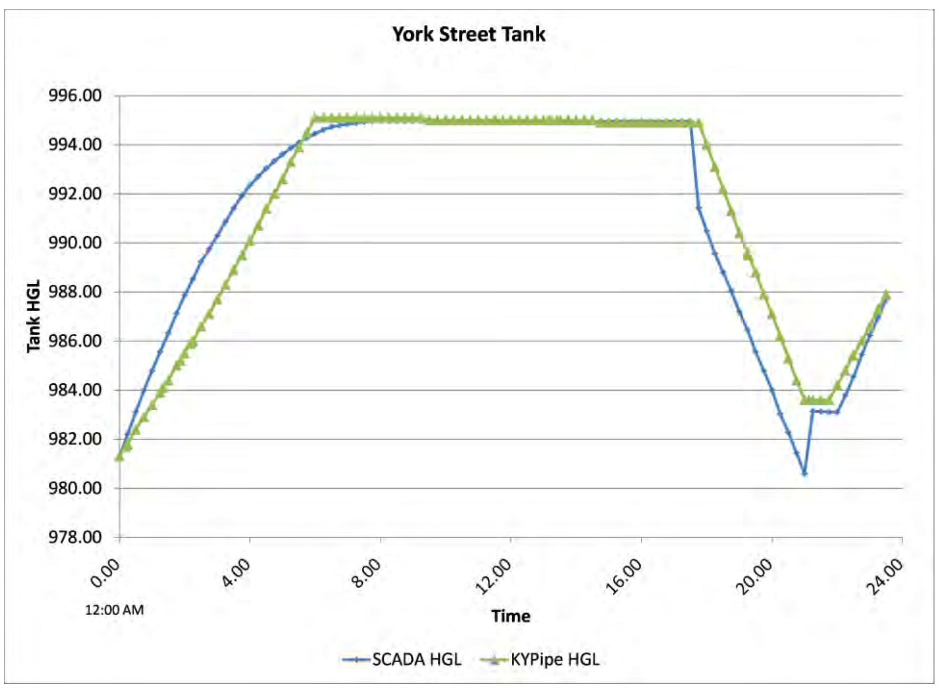


Figure 2.04-6 York Street Tank SCADA Comparison

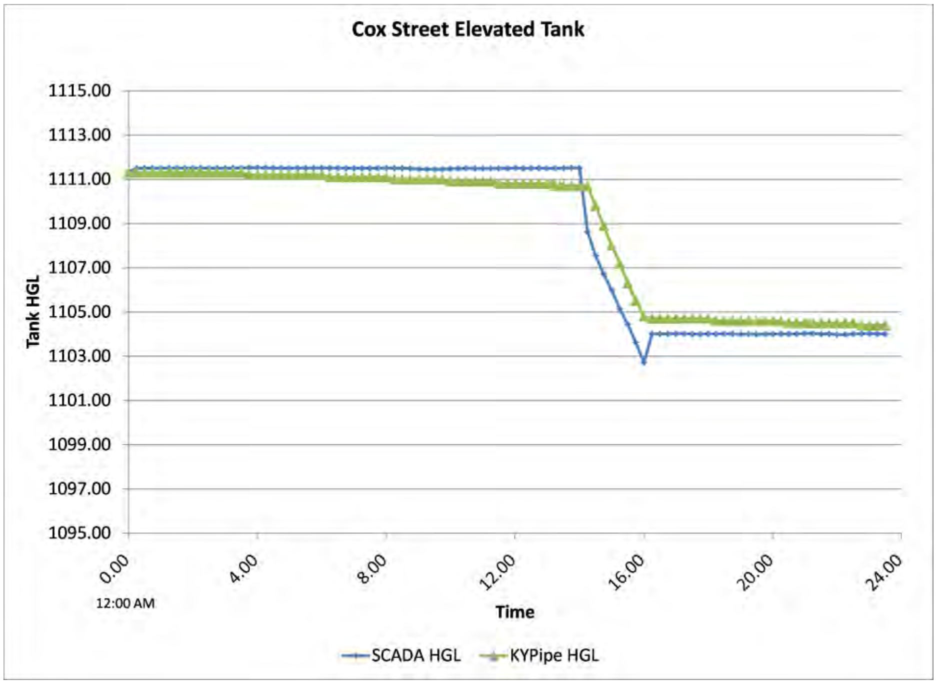


Figure 2.04-7 Cox Street Elevated Tank SCADA Comparison

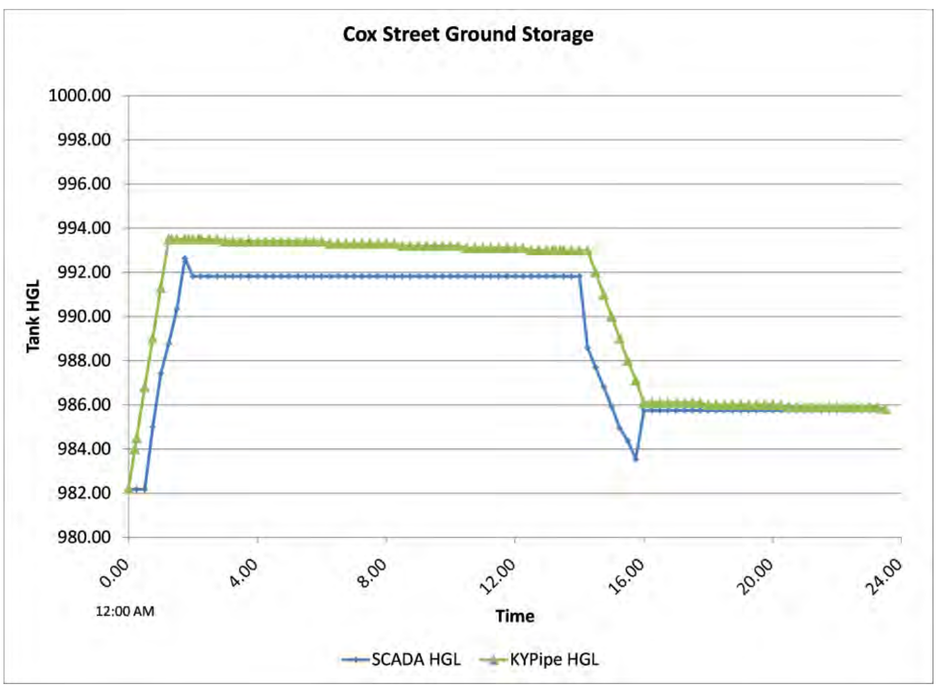


Figure 2.04-8 Cox Street Ground Tank SCADA Comparison

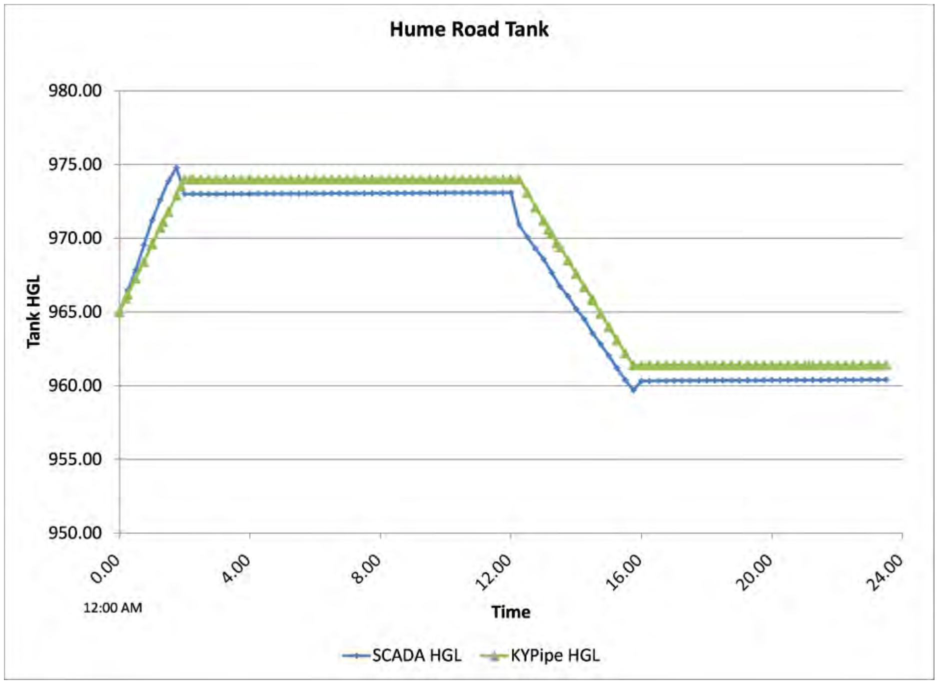


Figure 2.04-9 Hume Road Tank SCADA Comparison

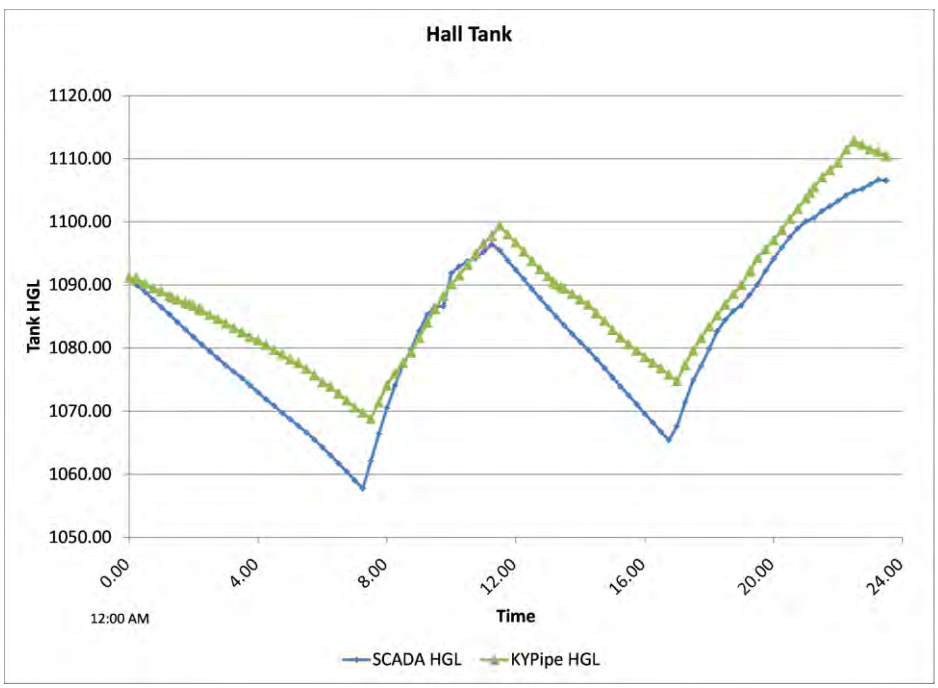


Figure 2.04-10 Hall Tank SCADA Comparison

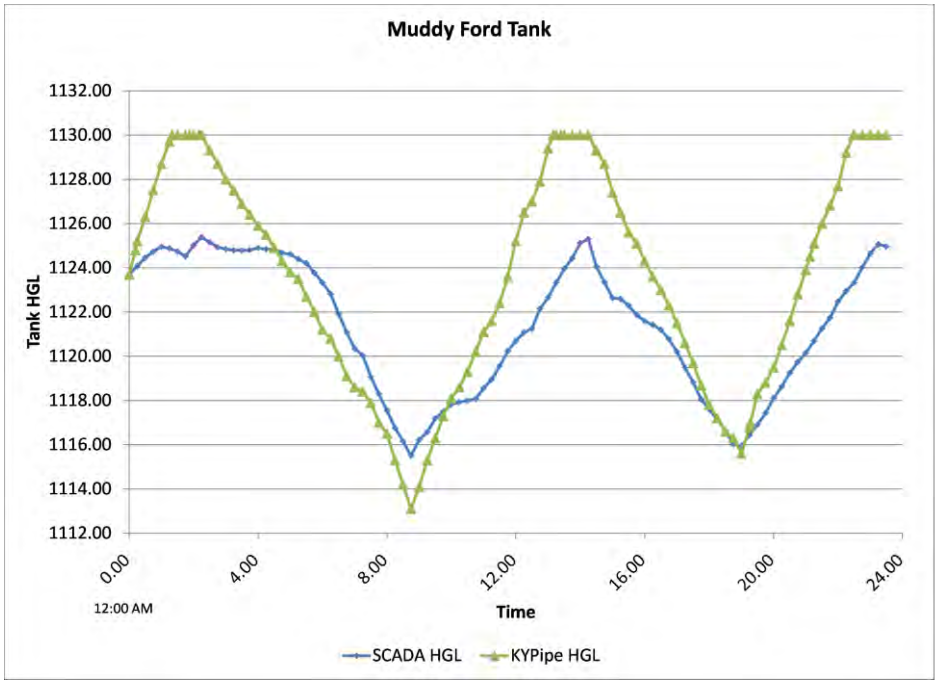


Figure 2.04-11 Muddy Ford Tank SCADA Comparison

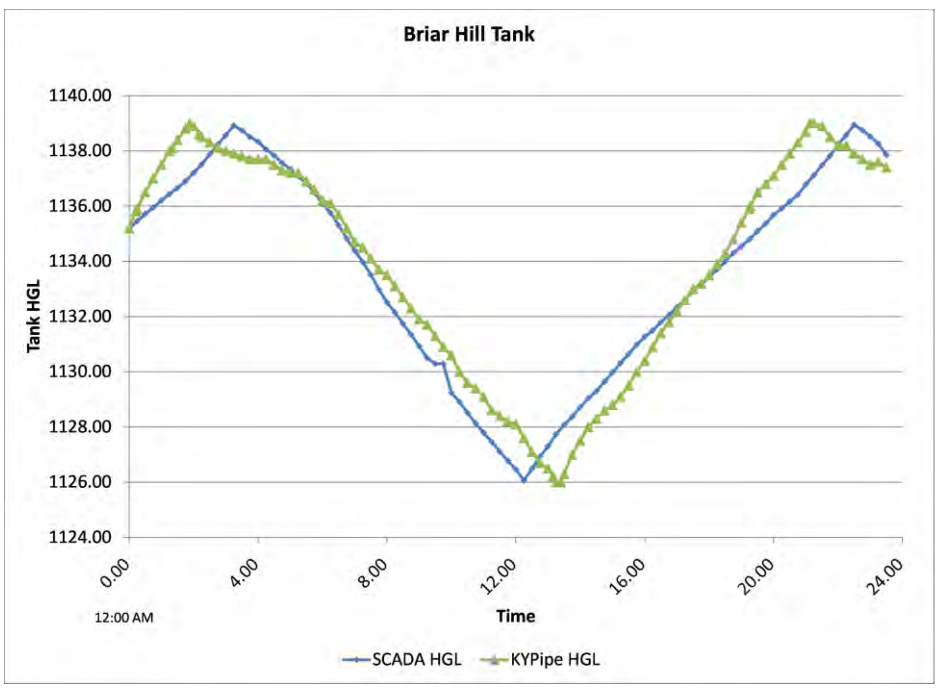


Figure 2.04-12 Briar Hill Tank SCADA Comparison

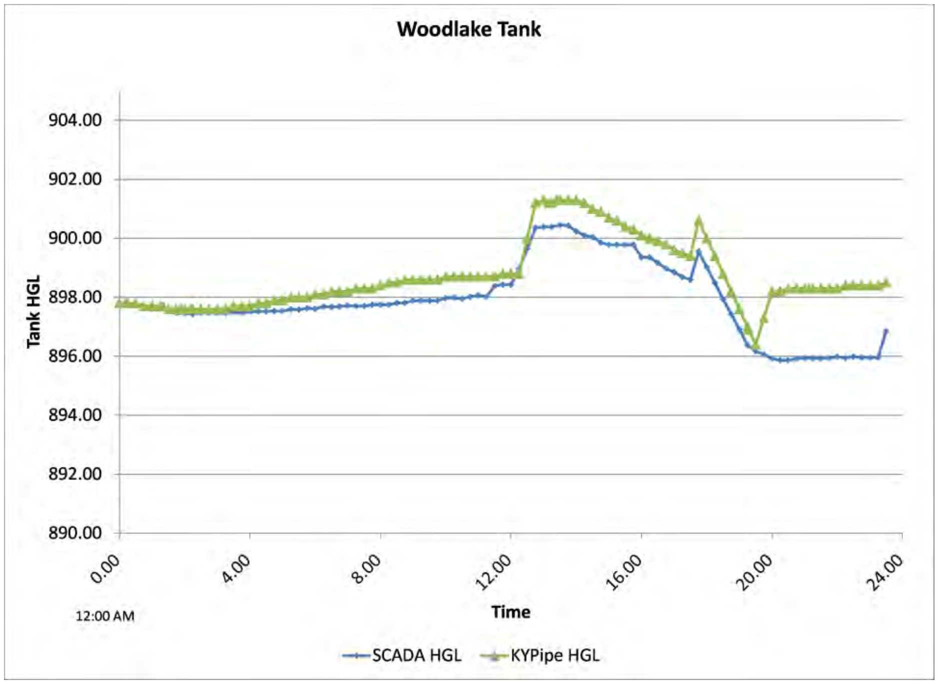


Figure 2.04-13 Woodlake Tank SCADA Comparison

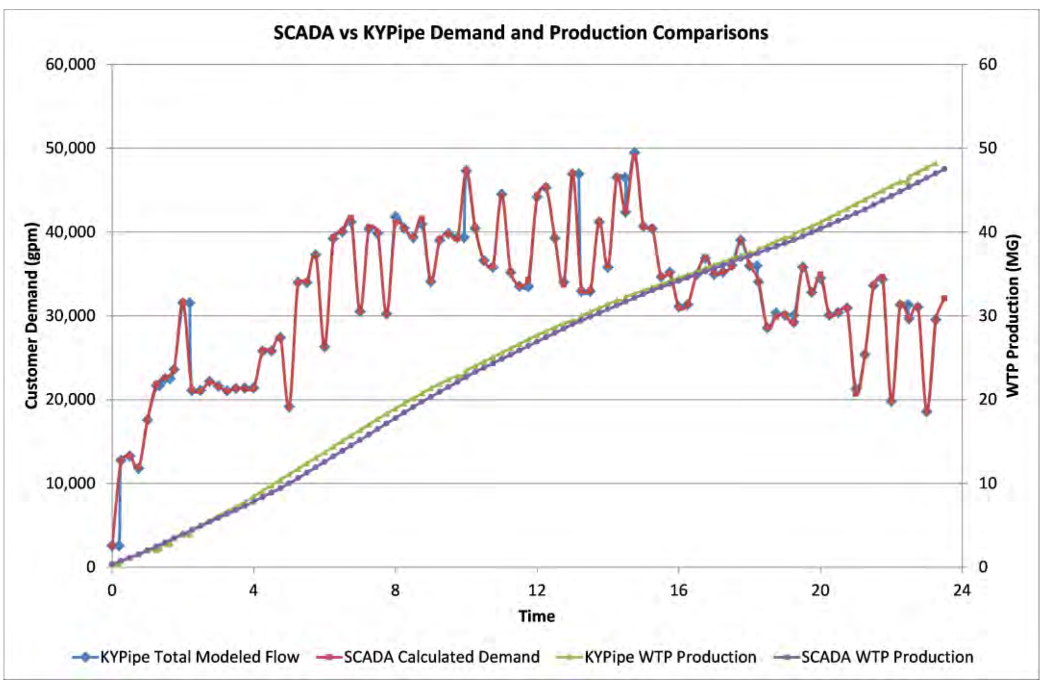


Figure 2.04-14 Demand and WTP Production

B. Northern Model

Hydrant flow tests were performed in the Northern system at six locations on August 15, 2011. Test locations were distributed throughout the system and located on pipes of various material and age to determine C factors throughout the entire Northern system. Figure 2.04-15 shows the location of the six hydrant flow tests.

The hydraulic calibration engine of the Pipe2010 program was used to calibrate the Northern model. Hydrant test results were recorded and input into the model at the appropriate locations. Pipe calibration groups were selected based on location and pipe age. Table 2.04-1 summarizes the pipe calibration groups used for the Northern model and calibrated C factors. Table 2.04-2 summarizes the measured residual pressures taken during the hydrant flow tests and the calibrated pressures observed by the Northern model.

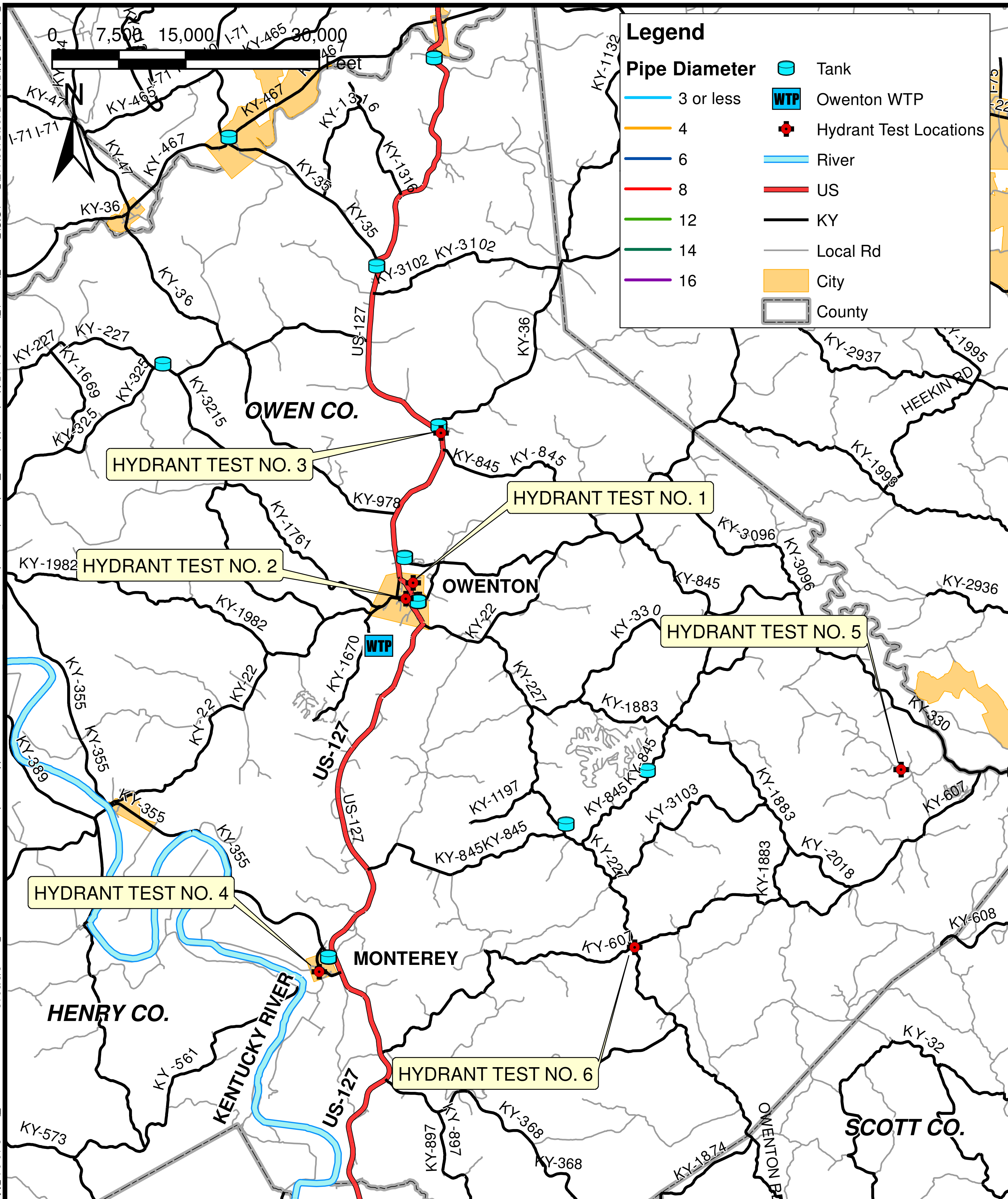
Initially the model calibration results for hydrant test no. 6 were not producing reasonable pressure results based on the measured hydrant test flow and pressure and the described operation of the Northern system. Additional field work was conducted by KAW staff who determined that a valve at the intersection of KY607 and KY227 that was thought to be closed during the hydrant tests was actually open. The calibration engine was run again on the Northern model with the updated valve information to produce the results shown in Tables 2.04-1 and 2.04-2. The results of the model calibration and hydrant flow tests were presented to KAW staff who determined that the model was satisfactorily calibrated.

Calibration Group	Description	Approximate Installed Year	Lower Bound	Upper Bound	Calibrated Roughness
0	Owenton Area	1940s	80	120	112
1	Owenton Area and North of the North MM	1960s	90	120	96
2	South of the South MM, Elk Lake and Monterey Area	1960s	90	120	118
3	Owenton Area and South of the South MM	1980s	100	140	134
4	North and South MM Areas	1990s	100	140	139
5	Owenton Area	2000s	110	140	140
6	North of the North MM	2000s	110	140	127
7	South of the South MM	2000s	110	140	137
8	Rockdale MM Service Area	2000s	110	140	140
9	New Columbus Tank Service Area	2000s	80	120	113

Table 2.04-1 Northern Model Pipe Roughness Calibration Results

Hydrant Test No.	Location Description	Measured Residual Pressure (psi)	KYPipe Calibrated Residual Pressure (psi)	Percent Difference
1	Carter Lane within Owenton	45	43.8	2.7%
2	Corner of Bryan Street and Madison Street within Owenton	30	32.7	9.0%
3	Near intersection of US127 and KY36	8	7.3	8.8%
4	Intersection of Taylor and Worth within Monterey	50	49.8	0.4%
5	Near Intersection of Fox Trail and Pleasant Grove Road.	8	10.9	36.3%
6	Intersection of KY607 and KY227	16	12.6	21.3%

Table 2.04-2 Northern Division Hydrant Flow Test Calibration Results



NORTHERN DIVISION
HYDRANT TEST LOCATIONS
HYDRAULIC MODELING FOR THE
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FIGURE 2.04-15
5493.117

SECTION 3
CENTRAL DIVISION OPERATIONS REVIEW

3.01 CURRENT SYSTEM OPERATION

The following describes the current intended operation of facilities in the Central Division under various demand conditions.

1. Richmond Road Station (RRS) WTP high service pumps are operated to maintain a discharge pressure of approximately 75 to 85 psi under normal operating conditions.
2. RRS high service pump No. 10 is throttled to allow smaller RRS pumps to operate.
3. Kentucky River Station 1 (KRS1) WTP high service pumps are operated to supplement production from RRS with a minimum production of 15 MGD.
4. KRS1 high service pump No. 12 is throttled because it would otherwise operate too far to the right of its curve.
5. Kentucky River Station 2 (KRS2) WTP provides supplemental production as needed and maintains a minimum output of approximately 5.5 MGD. The flow allows the transmission main to turn over and represents the flow at the minimum allowable pump motor speed.
6. Briar Hill Pumping Station (PS) operates to fill Briar Hill tank. Two variable frequency drives maintain a discharge pressure of approximately 90 to 95 psi when the pumps are operating. Briar Hill tank is allowed to drop approximately 12 feet before refilling. System pressures around the tank are maintained at approximately 45 to 55 psi with a minimum of 40 psi.
7. Sadieville tank is not in service because of inability to turn the tank over. Clintonville and Becknerville tanks are also not in service.
8. Ground storage facilities are pumped down and refilled approximately one-third each day.
9. The use of Russell Cave tank and PS has to be tightly monitored because of concerns regarding tank turnover and overpressurizing mains to the north of the tank.
10. Minimum desired system pressure is between 45 and 50 psi.

Table 3.02-1 lists the typical minimum, average, and maximum current day demands experienced in the Central Division.

Description	Minimum Day		Average Day		Maximum Day	
	MGD	GPM	MGD	GPM	MGD	GPM
System Demand	30	20,800	46	31,900	72	51,000
Peak-Hour Demand	45	23,300	71	57,600	112	77,800
KRS1 Production	15	10,400	24	17,100	37	26,700
KRS2 Production	5	3,500	8	5,900	11	7,300
RRS Production	10	6,900	14	10,500	24	17,000

Table 3.02-1 Central System Demand and WTP Production for Current Day Scenarios

The day used to determine calibration for the Central Division was selected as a current average demand day. Appendix A shows information on the status and operation of high service pumps, tanks, and booster pumps.

3.02 CENTRAL DIVISION AREAS OF CONCERN

KAW identified three areas of concern where high and/or low pressure are experienced. Model pressures for these areas for the existing system operating conditions were captured to compare to model results of revised operational improvements. Refer to Section 3.04 for a discussion of model results. These areas are:

1. Russell Cave Tank Area–When Russell Cave tank is in full operation, KAW indicated high pressures were experienced near the tank and north along Russell Cave Road when the Russell Cave PS was operating. Pressures north along Russell Cave Road when the Russell Cave PS was operating were high enough to cause water main breaks. KAW also indicated low pressures were experienced near the intersection of Greenwich Pike and Hume Bedford Pike southeast of the Russell Cave tank when the Russell Cave tank fills.

2. Southeast of Eastland tank–KAW indicated low pressures are experienced in the area roughly bounded by Todds Road, New Circle Road, Winchester Road, and Man O War Boulevard that is immediately southeast of the Eastland tank.

3. Parkers Mill and Clays Mill tank area–KAW indicated high pressures are experienced in the system when the Parkers Mill PS and Clays Mill PS are operating at the same time.

3.03 CENTRAL DIVISION CURRENT OPERATION AND DEMAND HYDRAULIC RESULTS

Current minimum, current maximum, and future minimum demand scenario pressure plots of the Central Division model are shown in Appendix B for reference purposes. Refer to Figures 2.04-1 through 2.04-13 for the current operation modeled tank levels.

3.04 CENTRAL DIVISION ALTERNATIVE OPERATION

A number of operational changes were investigated to identify potential improvements for the Central Division system. The following describes the changes made to the Central Division system operation.

A. Russell Cave Tank Valve Improvements

Currently the use of Russell Cave has to be tightly monitored. Part of the operational changes include valve improvements currently being installed to allow the operation of the Russell Cave tank and PS without negative impacts on the system. Russell Cave tank and PS will operate to fill and drain it one-third each day, similar to the existing operation of other ground storage tanks in the system. The proposed valve improvements include three new SCADA-controlled gate valves and one modulating butterfly valve near the Russell Cave tank.

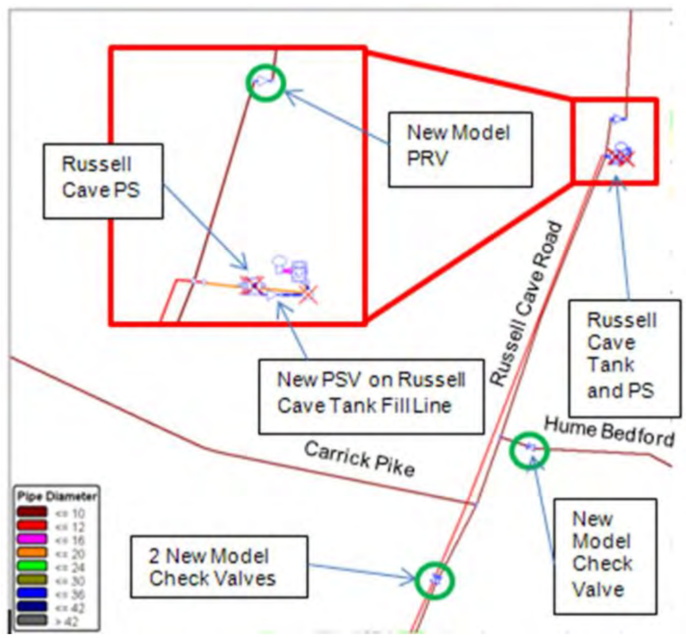


Figure 3.04-1 Proposed Russell Cave Valve Improvements

The modulating butterfly valve will be located on the 8-inch line north of the Russell Cave tank on Russell Cave Road at the Fayette County and Bourbon County line. Two of the gate valves will be located on the 8-inch and 12-inch mains on Russell Cave Road just south of its intersection with Carrick Pike and the third gave valve will be located on Hume Bedford Road near its intersection with Russell Cave Road. The gate valves south of Russell Cave tank will be used to prevent flow from the tank from flowing into the central portion of the system when the Russell Cave PS is operating and allow flow north when the Russell Cave tank needs to be filled. The proposed valves were modeled as check valves to simulate their intended operation. See Figure 3.04-1 for the location of this valve.

The modulating butterfly valve will partially close to reduce the pressure and flow going north into Scott County when the Russell Cave PS is operating. The proposed valve was modeled as a PRV with a hydraulic grade line (HGL) setting of 1160 to simulate its intended operation. Refer to Figure 3.04-1 for the location of this valve.

To prevent pressures from dropping below 30 psi during tank refill, a PSV was also included in the model on the fill line of the Russell Cave tank. Refer to Section 3.05 for a discussion of the PSV setting and model results.

B. New Control Valve Near Newtown PS

Along with the proposed valve improvements near Russell Cave, a new valve is also proposed for the Newtown PS. The new valve will be located on the discharge side of the Newtown PS. The valve will be a modulating butterfly valve similar to the one north of the Russell Cave tank that will partially close to reduce flow and pressure flowing north from the Central gradient. The proposed valve was modeled as a PRV with an HGL setting of 1160 to simulate its intended operation. For the proposed operation, a control valve was also placed in the model on the bypass line of the Newtown PS. This control valve in the model is set to open and close based on the level of the Muddy Ford tank. See Figure 3.04-2 for the location of the new valve in the model. These two modeled valve functions will be accomplished by one valve in the field.

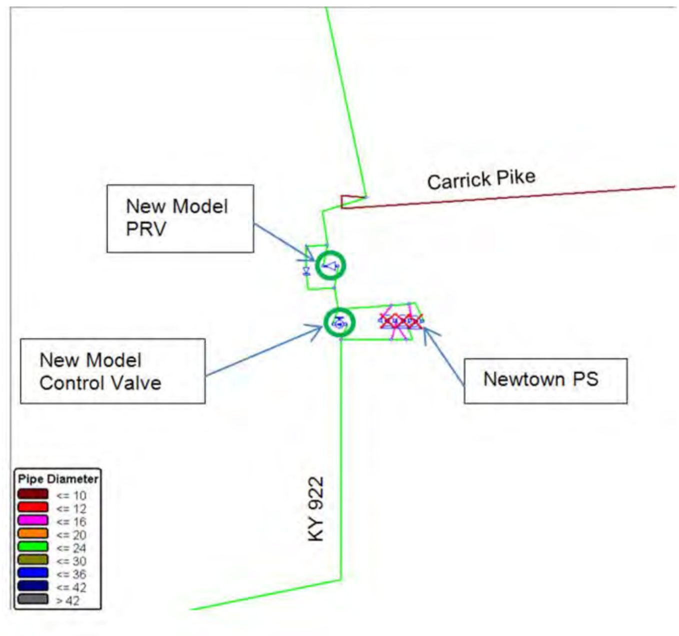


Figure 3.04-2 Proposed Newtown PS Valve Improvements

C. Isolation of the Briar Hill Tank Pressure Zone

KAW indicated the Briar Hill tank service area suffers from high water age compared to the rest of the Central Division system. KAW also indicated difficulties turning the tank over in an appropriate amount of time. Several isolation valves were added to the model to isolate the Briar Hill tank and improve tank turnover. Isolation valves were included on the 8-inch line on Muir Station Road before its intersection with the 12-inch line on Paris Pike. Another isolation valve was added at the intersection of Winchester Road and North Cleveland Road to maintain a loop in the Briar Hill tank service area and to create an additional loop. The valve at Winchester Road and North Cleveland Road was originally modeled as a check valve. However, under low demand scenarios it performs better as an isolation valve. Therefore, a control valve is the preferred option to allow automated control under various demand conditions. See Figure 3.04-3 for the location of the isolation valves in the model.

D. Rotating Fill and Drain Cycle for Ground Storage Tanks

Currently, KAW targets to fill and drain each of the eight ground storage tanks in the central gradient of the system by one-third each day to turn each tank over completely in three days. KAW staff have indicated this is challenging operationally. The revised operation will place the eight ground storage tanks into three separate cycle groups and operate them on a three-day rotating fill and drain sequence intended to maintain similar or improved tank turnover (water age) while simplifying operation of the system.

Two fill and drain sequences were considered for the revised three-day ground storage tank operation. Sequence one has the tank cycle groups completely drain on one day, completely fill the next day, and remain full the following day. The second sequence considered would have the tank cycle groups completely drain on one day, remain empty the next, and completely fill the following day. Sequence two could potentially improve water age by decreasing the amount of time the water sits idle in each ground storage tank. However this change amounts to only one additional day of idle storage. Sequence one was selected because at any given time, sequence one allowed for more volume in ground storage for use for a temporary high demand or fire flow.

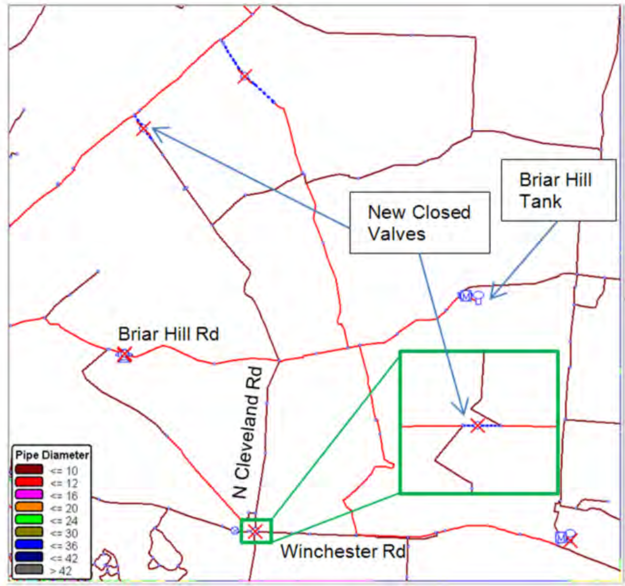


Figure 3.04-3 Proposed Briar Hill Tank Service Area Valves

The total volume of ground storage tanks is approximately 17 million gallons (MG) in the central gradient of the system. Tanks were divided into three cycle groups for the three-day rotation with similar total volumes in each cycle group.

Several options were also considered when selecting which ground storage tanks would be in each tank cycle group. The first option considered was grouping the tanks from south to north or east to west to simplify the tank cycle group operation. For example, Clays Mill 1 and 2 would be one group as the southernmost tanks, Parkers Mill, York Street, and one of the Cox Street tanks would be the second group, and Hume Road, Mercer Road, and the remaining Cox Street tank would be the third group. The second option placed tanks in different geographic areas of the system on the same tank cycle group. The second option was selected for analysis because it more effectively distributed the hydraulic impact of the tanks across the system and did not leave a particular area of the system with low storage volume. Table 3.04-1 shows the final tank cycle groups selected for analysis. The two Clays Mill tanks had previously been modeled as one larger tank. The model was modified to separate it into two tanks for the revised operation. Additional improvements may be required to operate Clays Mill tanks independently.

Tank Name	Volume (MG)
TANK CYCLE GROUP 1	
Clays Mill 1	3.0
York Street	1.0
Mercer Road	2.0
Subtotal	**6.0**
TANK CYCLE GROUP 2	
Parkers Mill	3.0
Hume Road	3.0
Subtotal	**6.0**
TANK CYCLE GROUP 3	
Clays Mill 2	3.0
Cox Street Elevated	1.0
Cox Street Ground	1.0
Subtotal	**5.0**
Total	**17**

Table 3.04-1 Tank Cycle Groups

3.05 CENTRAL DIVISION REVISED OPERATION HYDRAULIC RESULTS

Table 3.05-1 compares the target system demand and WTP production to the modeled values for the current system operation for a current average demand day.

Description	Target Values		Modeled Values	
	MGD	GPM	MGD	GPM
System Demand	46	31,900	45.3	31,500
KRS1 Production	24	17,100	24.5	17,000
KRS2 Production	8	5,600	8.5	5,900
RRS Production	14	9,700	15.1	10,500

Table 3.05-1 Current Average Day Demand and WTP Production

As shown in Table 3.05-1, WTP production and system demand modeled values are similar to target values for an average demand day. Therefore, model results reasonably represent actual WTP production and system demand for the target average demand day.

The following summarizes a comparison of modeled results of the current operation and the alternative operation discussed in Section 3.04.

Figures 3.05-1 through 3.05-4 display the average pressure and HGL for the current and revised operation models. Figures 3.05-5 through 3.05-12 show the minimum and maximum pressure plots for the areas of concern discussed in Section 3.03. Figures 3.05-13 through 3.05-18 show tank levels of the remaining tanks in the Central Division system for the revised operation average day simulation. Refer to Figures 2.04-1 through 2.04-13 for the current operation modeled tank levels.

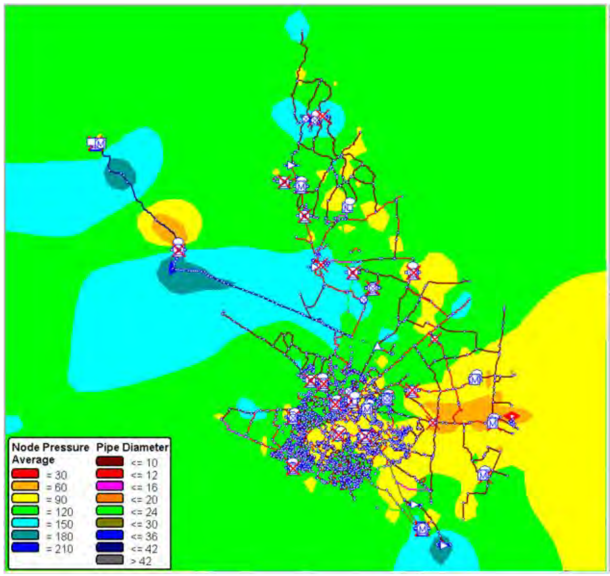


Figure 3.05-1 Current Operation Average Day Average Pressure

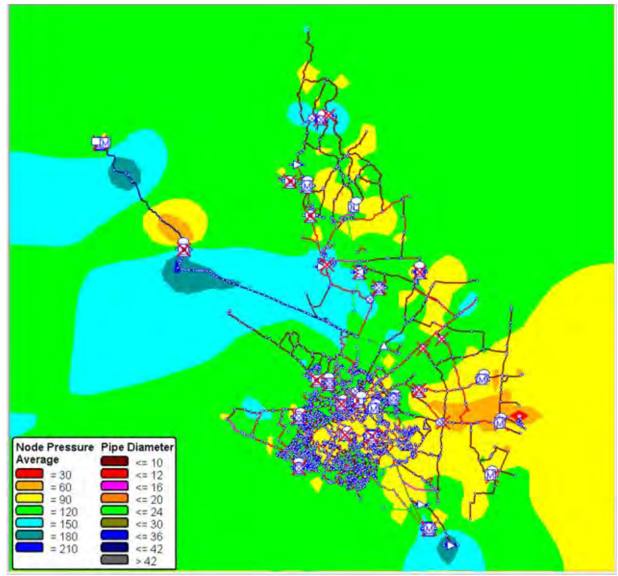


Figure 3.05-2 Revised Operation Average Day Average System Pressure

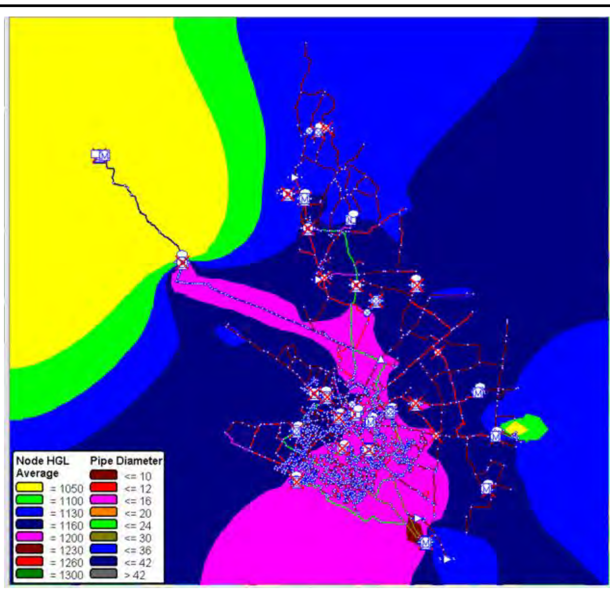


Figure 3.05-3 Current Operation Average Day Average HGL

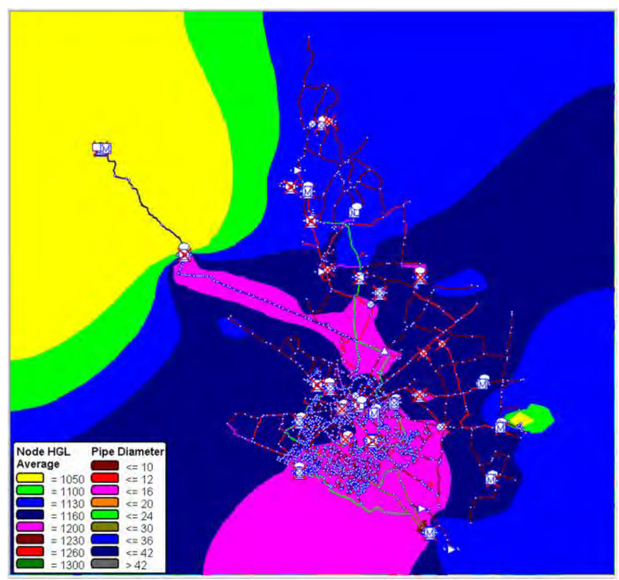


Figure 3.05-4 Revised Operation Average Day Average System HGL

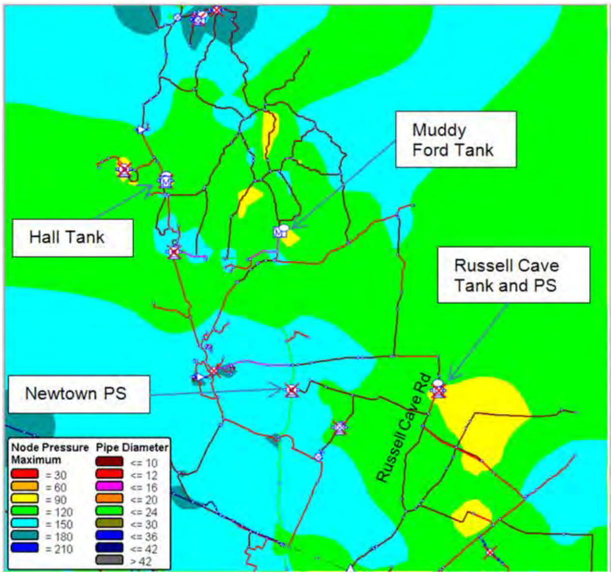


Figure 3.05-5 Current Operation Average Day Russell Cave Area Maximum Pressures

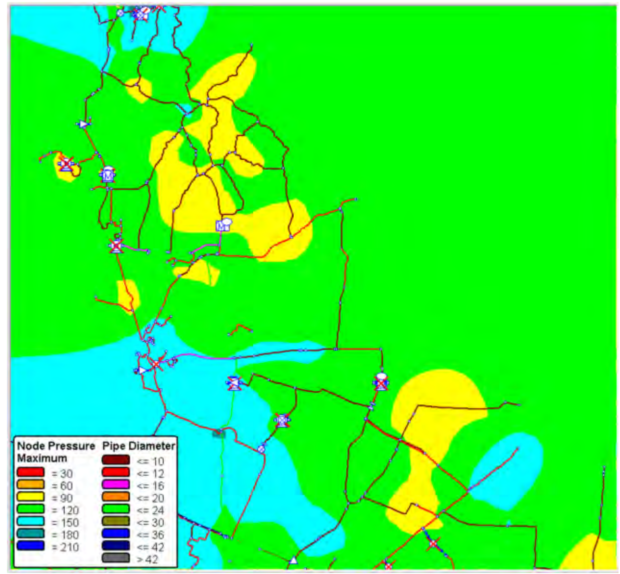


Figure 3.05-6 Revised Operation Average Day Russell Cave Area Maximum Pressure

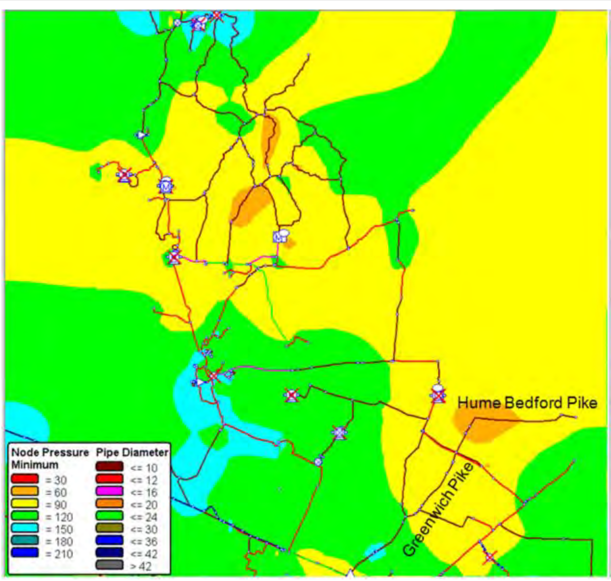


Figure 3.05-7 Current Operation Average Day Russell Cave Area Minimum Pressures

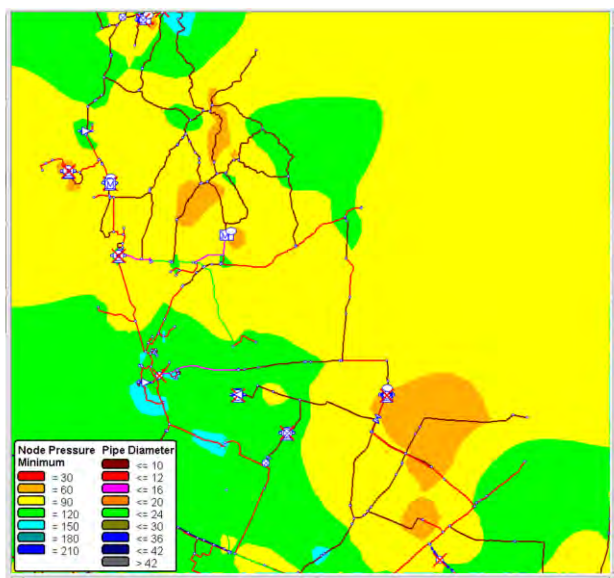


Figure 3.05-8 Revised Operation Average Day Russell Cave Area Minimum Pressure

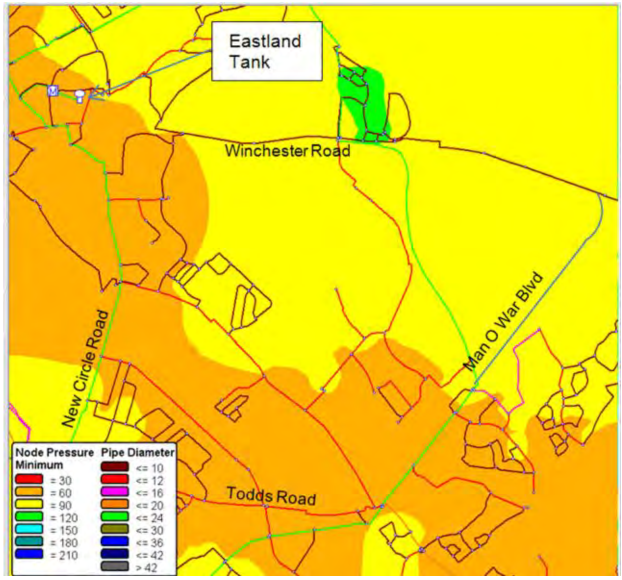


Figure 3.05-9 Current Operation Average Day Area of Concern Minimum Pressure

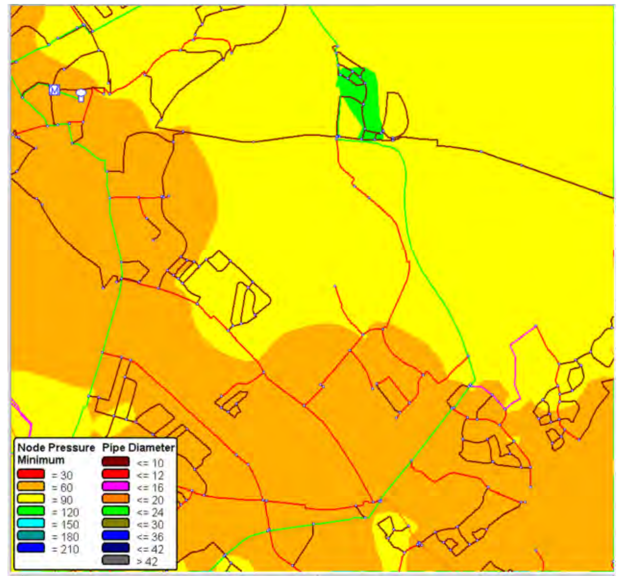


Figure 3.05-10 Revised Operation Average Day Area of Concern Minimum Pressure

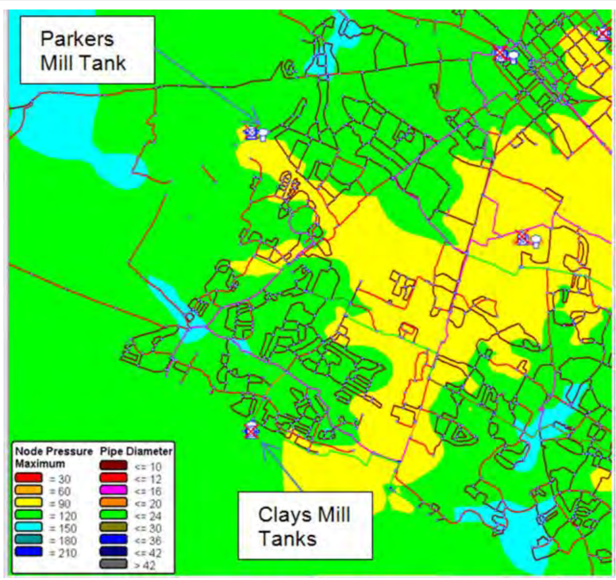


Figure 3.05-11 Current Operation Parkers Mill and Clays Mill Area Average Day Maximum Pressure

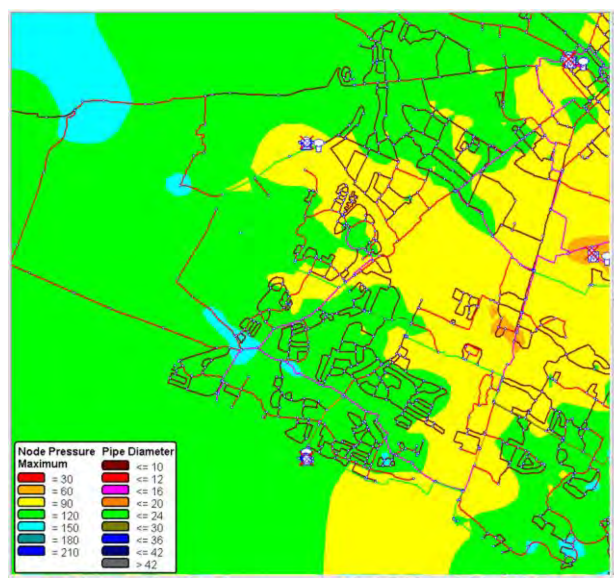


Figure 3.05-12 Revised Operation Average Day Parkers Mill and Clays Mill Area Maximum Pressure

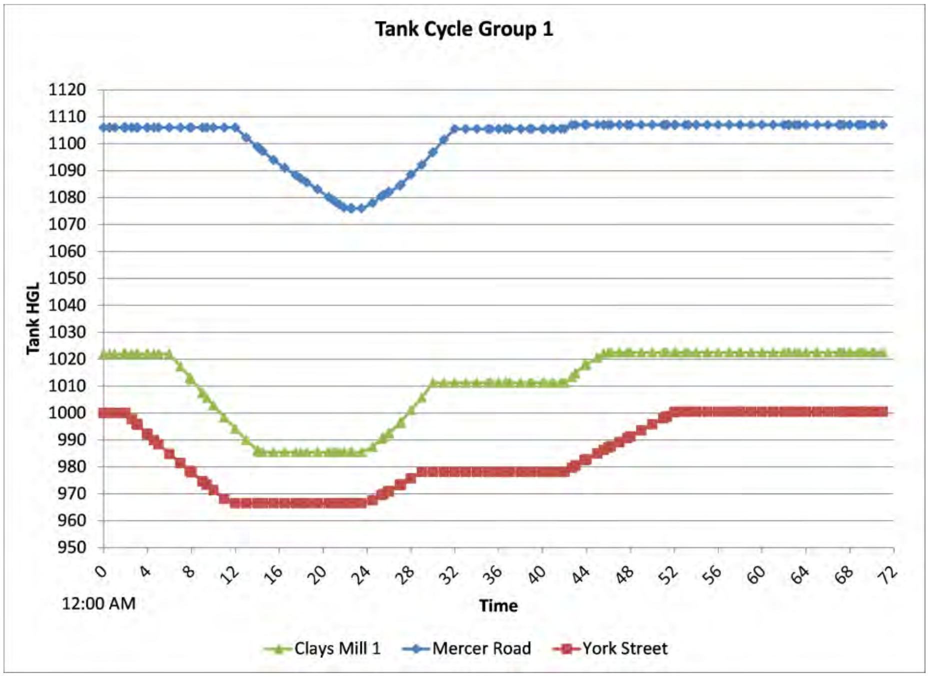


Figure 3.05-13 Tank Cycle Group 1
Revised Operation Average Day Tank Levels

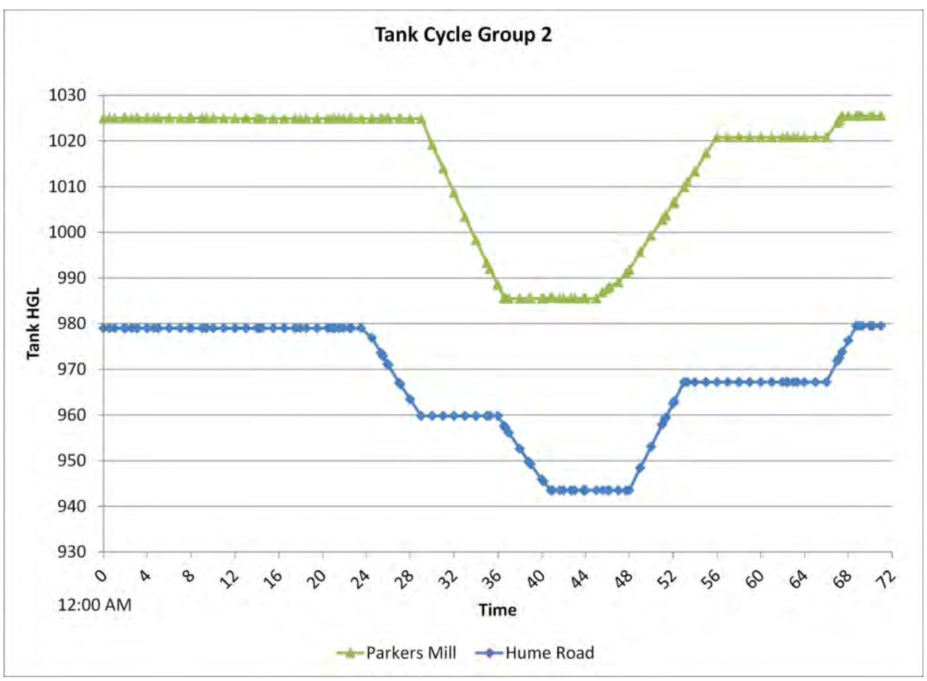


Figure 3.05-14 Tank Cycle Group 2
Revised Operation Average Day Tank Levels

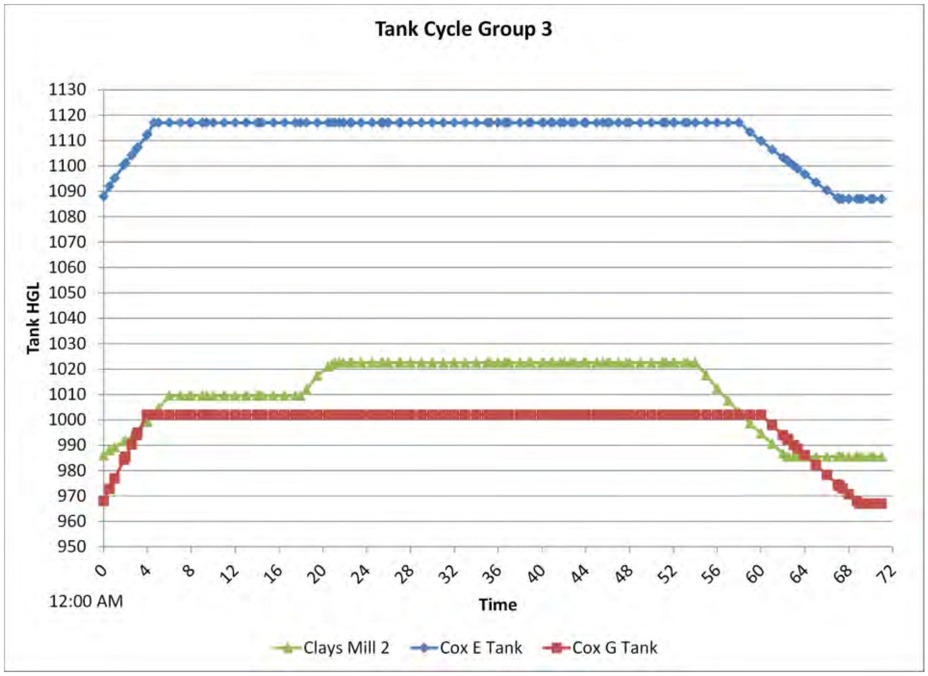


**Figure 3.05-15 Tank Cycle Group 3
Revised Operation Average Day Tank Levels**

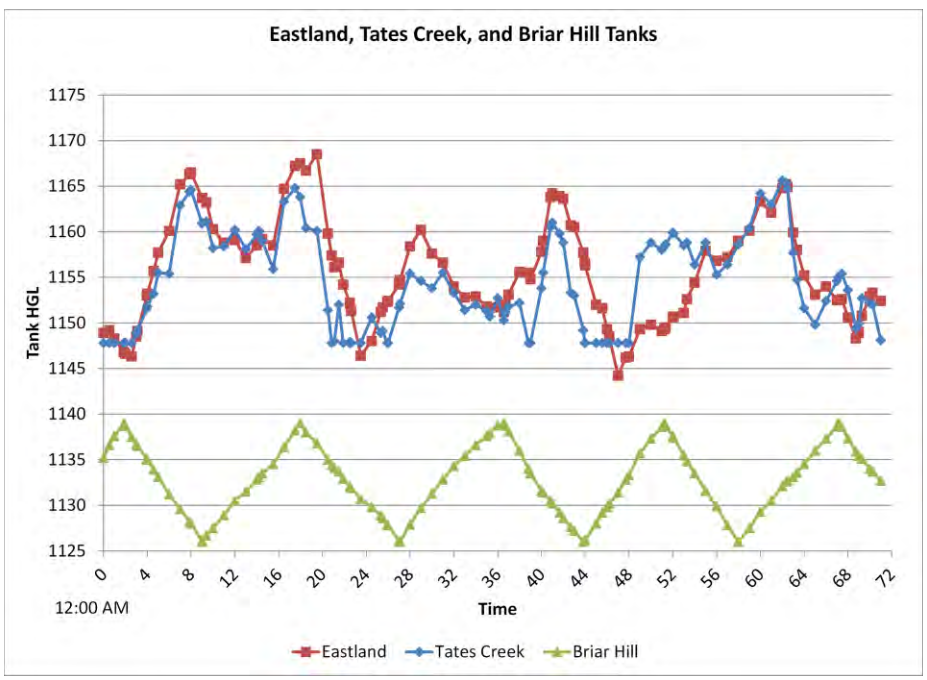


**Figure 3.05-16 Central Elevated Tanks
Revised Operation Average Day Tank Levels**

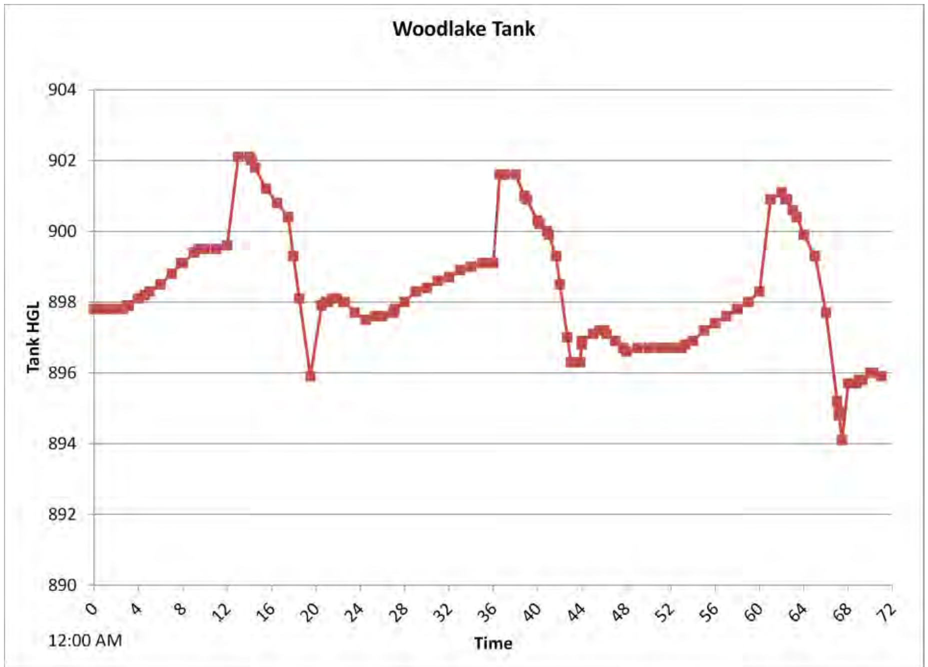


**Figure 3.05-17 Woodlake Tank
Revised Operation Average Day Tank Level**

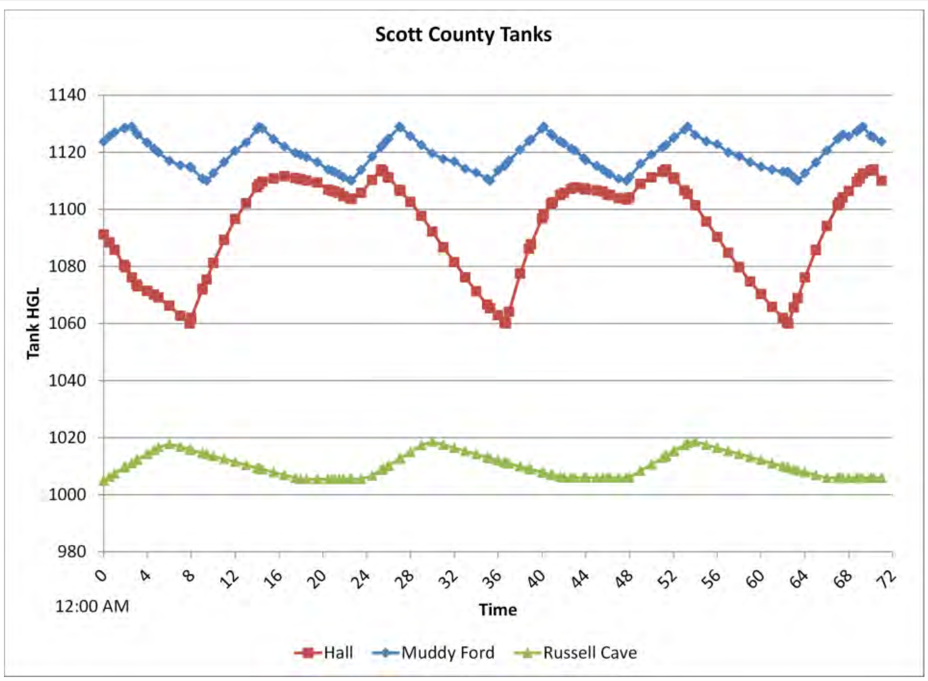


**Figure 3.05-18 Scott County Tanks
Revised Operation Average Day Tank Levels**

As Figures 3.05-1 through 3.05-4 show, overall average system pressures and average HGL results throughout the system for the revised operation are approximately equivalent to results observed in the current operation.

Under the current ground storage tank operation, the tanks are typically refilled one-third between 12 A.M. and 6 A.M. Under the revised tank operation, filling between 12 A.M. and 6 A.M. is not enough time to completely fill the ground storage tanks. Therefore, ground storage tanks were modeled with a second fill that occurred approximately between 6 P.M. and 12 A.M. to allow the tanks to completely fill.

Maximum pressures in the northern gradient of the system in the Muddy Ford and Russell Cave tank areas were reduced under the revised operation while maintaining similar minimum pressures as shown in Figures 3.05-5 and 3.05-6. Valve improvements at the Newtown PS allows the Muddy Ford tank to fill by opening the Newtown PS bypass main without requiring the pump station to operate.

Valve improvements successfully allowed Russell Cave tank to fill and drain without overpressurizing lines to the north of the PS while also maintaining minimum pressures during fill cycles. Russell Cave PS utilizes two large pumps and one smaller pump. Initially, the simulation was conducted using one large pump to drain the Russell Cave tank, which resulted in increased maximum pressures along Carrick Pike and Russell Cave Road (180 to 210 psi) compared to the revised operation (90 to 120 psi) and existing operation (90 to 120 psi) simulation. The simulation was rerun operating the smaller pump for a longer period of time. Using the small pump at the Russell Cave PS resulted in reduced maximum pressures conditions along Carrick Pike and Russell Cave Road near the Russell Cave tank while still providing enough flow to drain the tank one-third each day according to the intended operation of ground storage tanks in the system. The PSV valve on the Russell Cave tank fill line was set to a pressure setting of 40 psi to produce similar minimum pressure results at the intersection of Hume Bedford Pike and Greenwich Pike (30 to 60 psi) for the revised operation as the current operation. The pressure setting on the PSV could be further increased to maintain higher pressures near the intersection of Hume Bedford Pike and Greenwich Pike when the Russell Cave tank is filling.

The additional isolation valves in the Briar Hill area improved the rate of turnover in the Briar Hill tank. As shown in Figure 2.04-12, Briar Hill tank drained to a level of 1126 before the PS was called to run at approximately hour 14 of the existing system simulation. The revised operation drained the Briar Hill tank to a level of 1126 when the PS was called to run at approximately hour 9, as shown in Figure 3.05-6.

All tank cycle groups successfully filled and drained in the appropriate amount of time based on the revised ground storage tank operation. Additional flow was required from the WTPs during the third day when tank cycle group 3 drained into the system because tank cycle group 3 could only contribute 5 MG of volume into the system instead of 6 MG from tank cycle groups 1 and 2 because of the forced uneven split of tank volumes.

As indicated in Figures 3.05-9 and 10, minimum pressures in the area southeast of Eastland tank were approximately the same compared to the existing operation with a large portion of the area in the 30 to 60 psi range and some select high points experiencing pressure between 30 and 35 psi. Pressures between 25 to 30 psi were experienced at high points in this area on initial simulation runs where the Eastland tank HGL dropped below approximately 1135 feet. Adjustments were made to WTP high service operation and ground storage tank drain times to maintain slightly higher tank levels which resulted in increasing system pressures in the area to levels observed in the current operation simulation. Capital improvements could be implemented in this area to increase minimum pressures. See Section 4 for further discussion on potential capital improvements for the Central Division.

An additional 24-hour model simulation was performed using the revised operation where the Parkers Mill and Clays Mill tanks were placed in the same tank cycle group and were drained simultaneously to see if the model resulted in higher pressures as experienced by KAW staff. In general, the model did not show significantly increased pressures in the Parkers Mill and Clays Mill areas when both tank PSs were operating. With the Clays Mill and Parkers Mill booster pumps operating at the same time, pressures were generally less than 5 psi higher than the revised system operation when they were not operating together. Other factors could be contributing to the increased pressures in the system. Possible causes for the increased pressure are surge pressures from PS shutdown or a varied pumping scheme at the WTPs that was not used in this model simulation.

3.06 CENTRAL STORAGE VOLUME EVALUATION

An evaluation of the overall system capacity with an emphasis on storage was performed for each demand scenario for the revised operation. The evaluation was used to compare storage volume capabilities of the revised operation with existing operation.

AWWA M32 *Computer Modeling of Distribution Systems* provides a basis for determining total system volume required and divides storage requirements into three components: equalization storage, fire protection storage (if fire protection is provided by the system), and emergency storage. For the purposes of this evaluation, equalization storage was calculated as 15 percent of the system demand for the day, fire protection was calculated as 3,500 GPM for three hours (0.63 MG), and emergency storage was calculated as 50 percent of the sum of the equalization and fire protection storage. Table 3.06-1 shows the calculated storage requirements for the minimum, average, and maximum current day demand scenarios.

Description	**Minimum Demand Day**	**Average Demand Day**	**Maximum Demand Day**
System Demand (MGD)	30	46	72
System Storage Requirements			
Equalization Storage (MG)	4.50	6.90	10.80
Fire Flow Storage (MG)	0.63	0.63	0.63
Emergency Storage (MG)	2.57	3.77	5.72
Total Req. Storage	**7.70**	**11.30**	**17.15**

Table 3.06-1 Current Demand System Storage Requirements

Table 3.06-2 summarizes the available storage volumes for the tanks in the Central Division for the current and revised operation scenarios. For current operation available storage, calculations in Table 3.06-2 assume a worst-case scenario for ground storage tanks where they are currently all drained by one-third. Russell Cave available volume for the current operation is zero because it is on limited use for a worst-case scenario. For revised operation available storage, calculations in Table 3.06-2 assume a worst-case scenario where tank cycle group 1 (Clays Mill 1, York Street, and Mercer Road tanks), which is one of the two larger volume tank cycle groups, has been drained and is unavailable to contribute to useable storage. The revised operation also assumes Russell Cave tank is in use and is also drained by one-third to represent a worst-case scenario.

Tank	Total Tank Volume (MG)	Current Operation Available Volume (MG)	Revised Operation Available Volume (MG)
Clays Mill 1	3	2	0
Clays Mill 2	3	2	3
Parkers Mill	3	2	3
York Street	1	0.67	0
Mercer Road	2	1.33	0
Hume Road	3	2	3
Cox Street Elevated	1	0.67	1
Cox Street Ground	1	0.67	1
Russell Cave	1	0	0.67
Woodlake	3	3	3
Eastland	2	2	2
Tates Creek	0.5	0.5	0.5
Briar Hill	0.75	0.75	0.75
Muddy Ford	0.75	0.75	0.75
Hall	0.2	0.2	0.2
Total Volume	**25.2**	**18.54**	**18.87**

Table 3.06-2 Central Division Available Tank Storage Summary

Table 3.06-2 indicates that available storage for the current and revised operation scenarios exceeds the required storage for the minimum, average, and maximum demand scenarios shown in Table 3.06-1.

Under the current operating scenario, the ground storage tanks shown in Table 3.04-1 have a total storage volume of 17 MG and drain and fill one-third each day. Therefore, approximately 5.7 MG of ground storage per day is used by these tanks in the current operation. Revised operation uses 6 MG for two of the cycle days and 5 MG for one of the cycle days. Therefore, the revised operation will have almost the same effect on storage availability.

3.07 CENTRAL DIVISION CURRENT DEMAND DAY CONCLUSIONS

Model results indicate tank turnover and hydraulics will be similar or improve with the revised system operation while simplifying operation. Although there is no one single simulation that can cover the variability of system usage on a day-to-day basis, the revised operation suggests a promising approach to operating distribution facilities on a typical day.`

SECTION 4
CENTRAL DIVISION CAPITAL IMPROVEMENTS

4.01 AREAS OF INTEREST FOR CAPITAL IMPROVEMENT PROJECTS

In addition to the areas of concern identified in the alternative operation of the Central System in Section 3.02, other areas of interest were identified through modeling of the current Central System and known areas of concern by KAW. Areas of interest for water quality were identified by modeling the existing Central System under a current minimum demand scenario. Areas of interest for pressure and hydraulics were identified by modeling the existing Central System under a future maximum demand scenario. Capital improvements discussed further in Section 4.02 were selected to improve water quality and hydraulic conditions in these areas. Water quality modeling results for existing system and with proposed capital improvements are discussed in further detail in Section 4.04. Hydraulic modeling results for the existing system and with proposed capital improvements are discussed in further detail in Section 4.05. The following is a list of identified areas of interest. Figure 4.01-1 shows the location of the identified areas of interest within the Central Division model.

1. Briar Hill Tank Area–The Briar Hill tank service area was identified by KAW staff and through modeling as having slightly elevated water age compared to other portions of the system. KAW staff and modeling results also identified portions of the service area experiencing low pressures (below 45 psi). In addition, KAW operations staff indicated they have trouble turning the tank over.

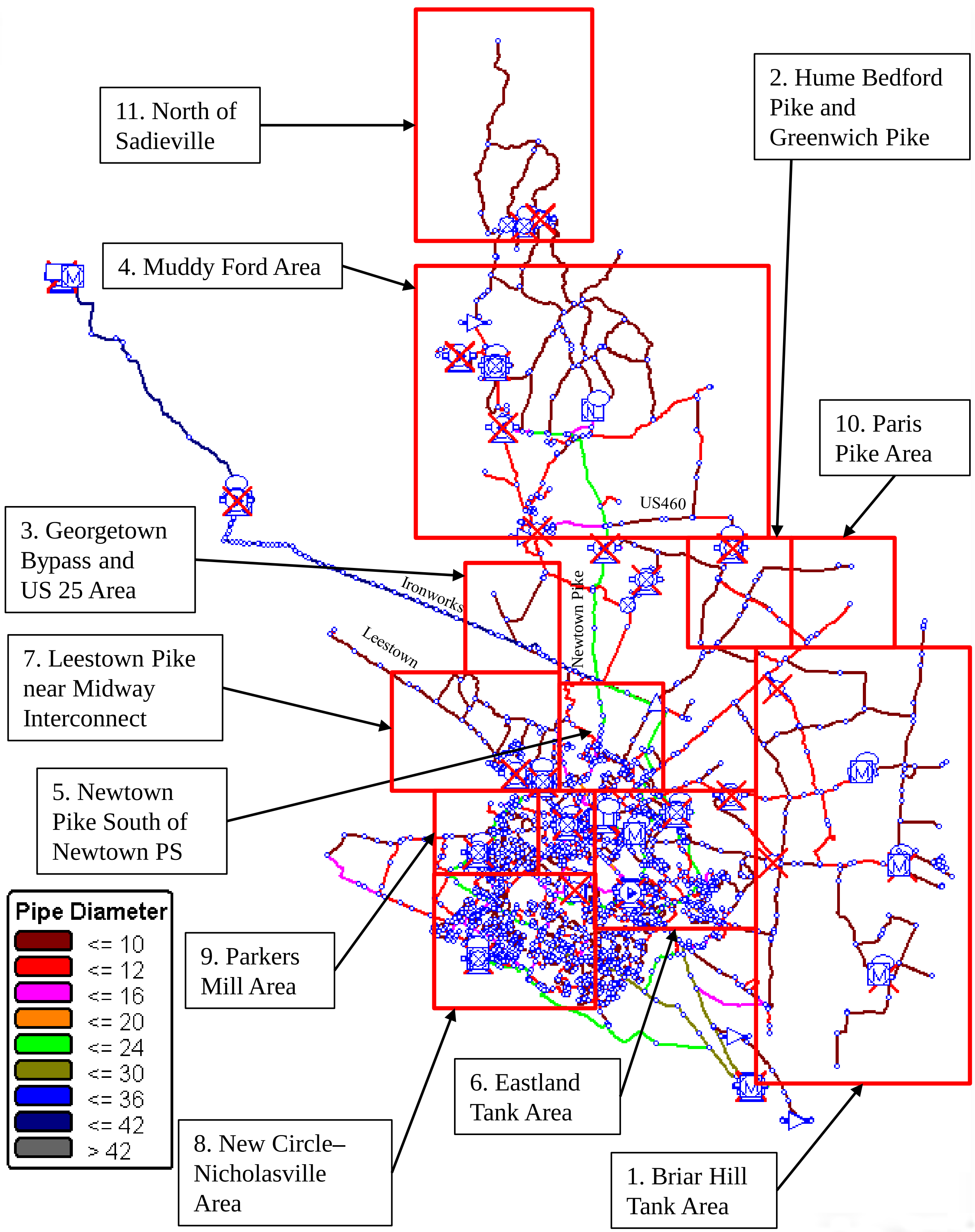
2. Hume Bedford Pike and Greenwich Pike Area–Modeling identified this area as having low pressures.

3. Georgetown Bypass and US 25 Area–Future demand is anticipated to increase in this area. Modeling of future demands indicated existing infrastructure was not sufficient enough to support predicted demand and maintain adequate system pressure (above 45 psi). KAW identified opportunities to increase the size of US 25 main south of Ironworks Road through highway improvements. KAW also identified improvements that could be made along Lisle Road as a means of providing redundancy to the Newtown Pike transmission main.

4. Muddy Ford Tank Area–Modeling identified areas around and north of the Muddy Ford tank with low pressures and elevated water age compared to other portions of the system.

5. Newtown Pike South of Newtown Pump Station–The 24-inch main that runs south from Ironworks along Newtown Pike is one of the primarily avenues for flow from KRS2 to reach the central portion of the system. The 24-inch main along Newtown Pike reduces to 16 inches between I-64 and New Circle Road, creating a potential bottleneck for flow from KRS2 to reach the central portion of the system.

6. Eastland Tank Area–As discussed in Section 3.02, areas southeast of the Eastland tank in the central portion of the system were identified by KAW staff and through modeling as areas experiencing low pressures. In addition, KAW identified the 6- and 8-inch main between Floyd Drive and Eastland Drive along New Circle Road as a problem. Access to the main is challenging because of bury depth, there have been several main breaks, and it is perceived to be a potential bottleneck.



AREAS OF INTEREST FOR
CAPITAL IMPROVEMENT PROJECTS
HYDRAULIC ANALYSIS FOR
COMPREHENSIVE PLANNING STUDY
KENTUCKY AMERICAN WATER
LEXINGTON, KENTUCKY

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FIGURE 4.01-1

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7. Leestown Pike near Midway Interconnect–The Central System currently provides water for the City of Midway. KAW identified the 8-inch main serving Midway as a potential bottleneck during high demand conditions.

8. New Circle-Nicholasville Area–Several high elevation areas along Nicholasville Road heading south out of downtown Lexington were identified through modeling. In addition, modeling identified several areas with elevated water ages compared to the rest of the system west of Nicholasville Road near New Circle Road and Man O War Boulevard.

9. Parkers Mill Tank Area–Modeling identified areas near the Parkers Mill tank with elevated water age compared to other portions of the system.

10. Paris Pike–Modeling identified areas of elevated water age compared to other portions of the system at the end of the Paris Pike main on the outskirts of the Central Division.

11. North of Sadieville–Modeling identified areas north of Sadieville at the edge of the Central Division with elevated water age compared to other portions of the system.

4.02 CENTRAL DIVISION PROPOSED CAPITAL IMPROVEMENTS

The following capital improvements were modeled in the Central System for the future minimum and maximum demand days to determine their impact on system water quality and hydraulics. Capital improvements have been divided based on their general geographic area within the Central Division. Capital improvements also include new valves discussed in Section 3 as part of the alternative operation of the Central Division. Main improvements were typically modeled as replacements and upgrades and not as a parallel main. If preferred, KAW could install the proposed capital improvements as parallel lines in these locations if space permits.

A. <u>Briar Hill Tank Area</u>

1. Operate Briar Hill tank within the top 10 feet of operating range.

2. Install a new 8-inch main connecting Bryan Station Road between Briar Hill Road and Muir Station Road (see Figure 4.02-1).

3. Replace the existing 4-inch main on Rockwell Road between Clintonville Road and Mimosa Drive with 6-inch main (see Figure 4.02-1).

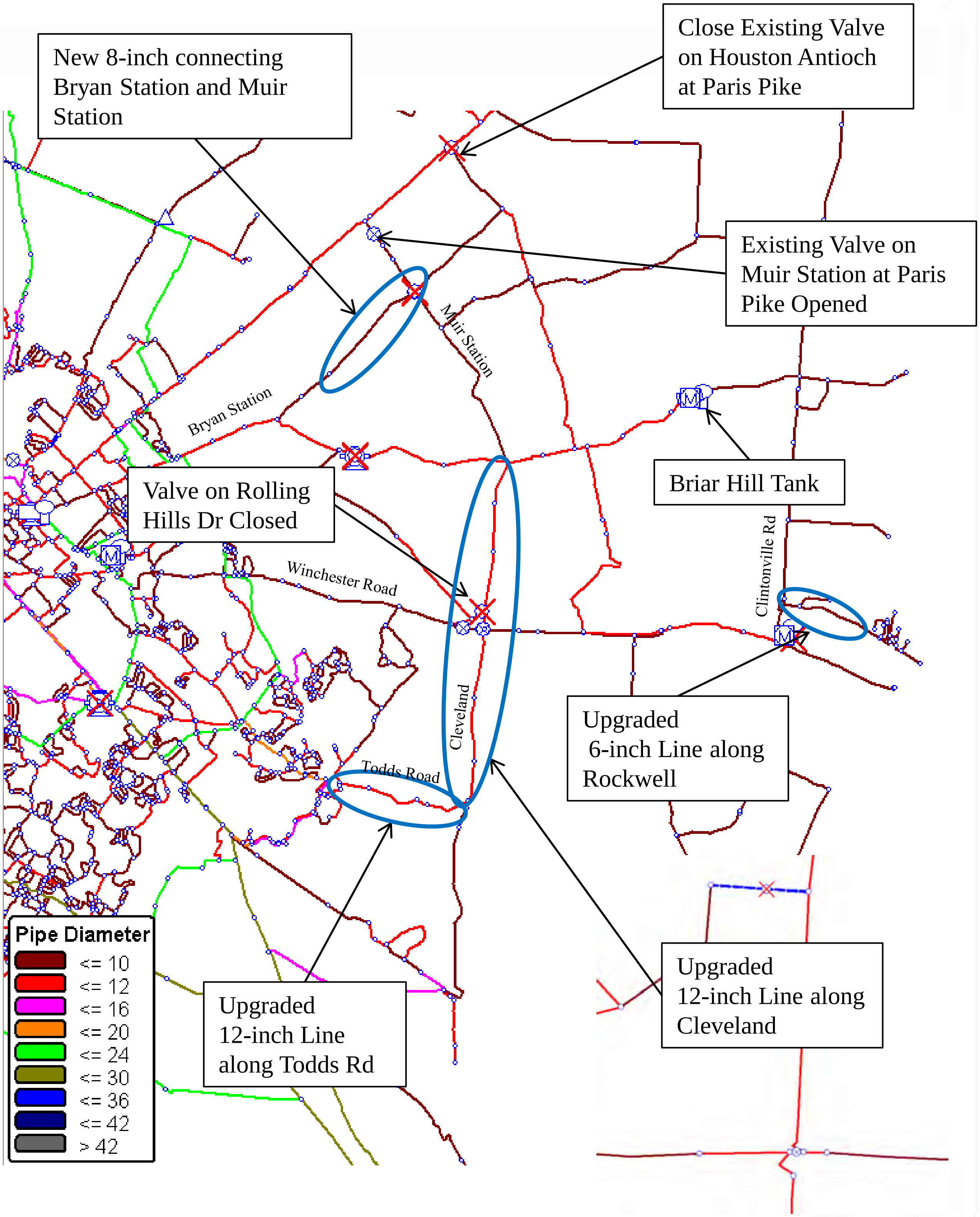
4. Open the valve on Muir Station Road at Paris Pike (see Figure 4.02-1).

5. Close the valve on Houston Antioch Road at Paris Pike (see Figure 4.02-1).

6. Close the valve on Rolling Hills Drive and North Cleveland (see Figure 4.02-1).

7. Upgrade existing mains on North Cleveland between Briar Hill and Todds Road to 12 inches. In addition, upgrade mains on Todds Road between North Cleveland and I-75 to 12-inches (see Figure 4.02-1).

8. Revised connectivity at intersection of Bryan Station Road and Muir Station Road (see Figure 4.02-2).



BRIAR HILL AREA
CAPITAL IMPROVEMENT PROJECTS
HYDRAULIC ANALYSIS FOR
COMPREHENSIVE PLANNING STUDY
KENTUCKY AMERICAN WATER
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FIGURE 4.02-1

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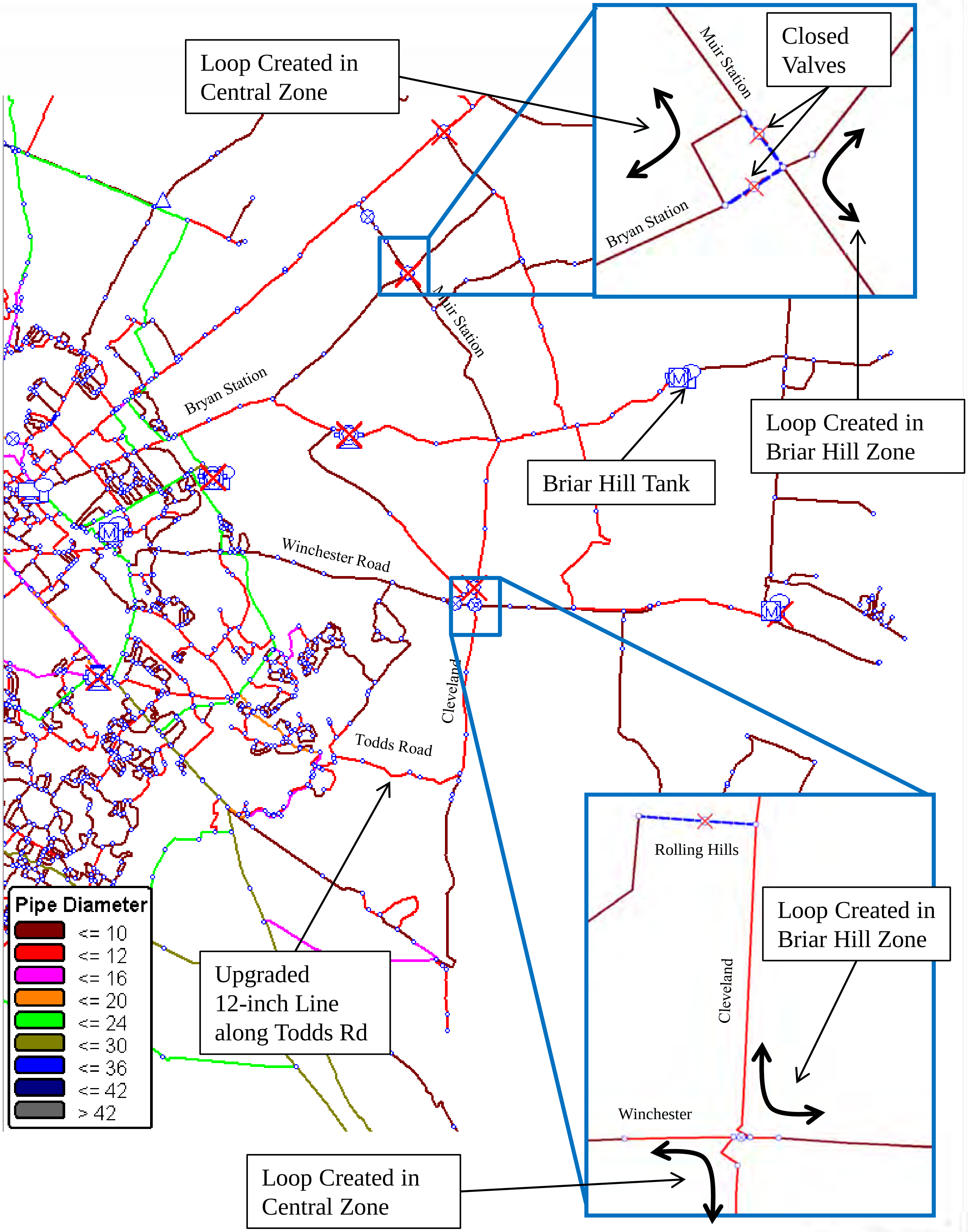

BRIAR HILL AREA IMPROVEMENTS
REVISED CONNECTIVITY
HYDRAULIC ANALYSIS FOR
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FIGURE 4.02-2

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a. Close valves on Bryan Station Road west of Muir Station and on Muir Station Road north of Bryan Station.

b. Install a small length of additional main added to create two loops at the intersection of Bryan Station Road and Muir Station Road.

9. Revised connectivity at the intersection of Winchester Road and North Cleveland Road. New control valve included at intersection to create two loops. This valve is modeled as a closed valve at all times for the future minimum demand scenarios and as a check valve for the future maximum demand scenarios(see Figure 4.02-2).

B. <u>Hume Bedford Pike and Greenwich Pike Area</u>

Upgrade the existing 3-inch main on Greenwich Pike between Hume Bedford Pike and Ferguson Road to an 8-inch main (see Figure 4.02-3).

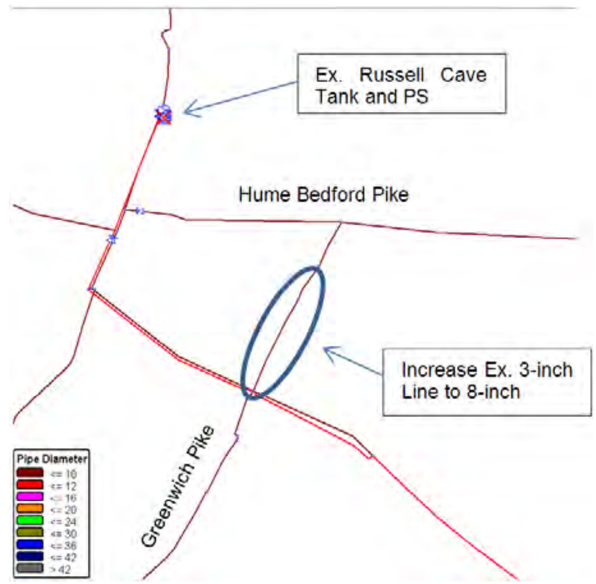


Figure 4.02-3 Greenwich Pike Capital Improvements

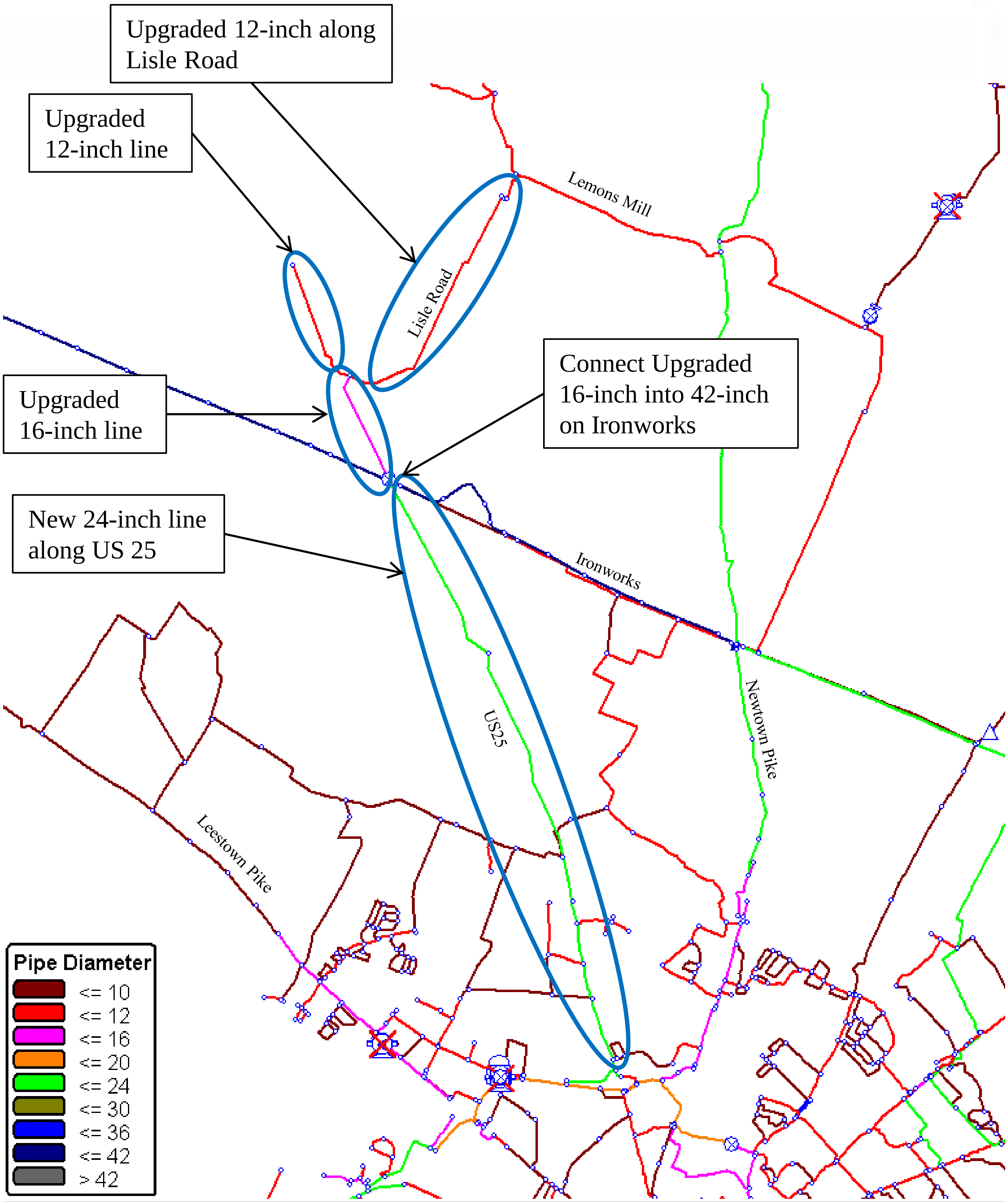
C. <u>Georgetown Bypass and US 25 Area</u>

1. Upgrade the existing 6-inch main on Lisle Road between US 25 and Lemons Mill Road to a 12-inch main (see Figure 4.02-4).

2. Connect into the 42-inch transmission main at US 25 and Ironworks with a new 16-inch main. The new 16-inch main will replace the existing 6-inch main on US 25 between Ironworks and Coleman Lane (see Figure 4.02-4).

3. Upgrade the existing 6- and 8-inch main on US 25 between Coleman Road and the Georgetown Bypass with a 12-inch main (see Figure 4.02-4).

4. Construct a new 24-inch main along US 25 from the 42-inch main on Ironworks to Kearney Ridge Boulevard connecting to the existing 16-inch main at the intersection of US 25 and Kearney Ridge Boulevard. It is anticipated that the new 24-inch main would be constructed in conjunction with the Kentucky Transportation Cabinet (KYTC) future highway realignment/relocation project for US 25. This was one of two options modeled to increase the flow capacity into the central portion of the system from KRS2 and the 42-inch main on Ironworks (see Figure 4.02-4). The other option is the Newtown Pike upgrade discussed in Section 4.02 E.



GEORGETOWN BYPASS AND US25 AREA CAPITAL IMPROVEMENT PROJECTS HYDRAULIC ANALYSIS FOR COMPREHENSIVE PLANNING STUDY KENTUCKY AMERICAN WATER LEXINGTON, KENTUCKY

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FIGURE 4.02-4

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D. Muddy Fork Tank Area

1. Replace the 2-inch and 4-inch mains on Barkley Road between Anderson Road and Morris Road with 6-inch mains (see Figure 4.02-5).

2. Install a new 6-inch main on KY32 between Burgess Smith Road and Davis-Turkey Foot Road (see Figure 4.02-5).

3. Install a control valve along I-75 north of Lemons Mill Road to reduce flow into Muddy Ford tank to allow turnover on a minimum demand day. Alternatively this could be accomplished by installing check valves on Cythiana Road north of I-75 and US 460 east of I-75 (see Figure 4.02-5).

E. Newtown Pike South of the Newtown PS

Upgrade the existing 16-inch main on Newtown Pike between I-75 and New Circle Road with a 24-inch main This was one of two options modeled to increase the flow capacity into the central portion of the system from KRS2 and the 42-inch main on Ironworks (see Figure 4.02-6). The other option is the new 24-inch main along US 25 discussed previously.

F. Eastland Tank Area

1. Operate Eastland tank in top 10 feet of operating range.

2. Increase the 6-inch main on New Circle Road between Eastland and Floyd Drive to 12-inch (see Figure 4.02-7).

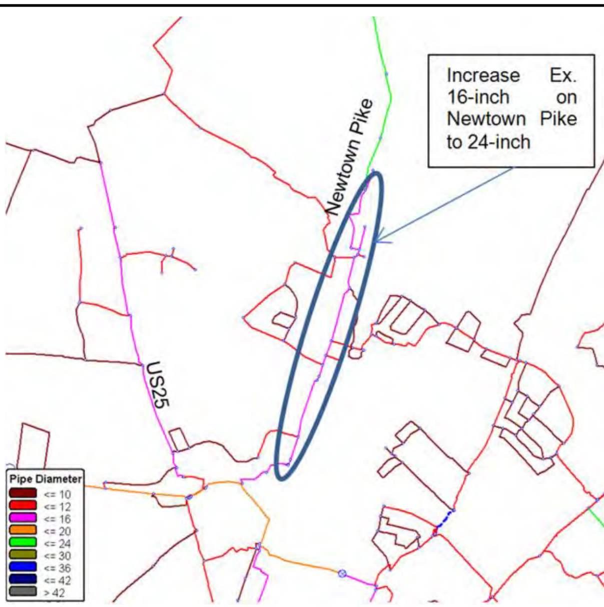


Figure 4.02-6 Newtown Pike Upgrade

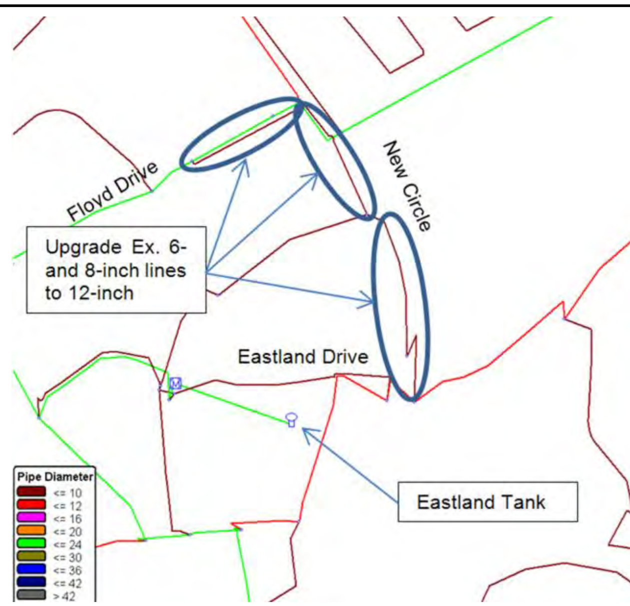
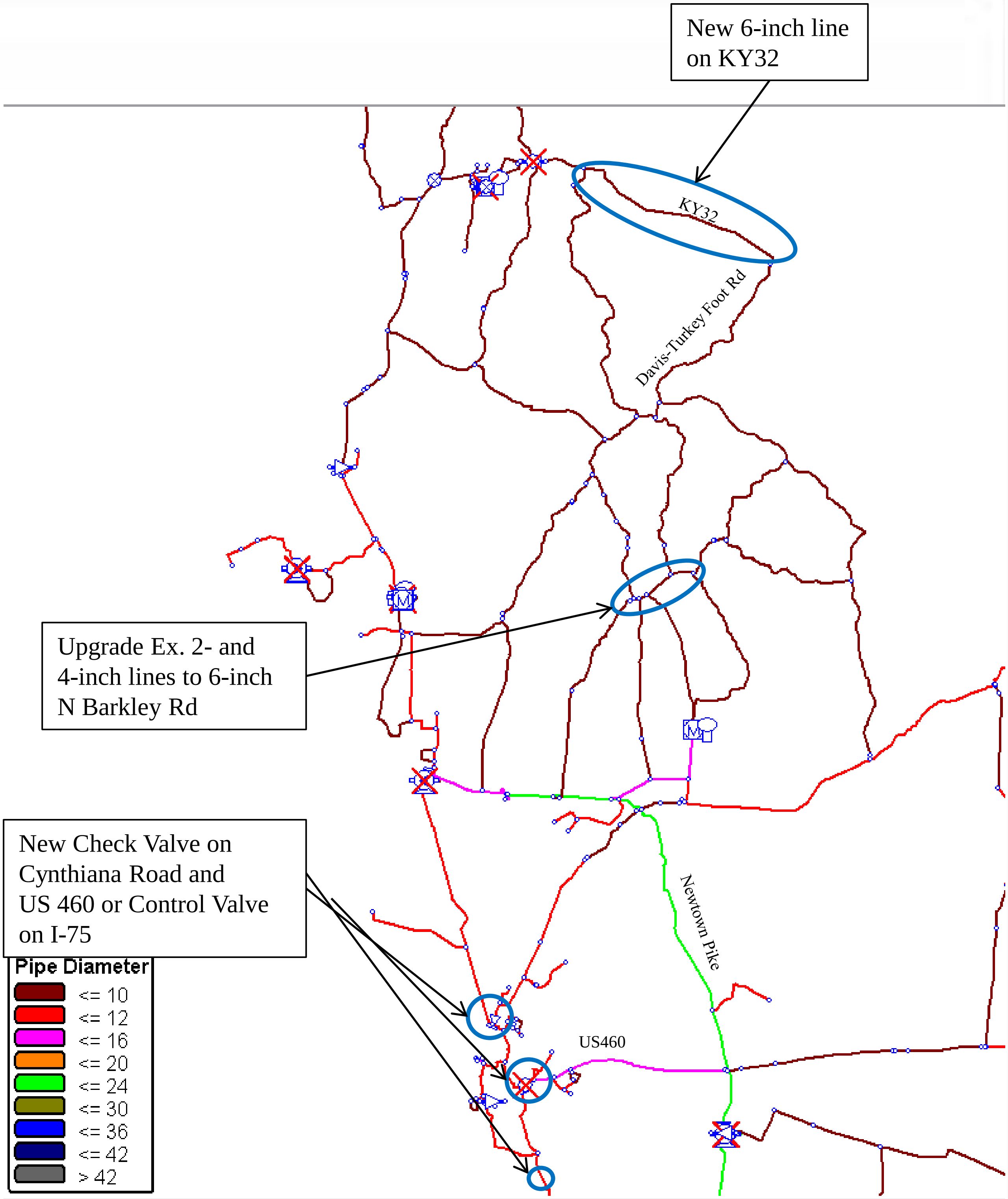


Figure 4.02-7 Eastland Tank Area Upgrades



MUDDY FORD TANK AREA
CAPITAL IMPROVEMENT PROJECTS
HYDRAULIC ANALYSIS FOR
COMPREHENSIVE PLANNING STUDY
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FIGURE 4.02-5

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G. Leestown Road Near Midway Interconnect

Increase the 6- and 8-inch mains on Leestown Road between Opportunity Way and Bradley Lane to 16-inch (see Figure 4.02-8).

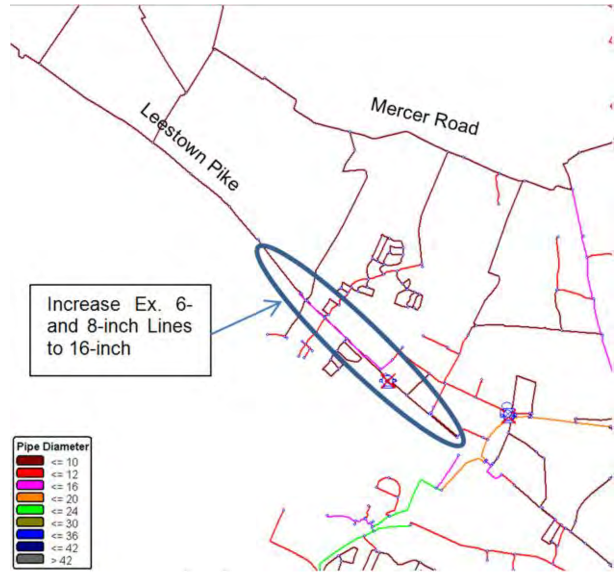


Figure 4.02-8 Leestown Road Upgrade

H. New Circle-Nicholasville Area

1. Install a new 8-inch main from the end of Sporting Court to Stone Road across New Circle Road (see Figure 4.02-9).

2. Install a new 8-inch main connecting existing 8-inch mains on Winthrop Drive across Man O War Boulevard (see Figure 4.02-9).

3. Install a new 6-inch main connecting Blackberry Lane to Willow Oak Circle and Mill Ridge Road (see Figure 4.02-9).

I. Parkers Mill Tank Area

An existing 8-inch main along Pine Needles Lane and Guilford Lane south of the Parkers Mill tank off Man O War Boulevard previously not included in the model was included in the model (see Figure 4.02-10). This change does not require any capital improvements.

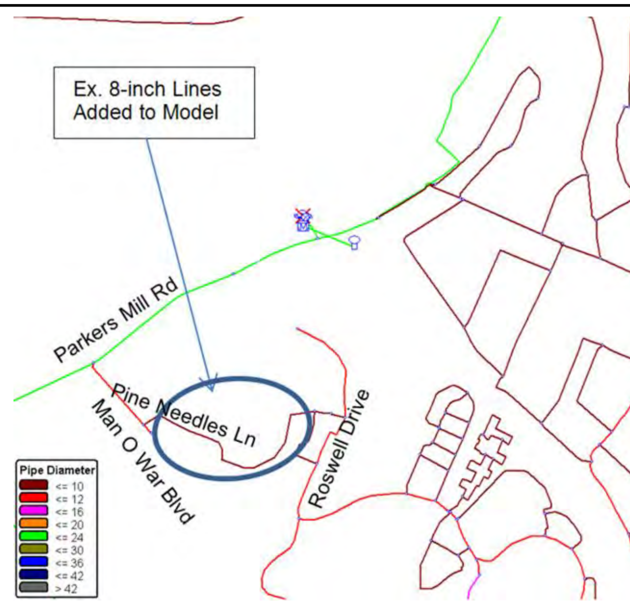
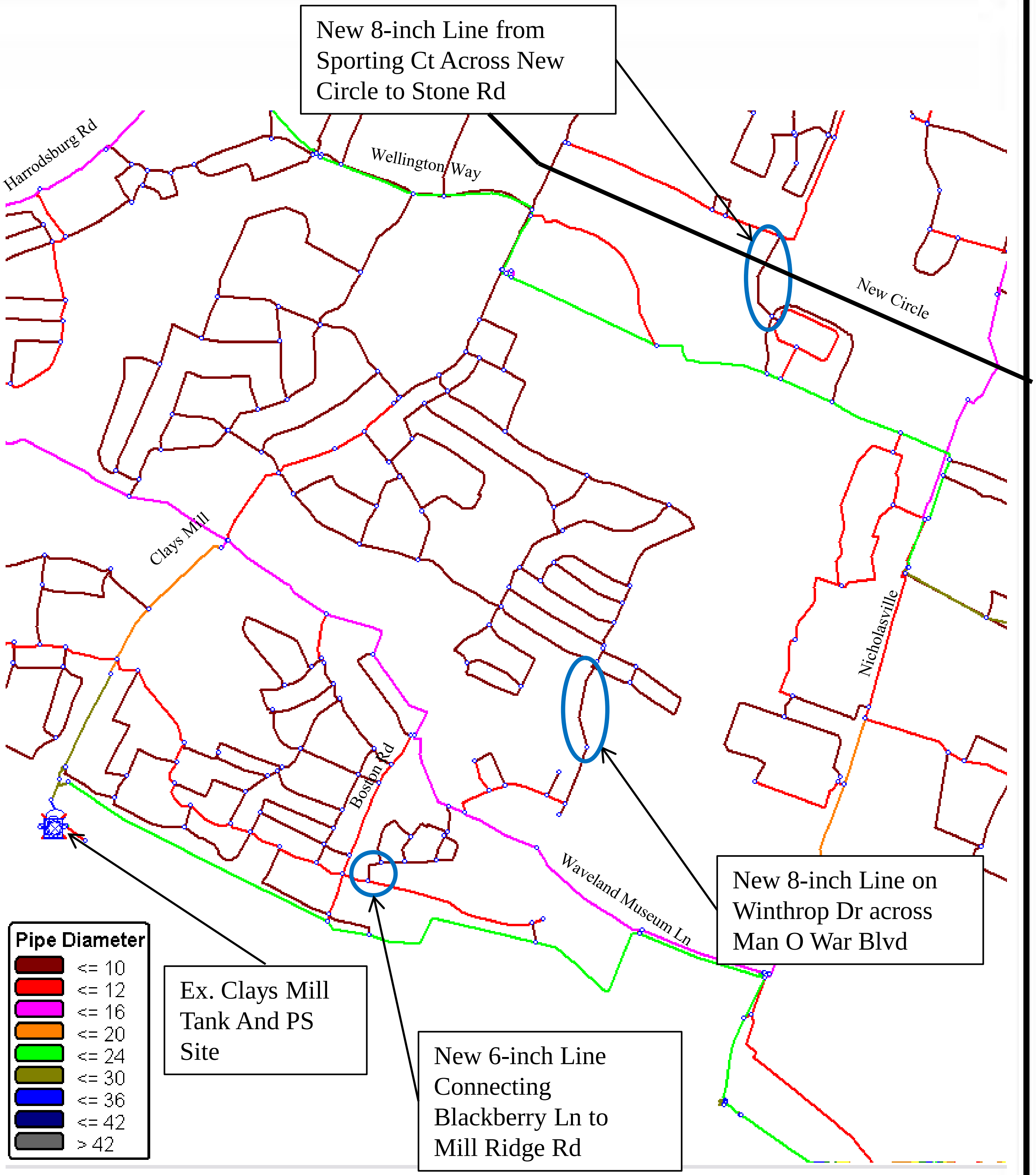


Figure 4.02-10 Parkers Mill Tank Area

J. Paris Pike

The modeled results showing elevated water age along Paris Pike is at the end of the 12-inch main on Paris Park on the edge of the Central Division's service area where mains cannot be easily looped. If water quality issues do exist in this area, KAW could discuss possible looping opportunities, water purchase to increase demand, or main flushing with the City of Paris.



NEW CIRCLE AND NICHOLASVILLE AREA CAPITAL IMPROVEMENT PROJECTS HYDRAULIC ANALYSIS FOR COMPREHENSIVE PLANNING STUDY KENTUCKY AMERICAN WATER LEXINGTON, KENTUCKY

STRAND ASSOCIATES®

FIGURE 4.02-9

5493.117

I. North of Sadieville

Modeling identified elevated water age north of Sadieville on the 8-inch main along US 25 at the Scott County and Grant County border. There are no other mains within the Central Division in this location to create additional loops. Potential loops could be created by connecting into Northern Division approximately 2 miles west or into other nearby water utilities near the end of the 8-inch main. Potential utilities to connect into near this area are the Georgetown Municipal Water and Sewer Service and the Corinth Water District. Creating loops by connecting into other nearby utilities would also require additional equipment to appropriately meter flows in and out of the looped areas.

4.03 CENTRAL DIVISION FUTURE DEMAND SCENARIOS

Future minimum demand scenarios were used to compare water quality conditions (water age) in the Central Division before and after capital improvements. Future maximum demand scenarios were used to compare the hydraulic conditions of the Central Division before and after capital improvements. Target future system demands for each scenario were supplied by KAW. Table 4.03-1 summarizes the future demand scenarios for the Central Division.

	Minimum Day		Average Day		Maximum Day	
Description	MGD	GPM	MGD	GPM	MGD	GPM
System Demand	31.5	21,875	50	34,700	80	55,555
Peak-Hour Demand	47.0	32,640	77	53,470	124	86,110
KRS1 Production	15.5	10,760	25	17,360	40	27,770
KRS2 Production	5.5	3,820	10	6,940	15	10,420
RRS Production	10.5	7,290	15	10,420	25	17,360

Table 4.03-1 Target Central Division Demand and WTP Production for Future Day Scenarios

The current maximum demand scenario had a system demand of 72 MGD, and the future maximum demand scenario had a system demand of 80 MGD, for a total increase in demand of 8 MGD between current and future maximum demand conditions. Of the 8 MGD, 6 MGD was allocated to seven areas identified by KAW that are expected to see increased development and demand. The remaining 2 MGD demand increase was allocated by increasing peaking factors across the Central Division. Demand allocation for the minimum and average future day scenarios was allocated similarly to the maximum future demand day, with 25 percent of the demand increase from the current to the future scenario allocated by increasing peaking factors across the system and the remaining 75 percent allocated to the seven areas identified by KAW.

A specified percent provided by KAW of the 6 MGD increase was allocated to each of the seven areas. Demand within the seven areas was further divided to one or more junctions depending on the size of the area and location of water mains. For areas where demand was allocated to more than one junction, demand was distributed evenly among the junctions. Table 4.03-2 summarizes the seven areas and the additional demand allocated to each area. Figures 4.03-1 through 4.03-7 show the locations of the junctions allocated additional demand.

Area	Area Description	Minimum Demand Day Additional Demand (MGD)	Average Demand Day Additional Demand (MGD)	Maximum Demand Day Additional Demand (MGD)
1	McGrathiana Pkwy and Aristides Blvd	0.225	0.6	1.2
2	North side of I-64/I-75 between Russell Cave Rd and Newtown Pike	0.1125	0.3	0.6
3	Area roughly enclosed by Winchester Rd, Deerhaven Ln, and I-75	0.225	0.6	1.2
4	Residential areas off of Hayes Blvd and Athens Boonesboro Rd	0.225	0.6	1.2
5	Area roughly enclosed by Delong Rd, Tates Creek Rd, and Armstrong Mill Pike	0.1125	0.3	0.6
6	Downtown area roughly enclosed by Euclid Ave, Jefferson St, Fourth St, Walton Ave, and Clay Ave	0.1125	0.3	0.6
7	Lexington Rd north of its intersection with Coleman Ln	0.1125	0.3	0.6
	TOTALS	**1.125**	**3**	**6**

Table 4.03-2 Central System Future Scenarios Additional Demand Descriptions

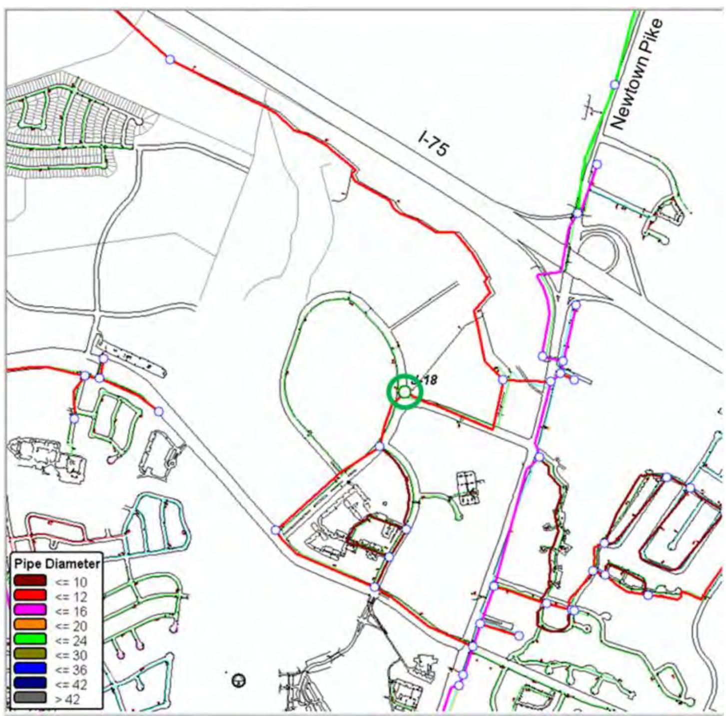


Figure 4.03-1 Future Demand Area 1 Allocation

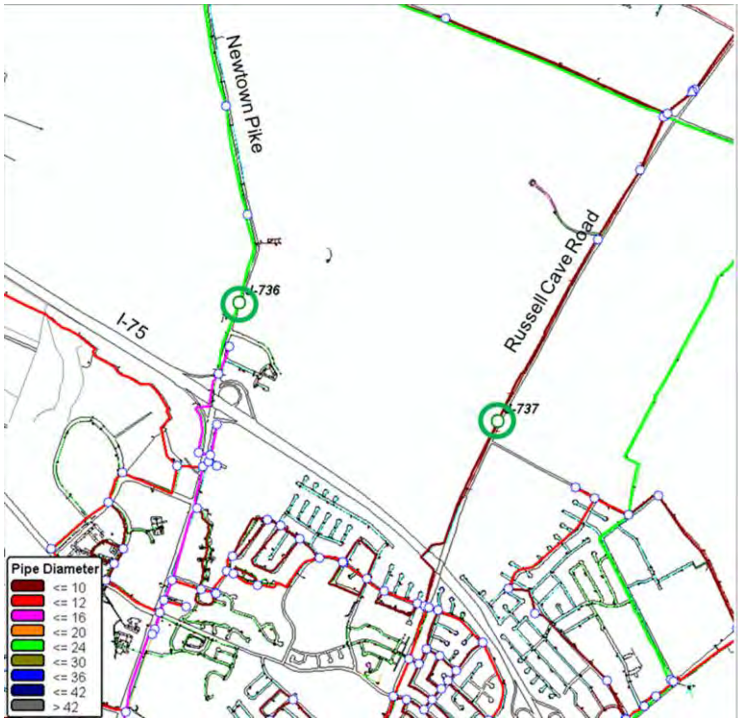


Figure 4.03-2 Future Demand Area 2 Allocation

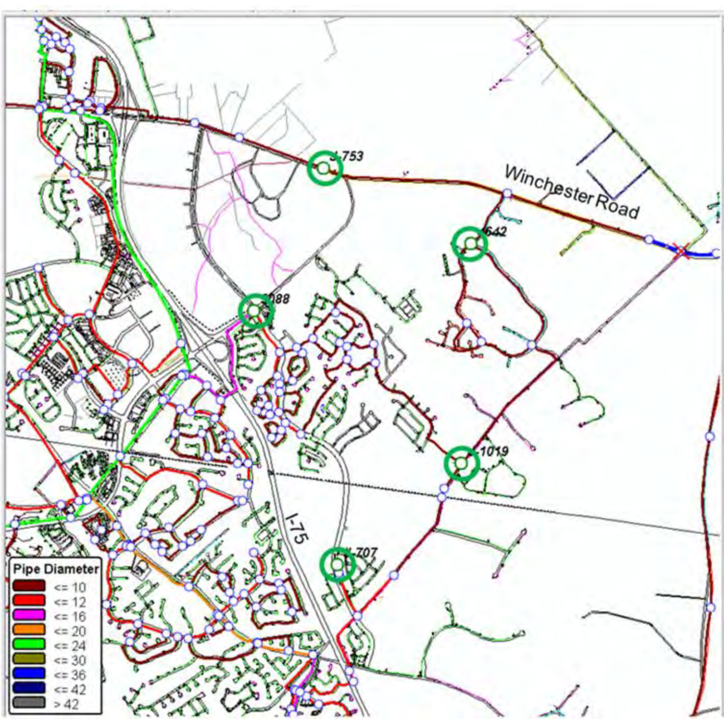


Figure 4.03-3 Future Demand Area 3 Allocation

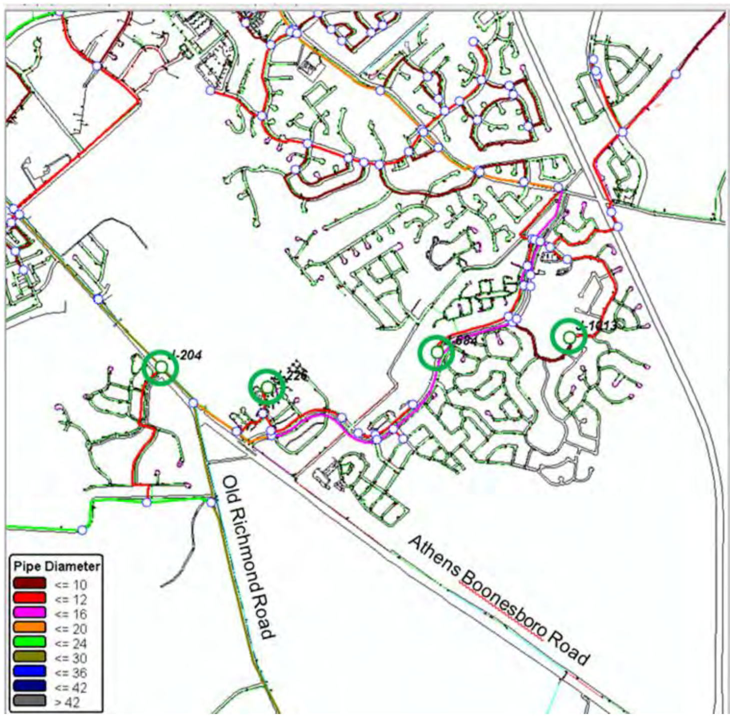


Figure 4.03-4 Future Demand Area 4 Allocation

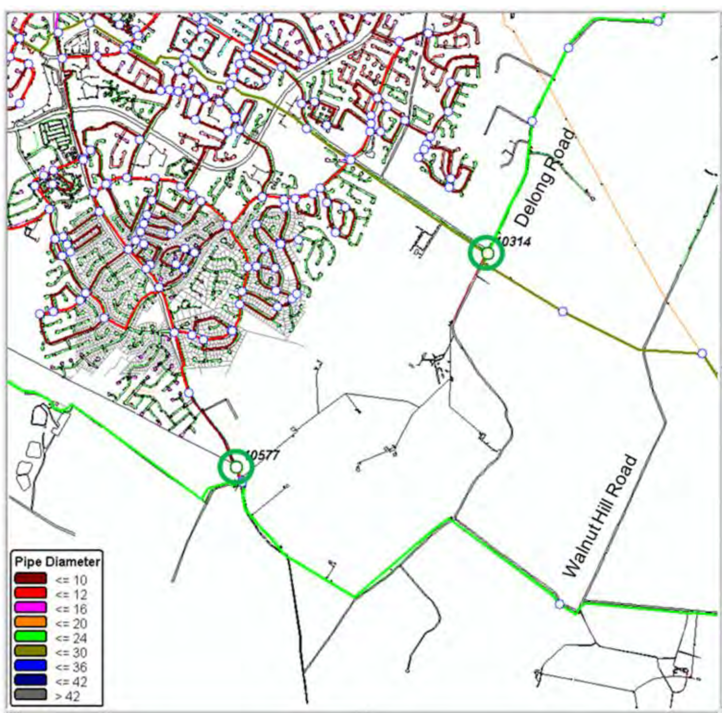


Figure 4.03-5 Future Demand Area 5 Allocation

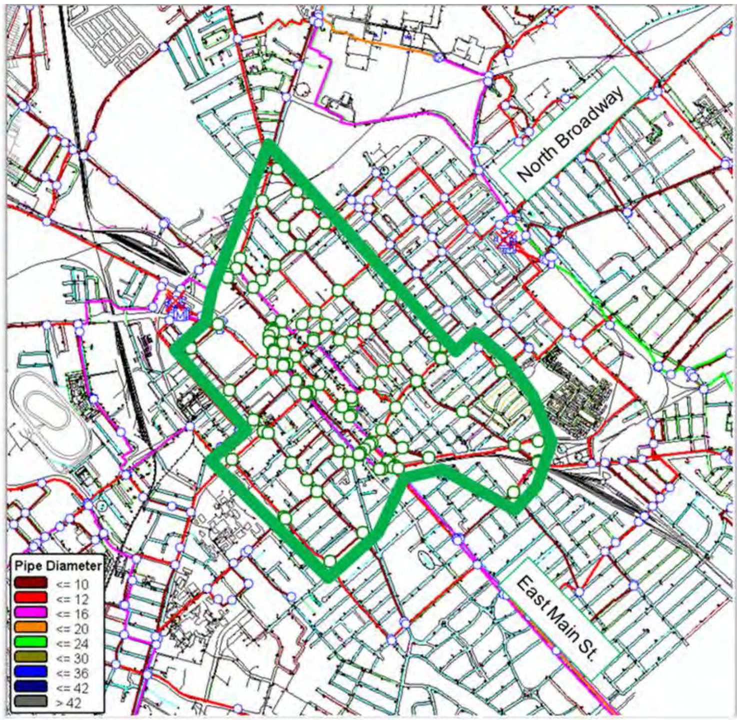


Figure 4.03-6 Future Demand Area 6 Allocation

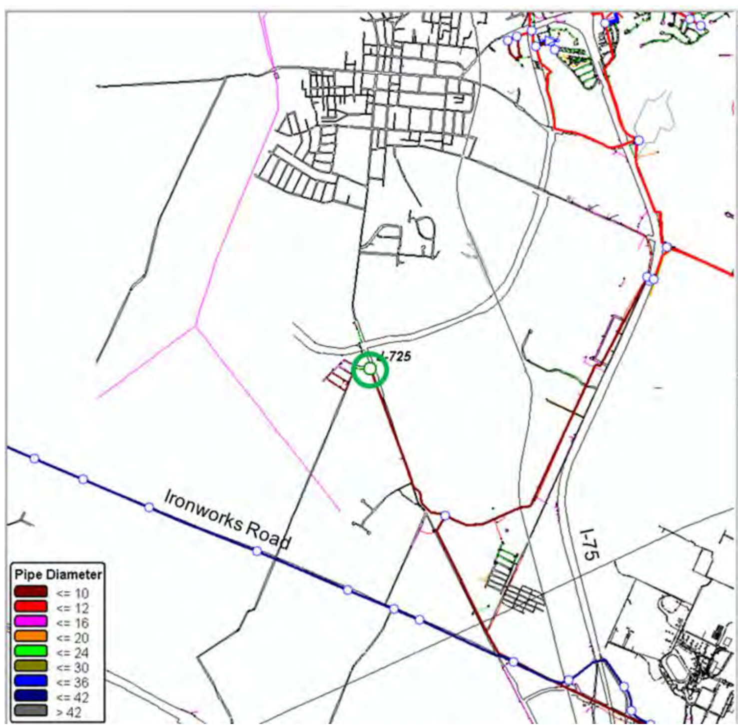


Figure 4.03-7 Future Demand Area 7 Allocation

4.04 MINIMUM DEMAND SCENARIO RESULTS

Average modeled water age for all junctions in the current minimum demand day model is shown in Appendix C. Average modeled water age for all junctions in the future minimum demand day model with proposed improvements is shown in Appendix D. The following minimum demand scenario water age modeling discussions do not comment on all areas with high water age shown in the figures. Areas shown with high water age in the figures that are not discussed are tanks, mains, pumps, or other infrastructure that were not in operation during the water age simulations and, therefore, show unrealistic elevated water age results. Only areas that show elevated water age and are in operation were targeted for capital improvements and are discussed in this section. Hours were the chosen unit for all modeled water age scenarios and are to be displayed in all figures associated with water age results.

It is important to note that the following water age results do not directly indicate where water quality conditions are poor, as there is no set water age where water quality is considered poor. Increased water age increases the potential for such things as disinfection byproducts (DBPs) formation but the water age at which DBPs forms depends on a number of outside factors that cannot be included in the following model simulations. The intended use of the water age results is to show relative water ages throughout the system to identify areas that would benefit more from capital improvements aimed at reducing water age.

Figures 4.04-1 and 4.04-2 show the overall average water age results for the current minimum demand day model and the future minimum demand day model with capital improvements, respectively.

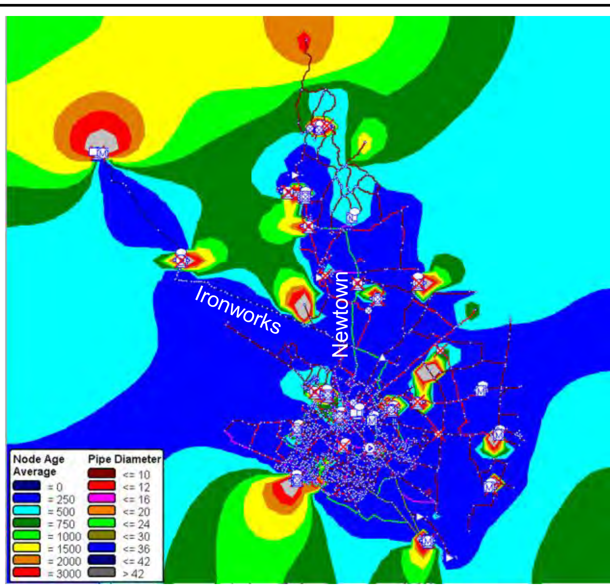


Figure 4.04-1 Current Minimum Demand Day Average Water Age

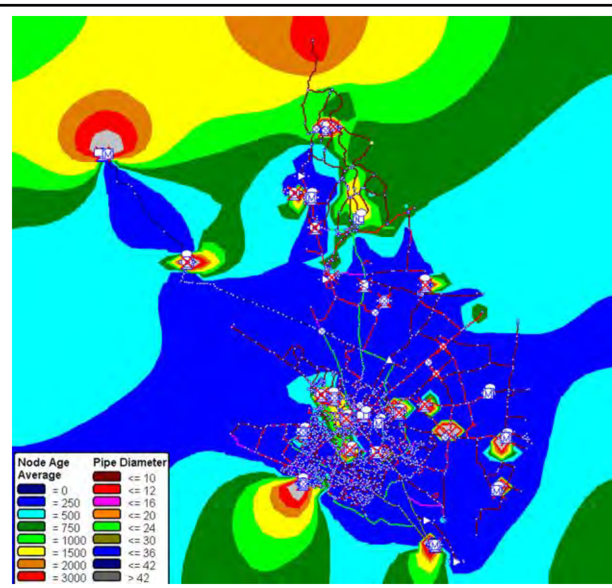


Figure 4.04-2 Future Minimum Demand Day Average Water Age with Capital Improvements

A. Briar Hill Tank Area

Figures 4.04-3 and 4.04-4 show the average water age results before and after implementing capital improvements for the Briar Hill area, respectively. Note that the water age colors for Figures 4.04-3 and 4.04-4 scale differently than other water age figures to capture the smaller differences in water age in the Briar Hill tank area.

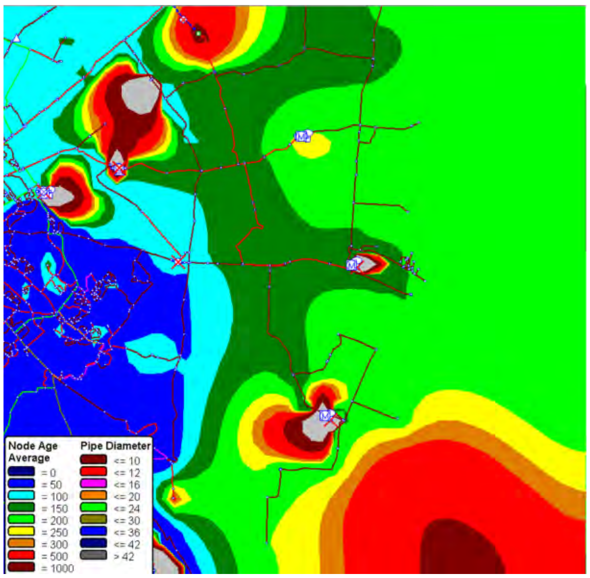


Figure 4.04-3 Briar Hill Tank Area Average Water Age without Capital Improvements

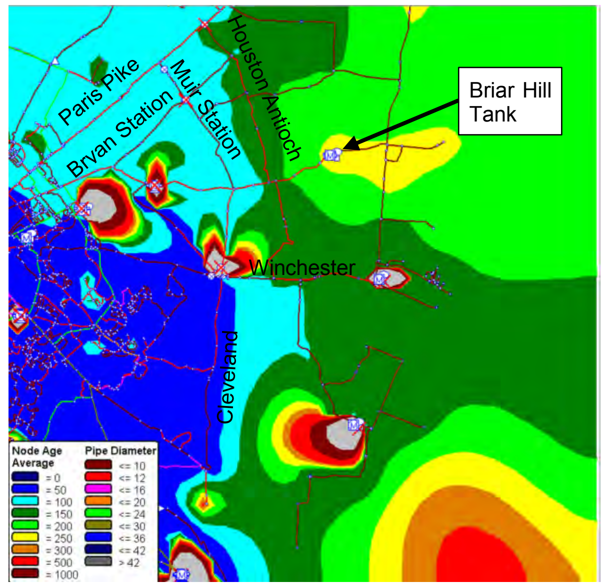


Figure 4.04-4 Briar Hill Tank Area Average Water Age with Capital Improvements

As the figures show, proposed capital improvements result in an overall improvement in water age in the Briar Hill area. The extension of the 8-inch main on Bryan Station Road to Muir Station Road along with the revised connectivity at that intersection eliminates two dead-end mains by creating two new loops at the intersection. There is a small increase in water age east of the Briar Hill tank and in the areas surrounding the Becknerville tank. The addition of the new and upgraded piping creates additional pipe volume in this area, potentially increasing the water age by a small amount in areas where no new loops were formed.

The revised connectivity and the closed valve on Houston Antioch Road further isolate the Briar Hill tank service area from the Eastland tank and the central portion of the system. Modeling indicates this will allow the Briar Hill tank to turn over within a 72-hour period during minimum demand scenarios.

The new control valve at the intersection of Cleveland and Winchester was initially modeled as a closed valve for the future minimum and maximum demand scenarios. However, modeling the valve as a check valve during a minimum demand day scenario causes the Briar Hill tank to remain full because of the HGL supplied by the central portion of the system and Eastland tank, which has an overflow 20 feet higher than Briar Hill. Therefore, the valve was modeled as a closed valve during minimum demand scenarios to isolate the Briar Hill tank from the central portion of the system, allowing the tank to properly turn over. The change in operation that is dependent upon demand could be addressed by installing a remotely controlled valve.

B. Muddy Ford Tank Area

Initial model runs of the future minimum demand day scenario with capital improvements did not include the proposed control valve on I-75 (could also be accomplished with check valves on Cynthiana Road and the US 460). With the proposed capital improvements described in the Georgetown Bypass and US 25 area, flow from KRS2 has another path north into the system other than through the 24-inch main on Newtown Pike. The connection to the 42-inch main at Ironworks Road and the improvements along Lisle Road allowed flow from KRS2 to enter the Muddy Ford area at a high HGL. This additional flow and low demand in the system caused the Muddy Ford tank to remain full. The control valve (or check valves) forces flow from KRS2 through Newtown Pike similar to the current operation of the system, allowing the Muddy Ford tank to turn over during minimum demand scenarios.

Figures 4.04-5 shows the average water age results for the current minimum demand day model without capital improvements for the Muddy Fork tank area. Figures 4.04-6 and 4.04-7 show the average water age results for the future minimum day model with and without the KY32 extension.

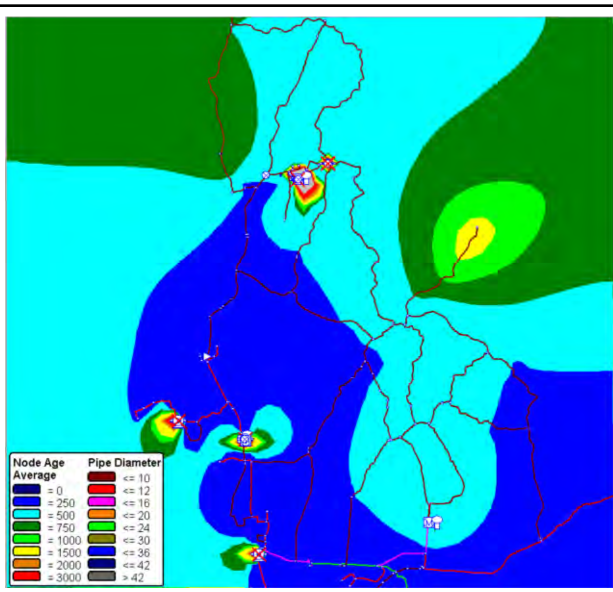


Figure 4.04-5 Muddy Ford Tank Area Average Water Age without Capital Improvements

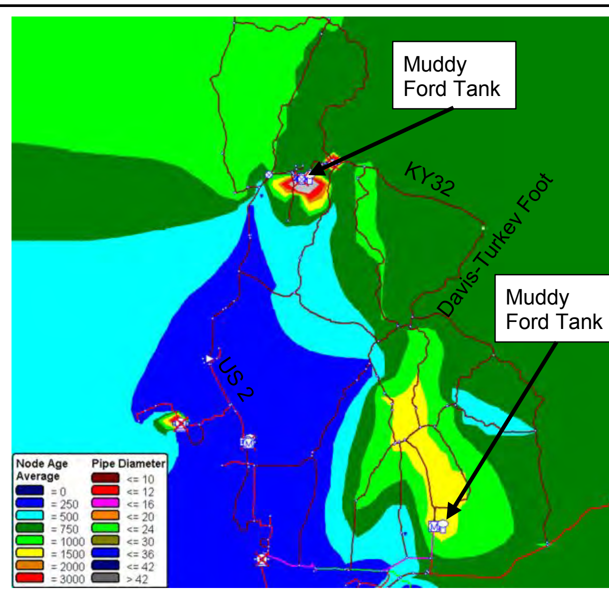


Figure 4.04-6 Muddy Ford Tank Area Average Water Age with Capital Improvements

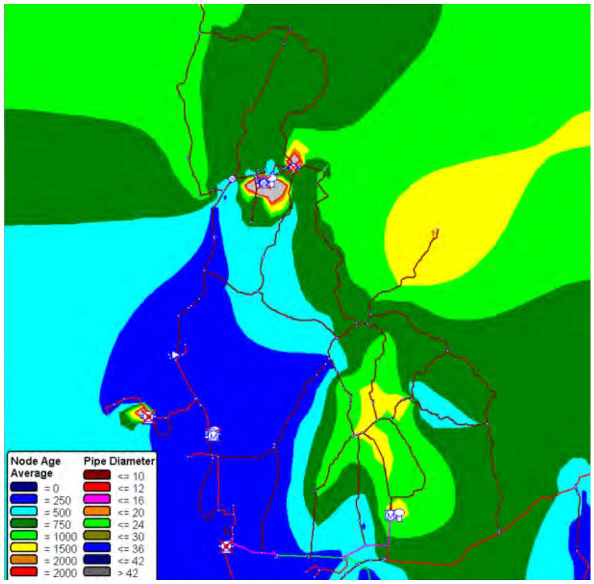


Figure 4.04-7 Muddy Ford Tank Area Average Water Age Without KY32 Improvements

As Figures 4.04-6 and 4.04-7 show, proposed capital improvements and operational changes cause increased water age results in the area north of the Muddy Ford tank. Initially, the KY32 extension was thought to be the cause of the increased water age results, but as Figure 4.04-7 shows, water age increases whether or not the extension is included in the simulation. While the KY32 extension reduces the water age along Davis-Turkey Foot Road, it also marginally increases water age north of the Muddy Ford tank and east of Sadieville. The model assumes no additional demand as added to the Central Division from the KY32 extension. Unless there are known water quality issues in this area or more customers can be identified along the extension to increase demand, this proposed capital improvement does not result in significant improvements to modeled water age.

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C. New Circle-Nicholasville Area

Figures 4.04-8 and 4.04-9 show the average water age results for the current minimum demand day model and the future minimum demand day model with capital improvements for the southern portion of the system, respectively.

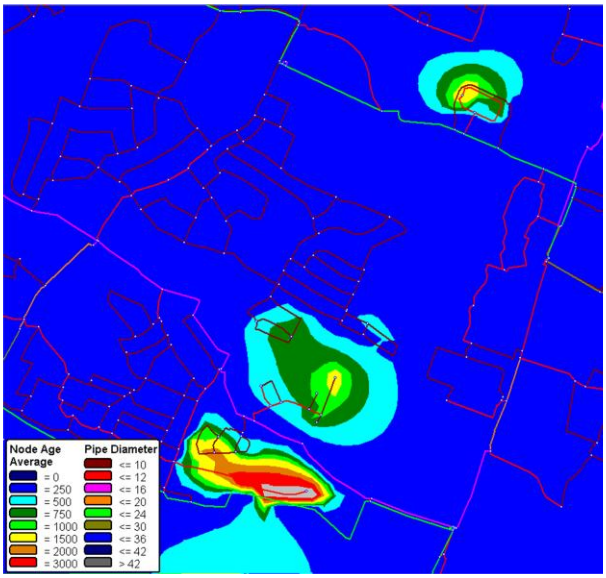

Figure 4.04-8 Southern Portion of System Average Water Age Without Capital Improvements

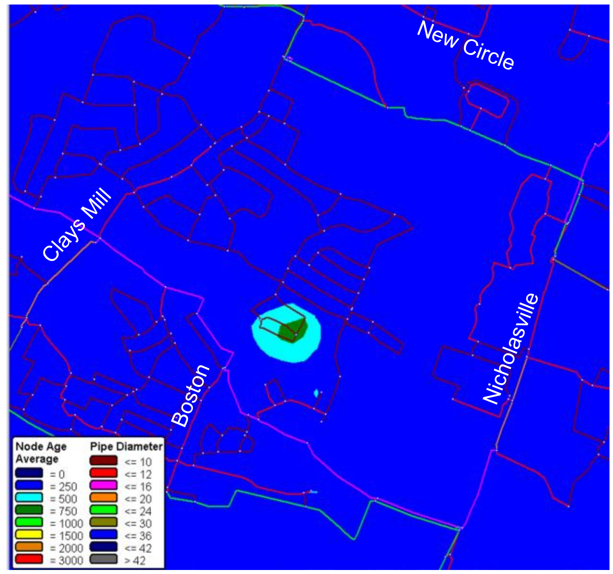

Figure 4.04-9 Southern Portion of System Average Water Age With Capital Improvements

Several loops were created in this area through the proposed capital improvements shown in Figure 4.02-9. In addition, the connection from Wyndham Hills Drive to the 24-inch KRS1 discharge main was included into the model. Water age in this area significantly decreased where the new loops were created. Pipe volume added was small for this area and did not appear to have an impact on water age of the surrounding nodes that were not directly impacted by the capital improvements.

D. Newtown Pike Upgrade South of New Circle vs. New Transmission Main on US 25

Figure 4.04-10 shows the average water age for the Mercer Road and Cox Street tank areas under the current minimum demand scenario. Figures 4.04-11 and 4.04-12 show the average water age for the Mercer Road and Cox Street tank areas with the Newtown Pike improvements and with the new 24-inch main along US 25, respectively. While neither improvement dramatically impacted pressures in the central portion of the system, the new main along US 25 consists of approximately 3 miles of new 24-inch pipe compared to approximately 2 miles of upgraded 16-inch to 24-inch pipe, adding considerably more pipe volume to the model. Adding more pipe volume into the model will increase the residence time of water in the pipes before it is used, resulting in an increased water age where the new 24-inch main along US 25 is connecting into the central portion of the system.

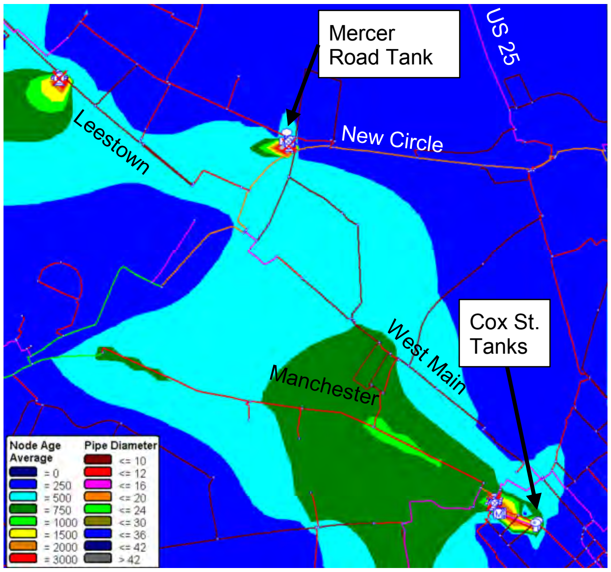

Figure 4.04-10 Mercer and Cox Street Tanks Area Average Water Age without Capital Improvements

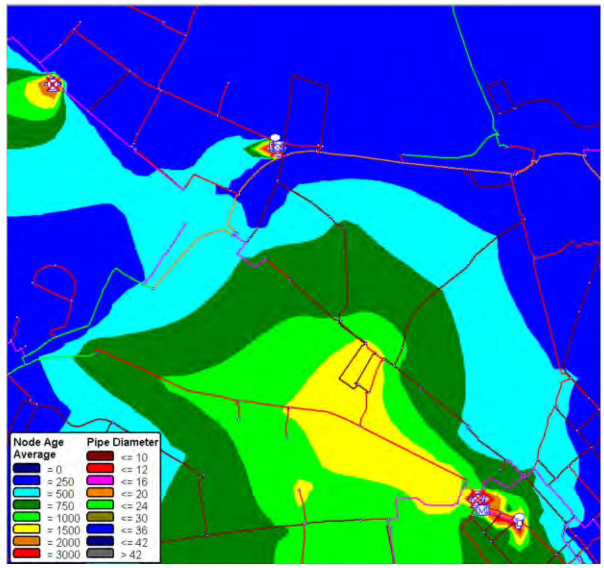

Figure 4.04-11 Mercer and Cox Street Tanks Area Future Average Water Age With US 25 Upgrades

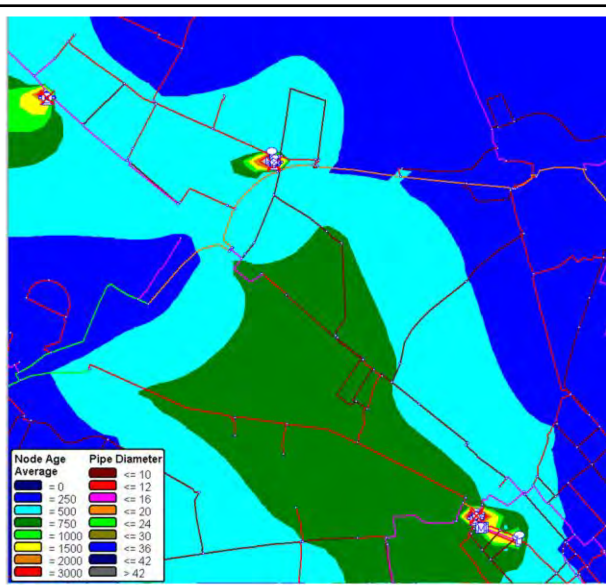

Figure 4.04-12 Mercer and Cox Street Tanks Area Average Water Age With Newtown Upgrades

E. Parkers Mill Tank Area

Figures 4.05-13 and 4.05-14 show the average water age results for the Parkers Mill tank area for a future demand day model without improvements and the future demand day model with the inclusion of the existing line along Pine Needles Lane and Guilford Lane, respectively.

Figure 4.05-13 Parkers Mill Area Average Water Age Without Capital Improvements

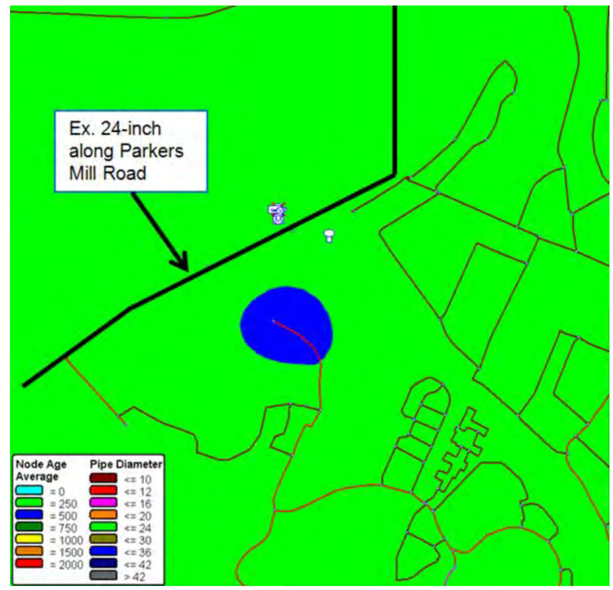

Figure 4.05-14 Parkers Mill Area Average Water Age With Addition of Existing Main

The inclusion of the existing line creates a loop and lowers the modeled water age of the dead-end lines along Man O War Boulevard and Roswell Drive. If this area is known to have poor water quality, the dead-end line on Roswell Drive could be extended to connect into the existing 24-inch main on Parkers Mill Road to create another loop, further reducing the water age at the end of Roswell Drive.

4.05 MAXIMUM DEMAND SCENARIO RESULTS

Figures 4.05-1 and 4.05-2 show the overall minimum pressure results for the future demand day model without improvements and the future demand day model with capital improvements, respectively.

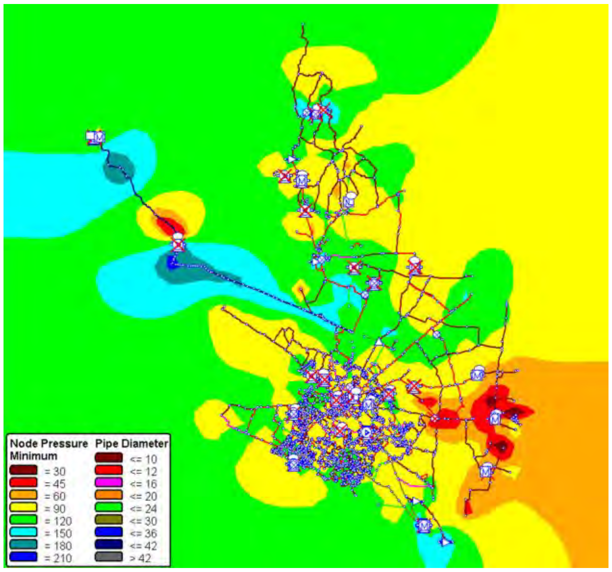

Figure 4.05-1 Central Division Minimum Pressures Without Capital Improvements

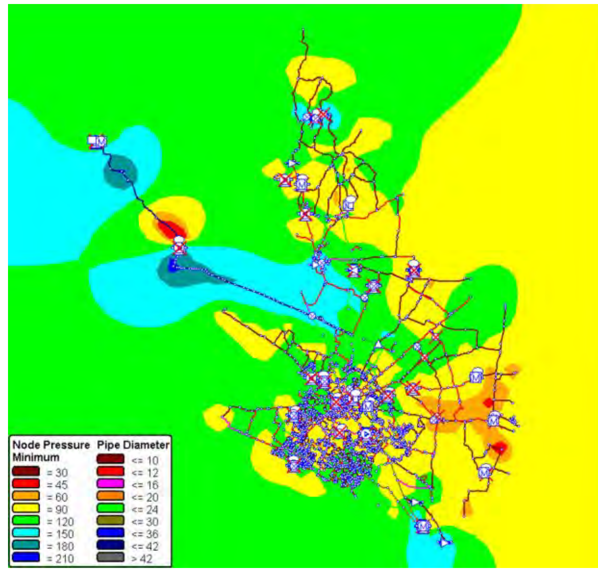

Figure 4.05-2 Central Division Minimum Pressures With Capital Improvements

Figures 4.05-3 and 4.05-4 show the overall maximum pressure results for the future demand day model without improvements and the future demand day model with capital improvements, respectively.

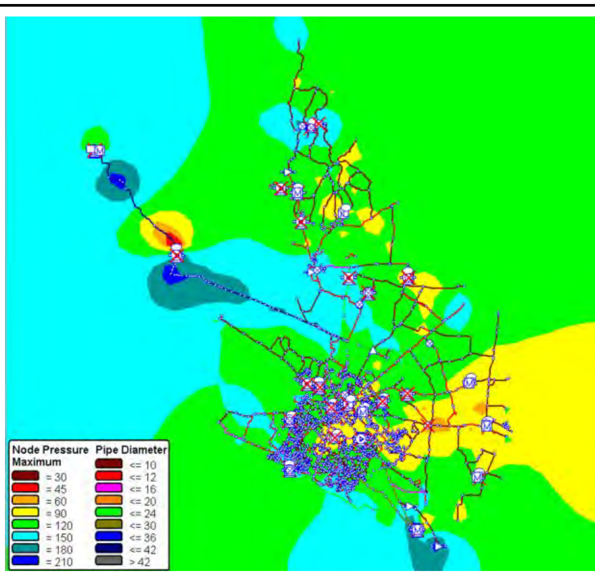

Figure 4.05-3 Central Division Maximum Pressures Without Capital Improvements

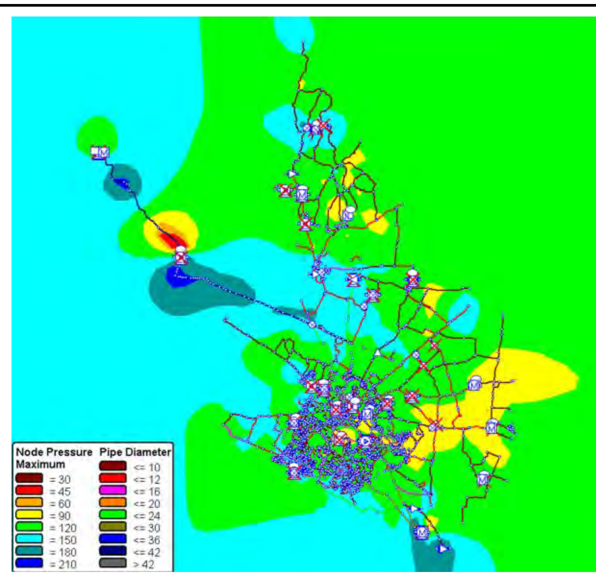

Figure 4.05-4 Central Division Minimum Pressures With Capital Improvements

As Figure 4.05-1 through 4.05-4 show, overall the distribution system provides adequate pressure. The following discussion focuses on the effects of capital improvements to address discreet areas within the distribution system.

A. Briar Hill Tank Area

Figures 4.05-5 and 4.05-6 show the minimum pressure results for the Briar Hill area for the future demand day model without improvements and the future demand day model with capital improvements, respectively.

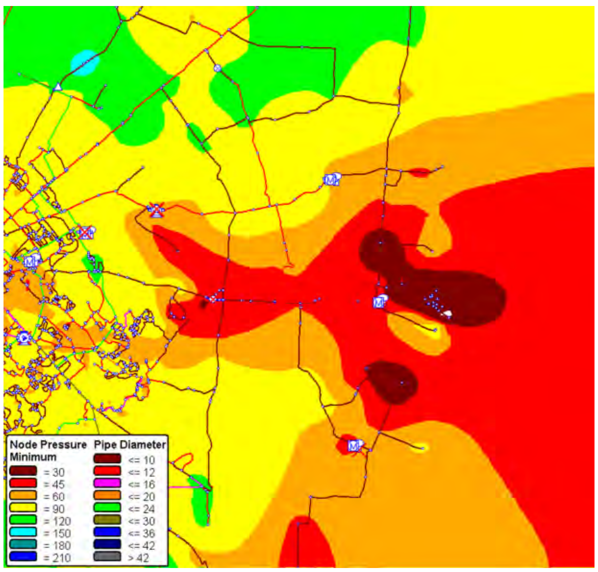

Figure 4.05-5 Briar Hill Area Minimum Pressures Without Capital Improvements

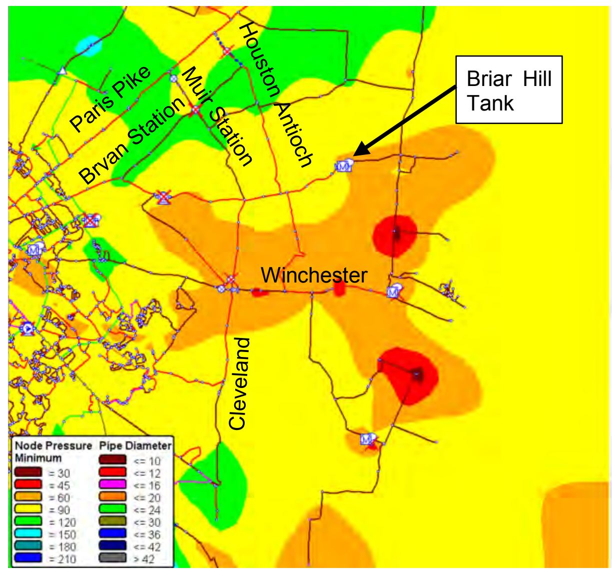

Figure 4.05-6 Briar Hill Area Minimum Pressures With Capital Improvements

Proposed capital improvements help maintain almost all pressures in the Briar Hill area above 45 psi except for a few local high elevation areas. Main improvements along Cleveland south of Winchester and along Todds Road in addition to the loop created from the connectivity changes at the intersection of Cleveland and Winchester improve pressures in the central system zone. Currently, flow is supplied to the Briar Hill PS primarily by two mains, the 12-inch main on Briar Hill Road and the 8- and 12-inch main on Royster Road. Flow coming from Royster Road must flow through 8-inch piping along Winchester Road. The high demands in the future maximum demand scenario cause high head losses in these mains. With the inclusion of the capital improvements, flow through Royster Road also flows from Cleveland south of Winchester through the upgraded 12-inch main, causing less head loss and increased pressures in the area.

For the future maximum demand day scenario, the proposed valve at the intersection of Cleveland and Winchester was modeled as a check valve. When modeled as a closed valve, model pressures along Winchester between Cleveland and Combs Ferry Road were typically all below 45 psi, similar to the results shown in Figure 4.05-6. When the Briar Hill PS is not in operation, these areas are supplied by the Briar Hill tank. During the high demand scenarios, the head loss from the tank to this area caused by increased flows was causing low pressures. Allowing the higher HGL central portion of the system to flow into the Briar Hill area during high demand scenarios when the Briar Hill PS is not operating provides adequate pressure without any additional main or infrastructure improvements.

B. Hume Bedford Pike and Greenwich Pike Area

Figure 4.05-7 shows the minimum pressure results for the future maximum demand scenario without capital improvements. Russell Cave pump station was not operated in this scenario. Valve improvements that are currently being constructed will allow the Russell Cave tank to operate more effectively without overpressurizing the system. However, this revised operation can also cause an area along Hume Bedford Pike to experience low pressures when the pumps are on as seen in Figure 4.05-8. When the Russell Cave PS is operating, flow to Hume Bedford is forced to flow through the 3-inch main along Greenwich Pike, causing high head losses through the small main. Figure 4.05-9 shows that upgrading this section of 3-inch main to 8-inch main allows modeled pressures in this area to more closely resemble pressures seen before the valve improvements.

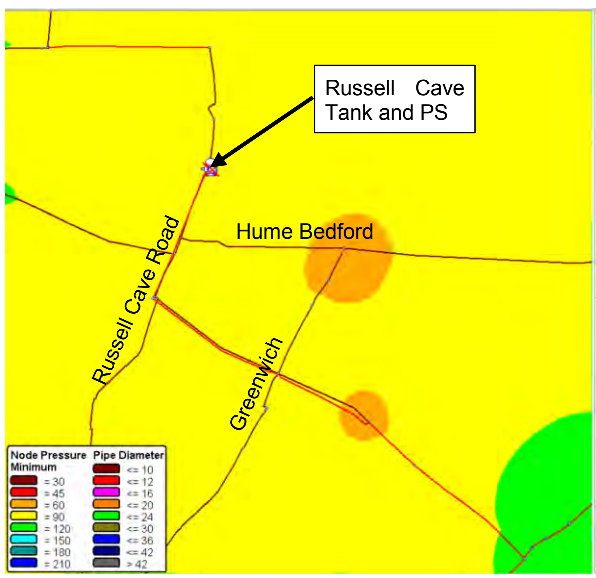

Figure 4.05-7 Hume Bedford and Greenwich Pike Area Minimum Pressures Without Capital Improvements

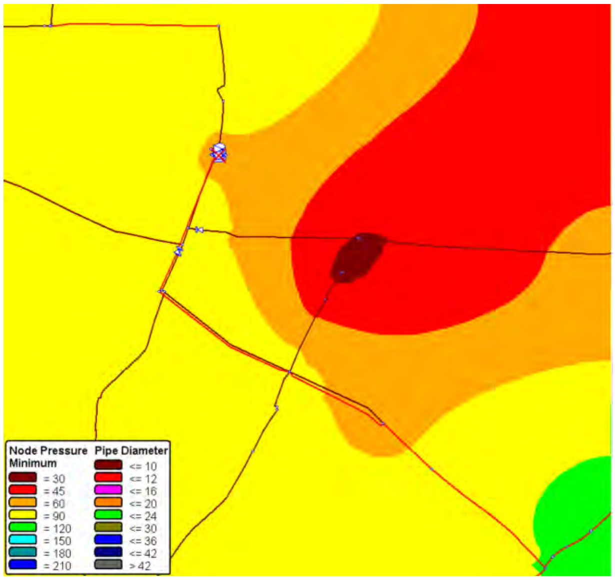

Figure 4.05-8 Hume Bedford and Greenwich Pike Area Minimum Pressures Without 8-Inch Main Upgrade

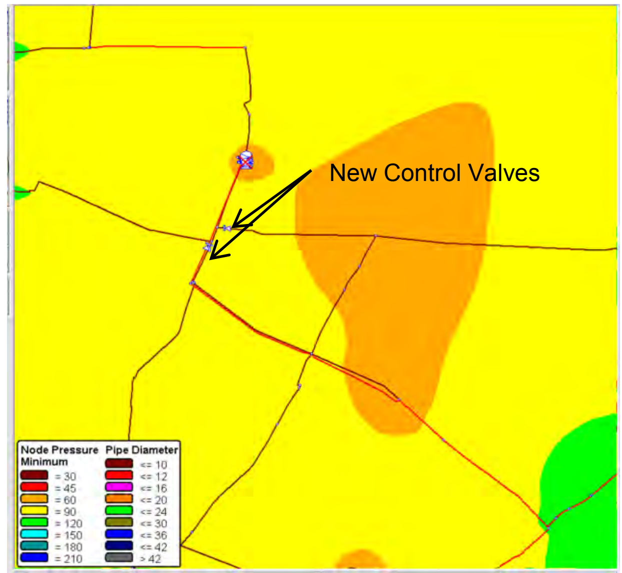

Figure 4.05-9 Hume Bedford and Greenwich Pike Area Minimum Pressures With 8-Inch Main Upgrade

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C. Georgetown Bypass and US 25 Area

Figures 4.05-10 and 4.05-11 show the minimum pressure results for the Georgetown Bypass and US 25 area for the future demand day model without improvements and the future demand day model with capital improvements, respectively.

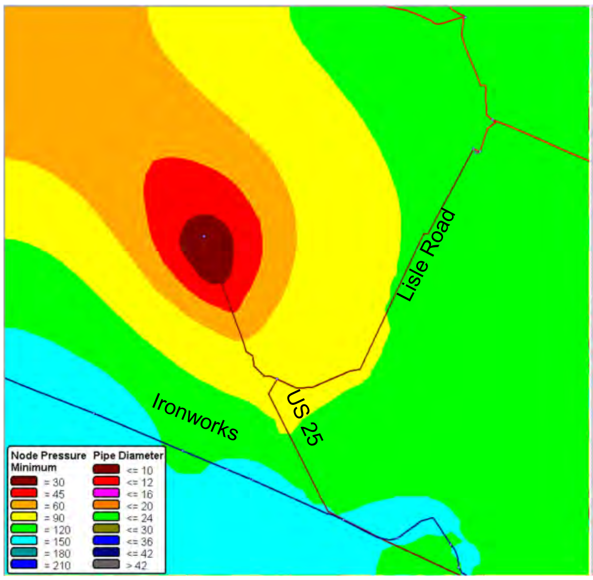


Figure 4.05-10 US 25 and Georgetown Bypass Area Minimum Pressures Without Capital Improvements

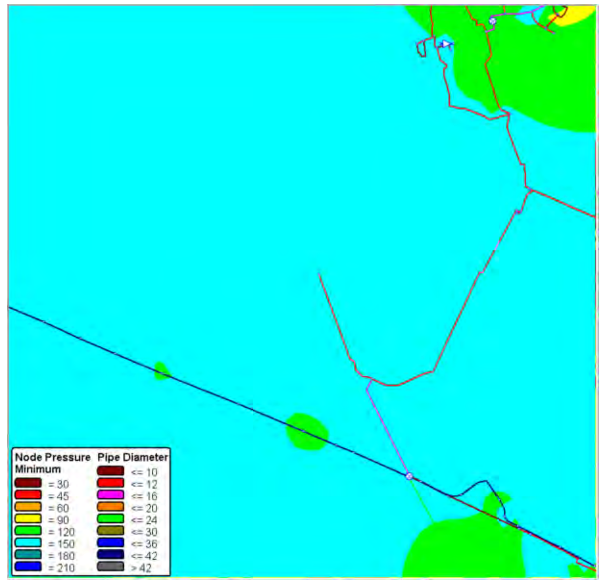


Figure 4.05-11 US 25 and Georgetown Bypass Area Minimum Pressures With Capital Improvements

Low pressures in the area shown in Figure 4.05-10 are a result of additional predicted demands of the Future Demand Area 7 described in Section 4.03. The connection into the 42-inch line on Ironworks at US 25 as well as the proposed main improvements along US 25 to the Georgetown Bypass provide adequate pressures for the anticipated demands in this area.

D. Muddy Ford Tank Area

Figures 4.05-12 and 4.05-13 show the minimum pressure results for the Muddy Ford tank area for the future demand day model without improvements and the future demand day model with capital improvements, respectively.

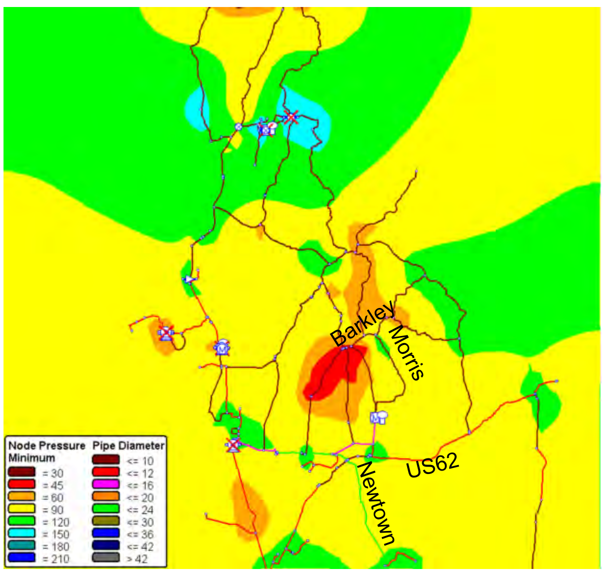


Figure 4.05-12 Muddy Ford Area Minimum Pressures Without Capital Improvements

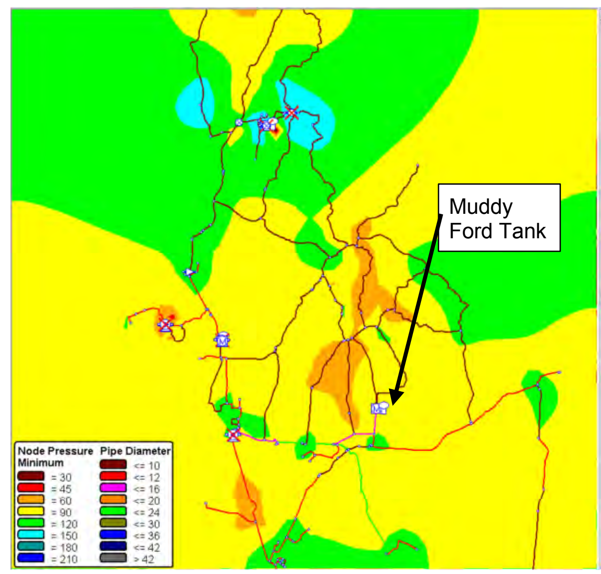


Figure 4.05-13 Muddy Ford Area Minimum Pressures With Capital Improvements

Although an existing 8-inch main runs north immediately outside the Muddy Ford tank along Morris Road, the diameter reduces to 2 inches on Barkley Road at its intersection with Morris Road. The increased demands through the small diameter piping for a future demand scenario causes additional headloss, reducing pressures below 45 psi. Proposed capital improvements in the Muddy Ford tank area utilize the existing 8-inch main on Morris Road to reach low pressure areas and only upgrade piping where necessary along Barkley Road. Another potentially more costly alternative includes upgrading the long stretches of small diameter pipe (4 inches or less) from the Muddy Ford tank along Gunnell Road. Either of the proposed capital improvements allow the area north of Muddy Ford tank to remain above 45 psi.

E. Newtown Pike Upgrade South of New Circle vs. New Transmission Main on US 25

Figures 4.05-14 and 4.05-15 display the minimum pressure results for the future demand scenario with capital improvements for the new US 25 transmission main and the upgraded Newtown Pike main south of Ironworks Road, respectively. Figure 4.05-16 shows the minimum pressure results for the future demand scenario without any capital improvements for comparison.

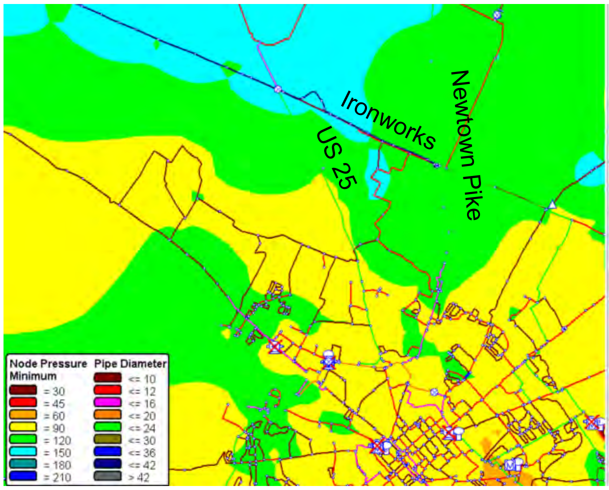


Figure 4.05-14 Future Maximum Demand With New US 25 Transmission Main

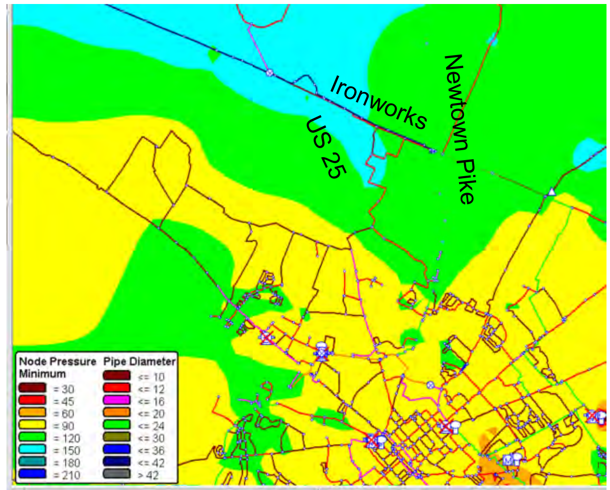


Figure 4.05-15 Future Maximum Demand With Upgraded Newtown Pike Line

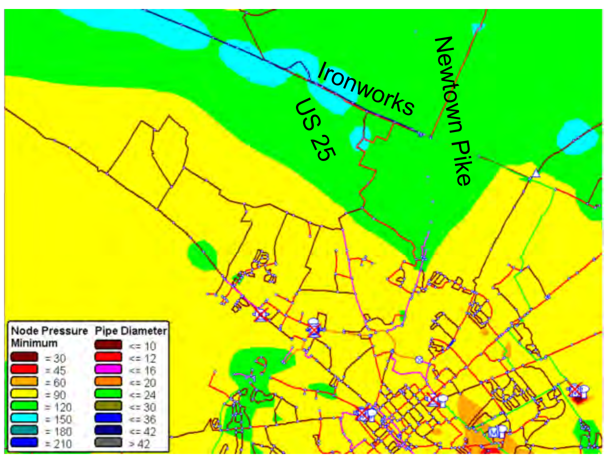


Figure 4.05-16 US 25 and Newtown Area Future Maximum Demand Without Capital Improvements

As Figure 4.05-16 shows, pressures in the US 25 and south Newtown Pike areas south of Ironworks Road without capital improvements are adequate, with minimum pressures typically between 60 psi and 90 psi indicated by the large yellow area in the figure. This indicates adequate pressures can be maintained without the US 25 or Newton Pike improvements. Comparing Figures 4.05-14 and 4.05-15, both proposed capital improvements yield similar pressures. In such cases, other factors must be considered.

As previously stated, the new main along US 25 consists of approximately 3 miles of new 24-inch pipe compared to approximately 2 miles of upgraded 16-inch to 24-inch pipe for Newton Pike. Though the US 25 improvements require more 24-inch main to be installed than the Newtown Pike improvements, KAW could reduce costs for the US 25 improvements by coordinating construction with KYTC and its current 6-year plan for US 25. Utility relocations along this corridor of US 25 are already planned because of road realignments. KAW could install the new 24-inch main along US 25 at the same time, which would reduce capital costs for the US 25 improvements. The US 25 improvements would also add redundancy to the system by creating another path to the central portion of the system from KRS2, which would be beneficial if a main break occurred on the Newtown Pike main south of Ironworks Road.

F. Eastland Tank Area

As discussed in Section 3, the areas immediately southeast of the Eastland tank in the central portion of the system experience low pressures. These areas typically experience low pressures because they are at a higher elevation relative to the rest of the central portion of the system. Lower pressures did not appear to be a function of piping pressure loss. Two options were discussed to raise pressures in these areas.

1. Create a New Boosted Pressure Zone–This option involves isolating the area southeast of Eastland tank and using a new booster pump to provide a additional pressure to the area. The area southeast of Eastland tank is highly interconnected into the central portion of the system. Creating a new pressure zone would require closing numerous mains, creating dead ends and potentially causing water quality issues unless additional mains were also installed within the new pressure zone to create loops. Because of the operational complexity and high cost associated with this option to only increase pressures for a small percentage of the customer base, it was not considered a feasible alternative at this time and therefore was not modeled.

2. Operate the Eastland Tank Within the top 10 Feet of its Operating Range–Eastland tank has an overflow elevation of 1,170 feet. SCADA data indicates the Eastland tanks drops below 1,140 feet and operates within a 25-foot range on an average day, operating roughly between 1,135 and 1,160 feet. Operating the Eastland tank within the 10 feet of its operating range would maintain higher average pressures throughout the central portion of the system. This option only involves operational changes and does not involve any new infrastructure. Figures 4.05-17 and 4.05-18 show the minimum pressure results for the area southeast of the Eastland tank for the future demand day model without improvements and the future demand day model with operational improvements, respectively.

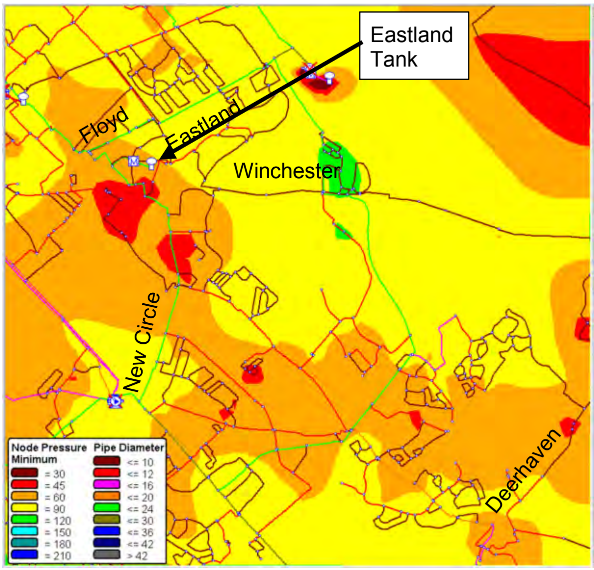


Figure 4.05-17 Eastland Tank Area Minimum Pressures As Currently Operated

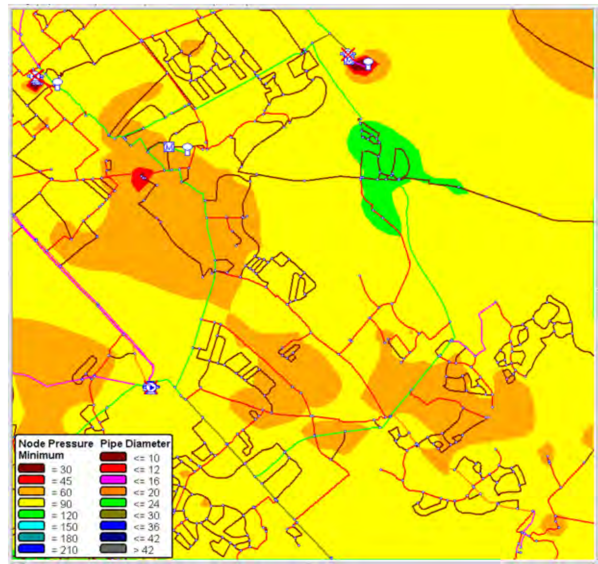


Figure 4.05-18 Eastland Tank Area Minimum Pressures With Operational Changes

The other proposed capital improvement in this area includes increasing the size of the 6- and 8-inch mains on New Circle Road between Eastland Drive and Floyd Drive to 12 inches. This main was identified by KAW as a bottleneck and as a main that has had breaks in the past. Because of its depth, it is expensive to repair. The improvements did not noticeably increase pressure but did reduce maximum velocities in the mains from 4.6 ft/s. to 1.7 ft/s.

G. Leestown Road Near Midway Interconnect

Although minimum pressures along Leestown Road toward the Midway interconnect remain above 45 psi, velocities in some of the existing piping are high. Velocities in the existing parallel 8-inch mains between Opportunity Way and Greendale Pike exceed 4.5 ft/s. These two parallel 8-inch mains combine into a single 8-inch main when crossing Greendale Pike before it connects into the 12-inch main on Greendale Pike. The modeled velocities in this small stretch of pipe exceed 9 ft/s during future maximum demand scenarios. Replacing the existing 6-inch and 8-inch mains with a single 16-inch main along Leestown Road between Opportunity Way and Bradley Lane reduces the maximum modeled velocity in the new main to 3.1 ft/s.

H. New Circle-Nicholasville Area

Figures 4.05-19 and 4.05-20 show the minimum pressure results for the area southeast of the Eastland tank for the future demand day model without improvements and the future demand day model with capital improvements, respectively.

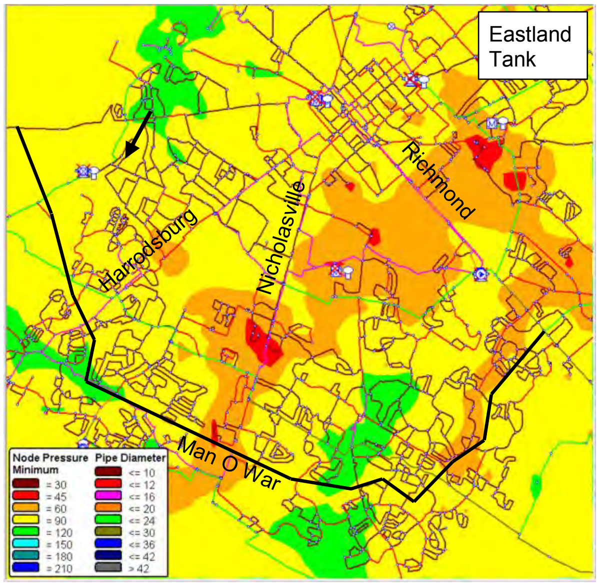


Figure 4.05-19 Southern Area Minimum Pressures–Eastland Tank as Currently Operated

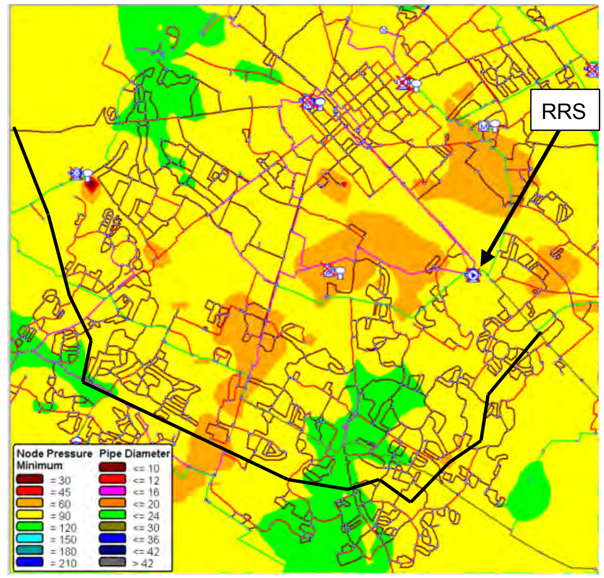


Figure 4.05-20 Southern Area Minimum Pressures–Eastland Tank Recommended Operation

Similar to the low pressures experienced southeast of the Eastland tank, low pressures identified in the southern portion of the system off Nicholasville Road near New Circle and Man O War Boulevard are a result of high relative elevations as opposed to distribution system bottlenecks. A separate boosted pressure zone could also be created to increase pressures in this area but was disregarded for similar reasons as the proposed boosted pressure zone near the Eastland tank. Because the central portion of the system is on the same pressure zone, operating the Eastland tank in the top 10 feet of its operating range will have a similar impact on this area and will help maintain higher average pressures.

4.06 CENTRAL DIVISION CAPITAL IMPROVEMENT CONCLUSIONS

Overall modeling results indicate the system can typically meet the projected future maximum demand and maintain a minimum pressure of 30 psi. Proposed capital improvements add redundancy to the system and help maintain a minimum pressure of 45 psi in most areas of the system while not substantially increasing maximum pressures.

In some cases, proposed capital improvements increased water age in some of the surrounding areas because of additional pipe volume in the system and the assumption that no new customers were placed on new mains. Capital improvements directed specifically at creating loops in the system typically reduced water age.

SECTION 5
NORTHERN DIVISION IMPROVEMENTS

5.01 CURRENT SYSTEM OPERATION

The following describes the current system operation of facilities in the Northern Division.

1. The Fairgrounds tank is the largest tank in the Northern Division and is located within Owenton. The Owenton WTP operates to maintain water levels in the Fairgrounds tank.

2. The Perry Street tank operates at the same HGL as Fairgrounds tank but the overflow is 5 feet lower. To help the Perry Street tank turn over, a control valve opens/closes based on the Fairgrounds tank level.

3. The Bromley tank is north of Owenton and serves the Bromley and Sparta areas. The Bromley tank has a separate fill and drain line. The tank is filled by the Fairgrounds tank, but customers downstream of Bromley on the drain line side only see the HGL of the Bromley tank. The fill line has an altitude valve that opens and closes based on the water level in the Bromley tank to turn the tank over.

4. The Monterey tank serves the town of Monterey, south of Owenton off US 127. The Monterey tank is set up similar to the Bromley tank with a separate fill and drain line so that the town of Monterey only sees the HGL of the Monterey tank. The valve on the fill line is manually opened approximately once every three days to fill the tank.

5. The Wheatley tank and Wheatley service area is currently not connected to the Owenton WTP system although the infrastructure is owned and maintained by KAW. The Wheatley tank is served by the Carroll County Water District (CCWD No. 1).

6. The Glencoe tank is not connected to the Owenton system although the infrastructure is owned and maintained by KAW. It is served entirely by the Gallatin County Water District (GCWD).

7. The New Columbus tank is refilled by the New Columbus PS. The pump station is turned on approximately once every day for six hours to fill the tank.

KAW staff have difficulty turning the water over in several tanks in the system because these tanks' overflow elevations are significantly lower than the Fairgrounds, Perry Street, and New Columbus tanks. The following tanks in the Northern Division are currently on limited use because of turnover issues previously mentioned:

1. Sparta
2. Glencoe
3. Elk Lake
4. Long Ridge
5. Hesler

Appendix E includes information on the status and operation of facilities in the Northern Division.

5.02 IMPROVEMENTS RECENTLY DESIGNED

Strand Associates has recently completed design of a major capital improvement project to change the source of water being provided to the Northern Division. When construction is completed, the Northern Division's source of supply will change from its existing Owenton WTP to the KRS2 WTP on the Franklin/Owen County border.

The following is a summary of these improvements:

- 14 miles of 16-inch transmission main from the KRS2 WTP to a point just outside of the City of Owenton.
- 300,000-gallon elevated storage tank near the City of Monterey.
- 2 MGD pump station.
- 600,000-gallon elevated storage tank just outside of the City of Owenton.
- 4 miles of 6-inch main along KY 607 between Sawbridge Creek Road and KY 227.

These improvements are displayed in Figure 5.02-1. The design basis for these improvements and additional details that are incorporated into the model may be found in the 2012 report titled *Northern Division Connection Design Report*. Incorporation of these improvements was used as the baseline for modeling scenarios conducted to determine the need for additional improvements.

5.03 NORTHERN DIVISION DEMAND SCENARIOS

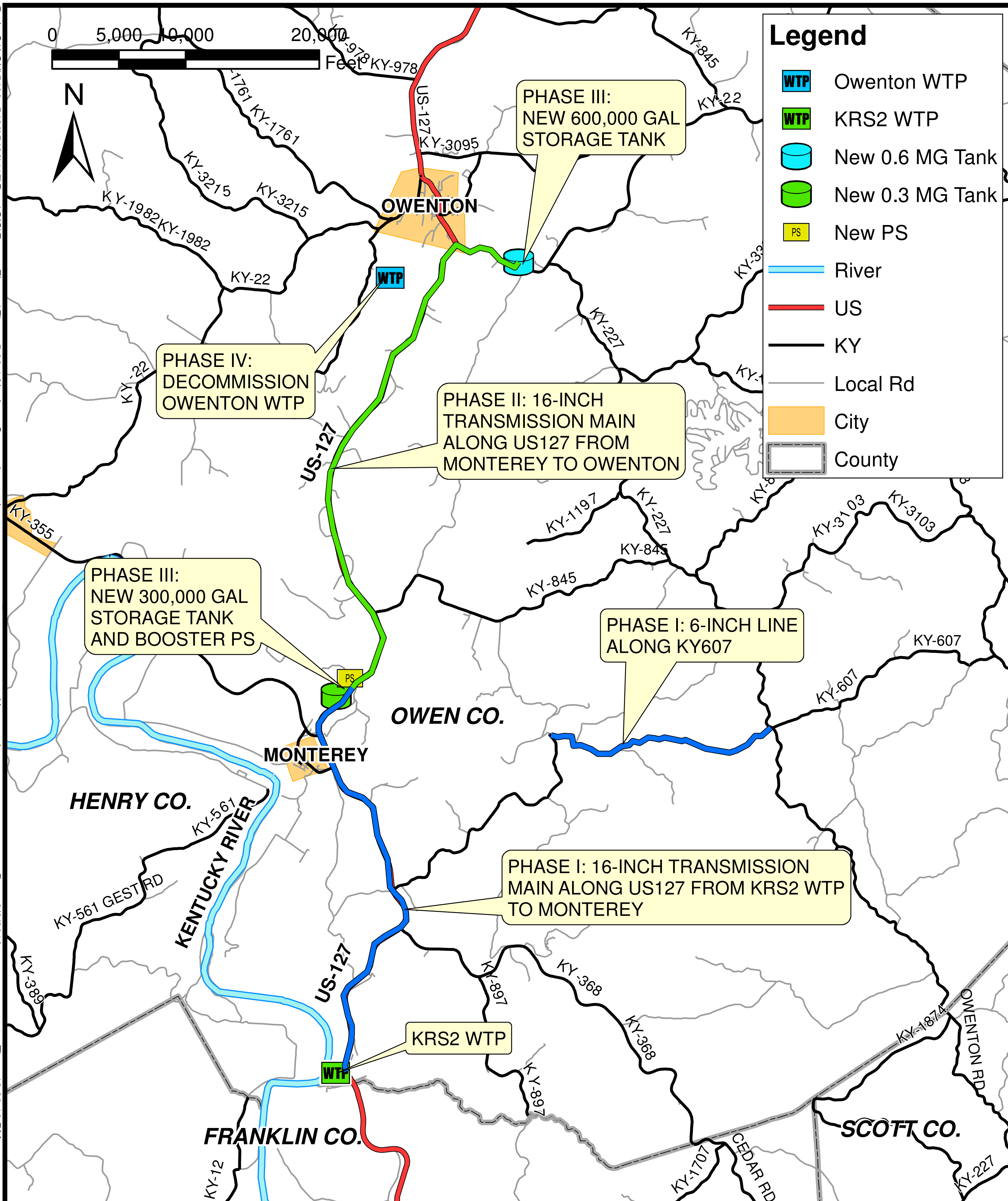
KAW identified current and potential future system demand conditions of interest for the purposes of modeling. Future minimum day demand scenarios were used to compare water quality conditions (water age) in the Northern Division before and after implementation of proposed capital improvements. Future maximum day demand scenarios were used to compare the hydraulic conditions of the Northern Division before and after capital improvements. Other modeling baseline scenarios were conducted, and the results of these simulations are provided in Appendix F. Table 5.03-1 summarizes the demand scenarios for the Northern Division.

Description	Minimum Day (MGD)		Average Day (MGD)		Maximum Day (MGD)	
	Current	Future	Current	Future	Current	Future
System Demand	0.6	0.8	1.0	1.3	1.5	2
KRS2 Production	5.5	5.5	9	10	12	15

Table 5.03-1 Target Northern Division Demand and KRS2 WTP Production Scenarios

The demand increases between current and future were allocated in the model by increasing peaking factors across the Northern Division.

Path: S:\LOU\5400--5499\5493\117\Data\GIS\Report Figures\Figure 5.02-1 Northern Connection.mxd
User: andrewe
Date: 2/9/2012
Time: 12:39:12 PM



NORTHERN DIVISION CONNECTION

HYDRAULIC MODELING FOR THE COMPREHENSIVE PLANNING STUDY
KENTUCKY AMERICAN WATER
LEXINGTON, KENTUCKY

STRAND ASSOCIATES

FIGURE 5.02-1
5493.117

5.04 IDENTIFIED AREAS OF INTEREST

The primary interest KAW identified was to evaluate the improvements needed to supply areas currently served by CCWD No. 1 and GCWD with water from KRS2 Pool 3. Areas of interest for water quality were identified by conducting water age modeling the existing Northern Division under a current minimum day demand scenario. Areas of interest for pressure and hydraulics were identified by modeling the Northern Division distribution system under a select future maximum day demand scenario. Capital improvements selected to improve water quality and hydraulic conditions in the identified areas of interest are discussed in detail in Section 5.05. Water quality modeling results for the baseline condition and with proposed capital improvements are discussed in further detail in Section 5.06. Hydraulic modeling results for the existing system and with proposed capital improvements are discussed in detail in Section 5.07. The following is a list of identified areas of interest. Figure 5.04-1 shows the location of the identified areas of interest within Northern Division model.

1. Areas North of Fairgrounds Tank–Hydraulic modeling of future demands indicated existing infrastructure was not sufficient to support predicted demand and supply water to areas currently served by CCWD No. 1 and GCWD while maintaining adequate system pressure (above 45 psi).

2. Areas Currently Served by CCWD No. 1 (Wheatley Tank Area)–KAW wants to stop buying water from CCWD No. 2 and serve this area with water pumped from KRS2. This area is currently in a different pressure zone and modification will be needed to incorporate it into the Fairgrounds pressure zone.

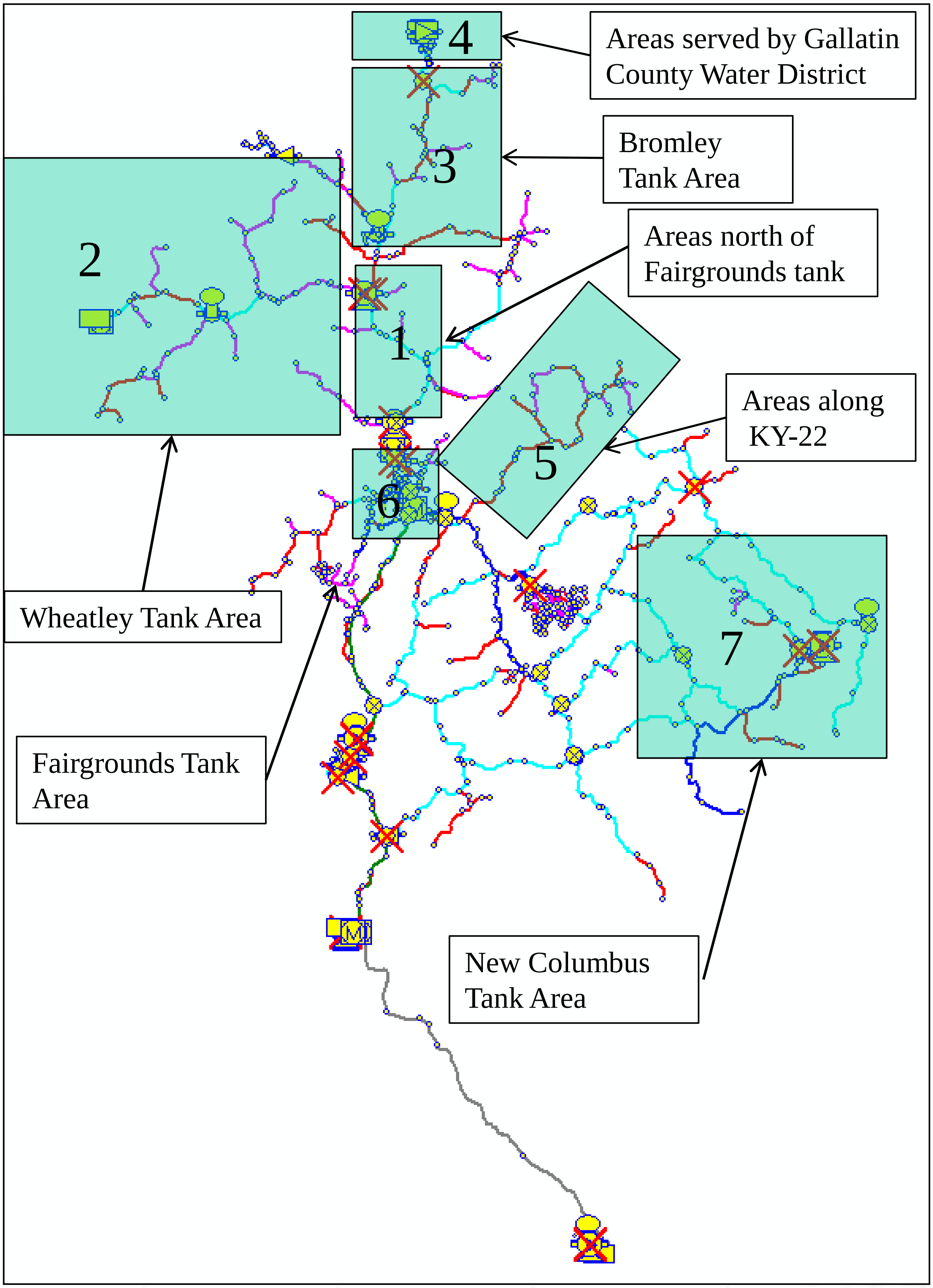
3. Bromley Tank Area–Modeling identified this area as having low pressures.

4. Areas Currently Served by GCWD–KAW wants to stop buying water from GCWD. This area is currently in a different pressure zone and modification will be needed to incorporate it to the Northern Division pressure zone.

5. Areas Along KY-22–Modeling identified areas around and north of the New Owenton tank with low pressures.

6. Fairgrounds Tank Area–Baseline modeling indicated that piping improvements are required to turn the tank over during low demand and fill the Fairgrounds tank with the projected increased demands on it.

7. New Columbus Tank Area–Modeling indicated elevated water age compared to other portions of the system for the New Columbus Tank and areas supplied by this tank. This is because this standpipe currently serves a very small demand. KAW operations staff also indicated they have difficulty turning over the water in this tank. In addition, KAW wants to eliminate the need to service the New Columbus Tank area from the Georgetown Municipal Water and Sewer Service.



NORTHERN DIVISION IDENTIFIED AREAS OF INTEREST
HYDRAULIC ANALYSIS FOR COMPREHENSIVE PLANNING STUDY
KENTUCKY AMERICAN WATER
LEXINGTON, KENTUCKY

STRAND ASSOCIATES

FIGURE 5.04-1

5493.117

5.05 NORTHERN DIVISION PROPOSED CAPITAL IMPROVEMENTS

The following capital improvements were modeled in the Northern Division under estimated future minimum and maximum day demand conditions to review potential impacts on system water quality and hydraulics. Capital improvements have been divided based on their general geographic area within the Northern Division. Main improvements were typically modeled as line replacements and upgrades and not as a parallel main. If preferred, KAW could install the proposed capital improvements as parallel lines in these locations if desired.

A. Areas North of Fairgrounds Tank

1. Upgrade existing 6-inch on US-127 from Fairgrounds Road to intersection of US-127 and KY-227 (New Liberty Sparta Pike) to 12-inch. Figure 5.05-1 show the proposed improvements.

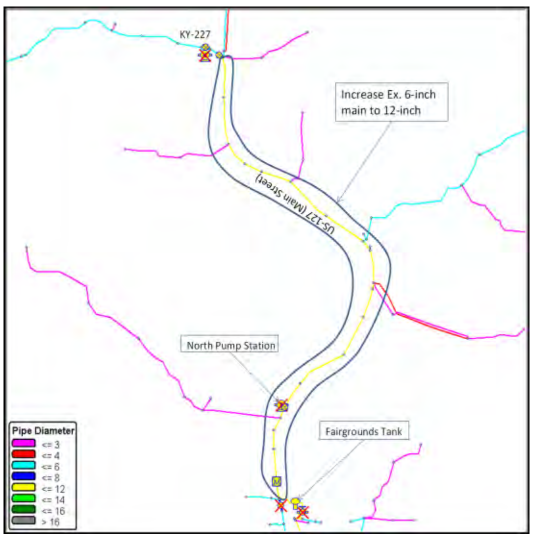


Figure 5.05-1 Areas North of Fairgrounds Tank Capital Improvements

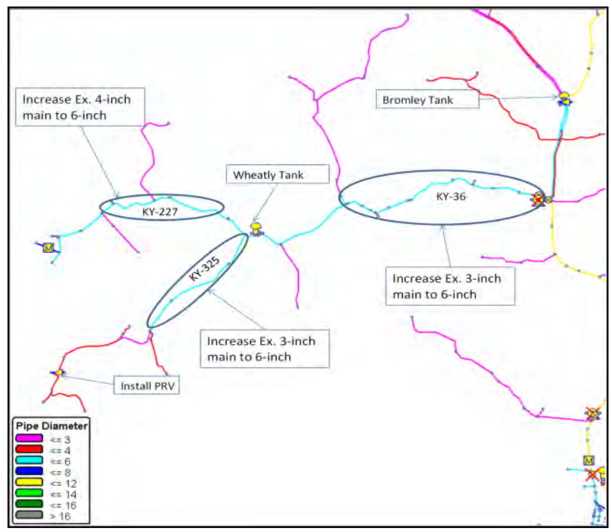


Figure 5.05-2 Wheatley Tank Area Capital Improvements

B. Areas Currently Served by CCWD No. 1 (Wheatley Tank Area)

1. Open closed valve at intersection of US-127 and KY-36.
2. Upgrade existing 3-inch on KY-36 to 6-inch.
3. Upsize existing 3-inch main on KY-325 to 6-inch.
4. Change location of altitude valve to the new influent line to allow filling from the Fairgrounds tank in lieu of CCWD No. 1.
5. Install PRV on main along KY-325 just before the significant drop in elevation.
6. Upgrade existing 4-inch main on KY 227 to 6-inch.

Figure 5.05-2 shows the proposed improvements.

C. Bromley Tank Area

1. Install flow control valve at influent to tank.

2. Upgrade existing 6-inch main along US-127 to 12-inch from the Bromley tank to the intersection Eagle Hill Road.

3. Upsize existing 4-inch main on Eagle Hill Road to 6-inch.

Figure 5.05-3 shows the proposed improvements.

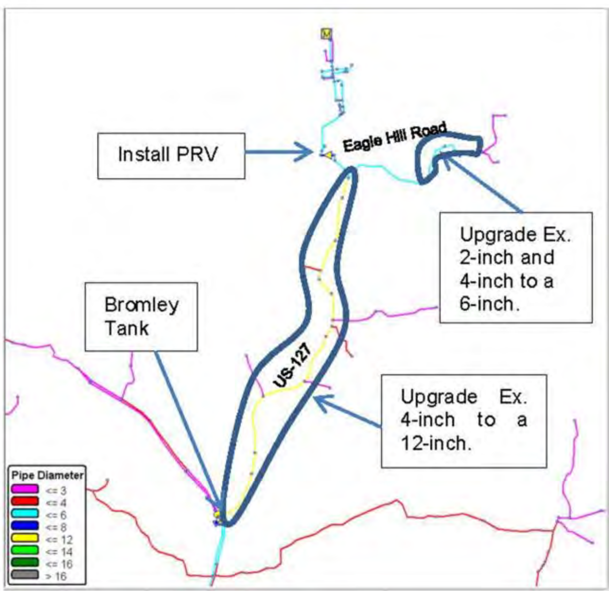


Figure 5.05-3 Bromley Tank Area Capital Improvements

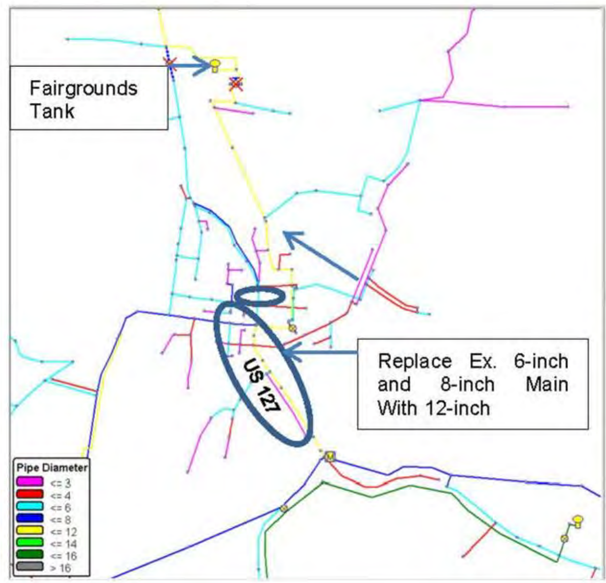


Figure 5.05-4 Fairgrounds Tank Area Capital Improvements

D. Areas currently served by GCWD

Install PRV. Figure 5.05-3 shows the proposed improvements.

E. Fairgrounds Tank Area

1. Replace the 6-inch main on US 127 from intersection of US 127 and KY-22 with 12-inch mains.

2. Replace the 8-inch main on Seminary Street with 12-inch mains.

Figure 5.05-4 shows the proposed improvements.

F. Areas along KY-22 North of New Owenton Tank

Upgrade the existing 4-inch main along KY-22 from KY-227 to KY-845 with a 6-inch main. Figure 5.05-5 shows the proposed improvements.

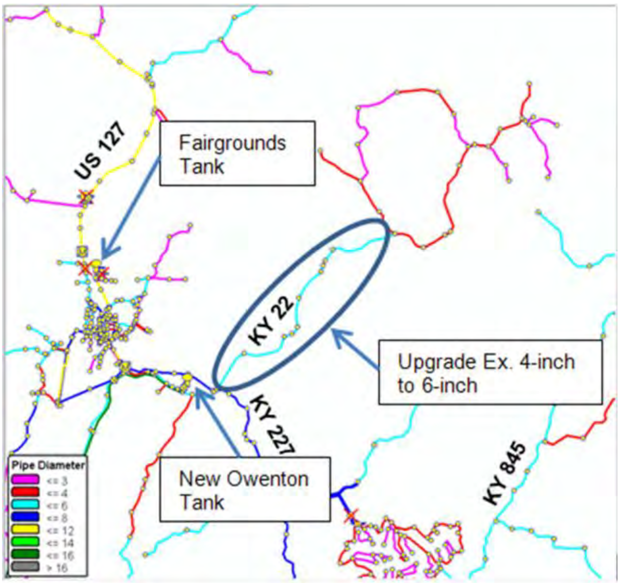


Figure 5.05-5 Areas Along KY-22 North of New Owenton Tank Capital Improvements

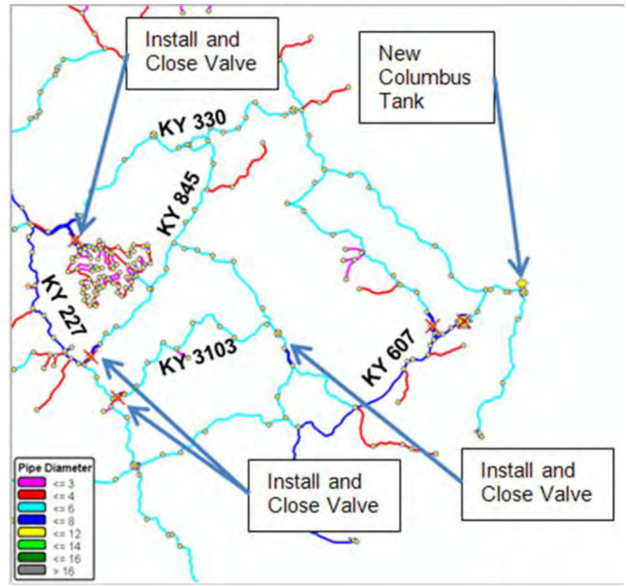


Figure 5.05-6 New Columbus Tank Area Capital Improvements

G. New Columbus Tank Area

1. Install a closed valve at the intersection of KY-3103 and KY-227.
2. Install a closed valve at the intersection of KY-845 and KY-227.
3. Install a closed valve at the intersection of KY-330 and KY-227.
4. Install a closed valve on KY-1883.

Figure 5.05-6 shows locations of the proposed improvements.

5.06 MINIMUM DAY DEMAND SCENARIO RESULTS

The following minimum demand scenario water age modeling discussions do not comment on all areas with high water age shown in the figures. Areas shown with high water age in the figures that are not discussed are tanks, mains, pumps, or other infrastructure that were not in operation during the water age simulations and, therefore, show unrealistic elevated water age results. Only infrastructure that shows elevated water age, and is in operation, was targeted for capital improvements and discussed in this section. All modeled water age scenario results and associated figures are presented and discussed in terms of hours.

It is important to note the following water age results do not directly indicate where water quality conditions are poor, as there is no set water age where water quality is considered poor. Increased water age has been known to correlate with water quality issues such as an increase in contaminants of certain disinfection byproducts (DBPs) and decrease in disinfectant concentration. Such water quality concerns depend on a number of outside factors that are not included in the following model simulations. The intended use of the water age results is to show relative water ages throughout the system to identify areas that would benefit more from capital improvements aimed at reducing water age.

Figures 5.06-1 and 5.06-2 show the overall average water age results for the current minimum demand day model and the future minimum demand day model with capital improvements, respectively.

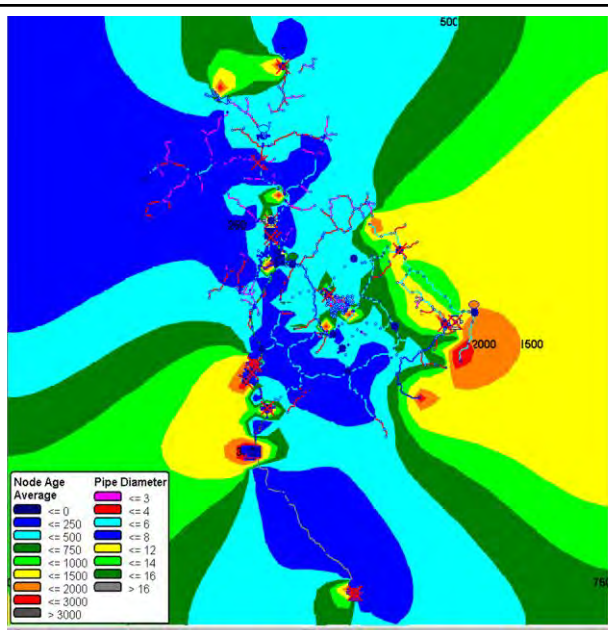


Figure 5.06-1 Current Minimum Demand Day Average Water Age

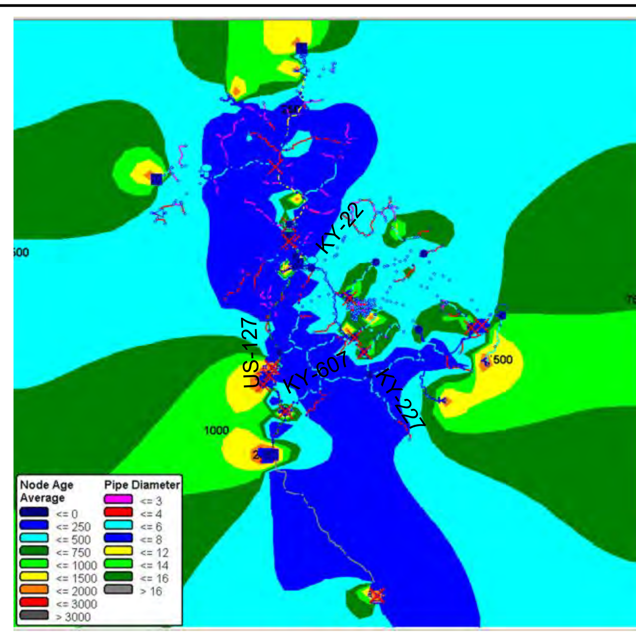


Figure 5.06-2 Future Minimum Demand Day Average Water Age with Capital Improvements

A. Southern Portion of Distribution System

Figures 5.06-3 and 5.06-4 show the average water age results before and after implementing capital improvements for the southern portion of the distribution system, respectively.

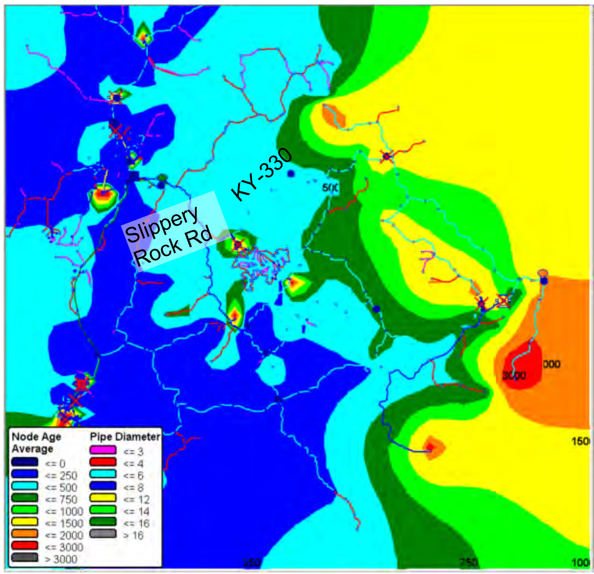


Figure 5.06-3 Southern Portion Average Water Age without Capital Improvements

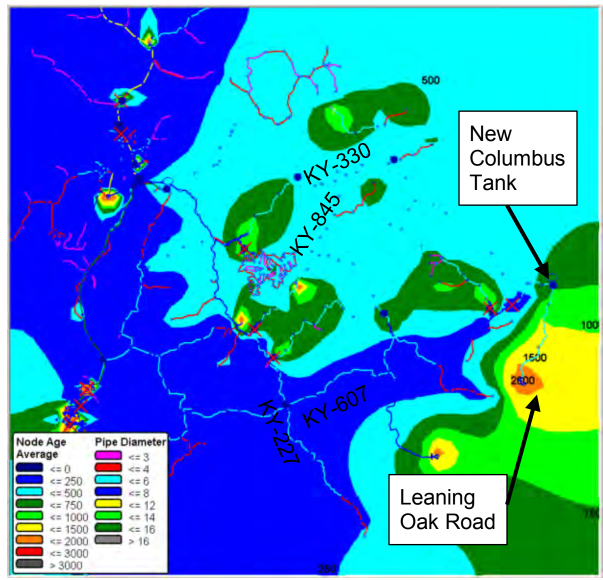


Figure 5.06-4 Southern Portion Average Water Age with Capital Improvements

As the figures show, proposed capital improvements result in an overall reduction in water age in the New Columbus Tank Area. Closing of valves and allowing New Columbus to serve more area reduced the time for the tank to turn over. While the water age at the end of Leaning Oak Road improved, it appears this area may require additional flushing of this dead end main. Connecting this main to nearby mains does not appear feasible given the proximity to the New Columbus pump station.

Other alternatives considered included installing control valves just north of the intersection of KY-227 and Slippery Rock Road to prevent the New Owenton tank from feeding the New Columbus tank pressure zone and on KY-845 to prevent the New Monterey tank booster pump station from feeding the New Columbus tank pressure zone when the New Columbus tank is relatively full. This alternative improved water age; however, areas along US-227 experienced minimum pressures below 30 psi during maximum day extended period simulations. Since pressures greater than 30 psi are required by KDOW, this alternative was not pursued further.

B. Northern Portion of Northern Distribution System

Figures 5.06-5 and 5.06-6 show the average water age results for the current minimum demand day model and the future minimum demand day model with capital improvements for the areas north of the Fairground tank, respectively. Most of the improvements in this area were identified to improve minimum pressures and not water age. However, increasing infrastructure size might lead to elevated water age. Water age did increase in areas west of the Wheatley tank. In the existing condition simulation, Wheatley tank and areas around it were fed from CCWD No. 1, the source of which was modeled as a reservoir. In the future simulation, that area is being supplied with water from KRS2 Pool 3. This water travels a longer distance, which increases the water age. The same applies to areas that are currently being fed by GCWD on the far north end of the distribution system.

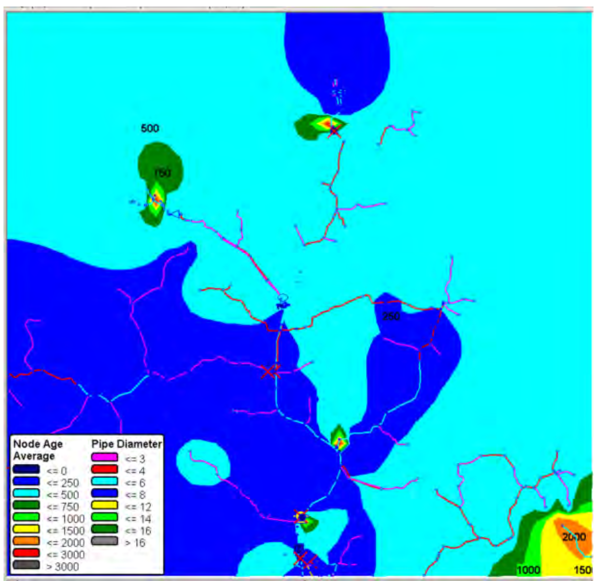


Figure 5.06-5 Northern Portion Average Water Age without Capital Improvements

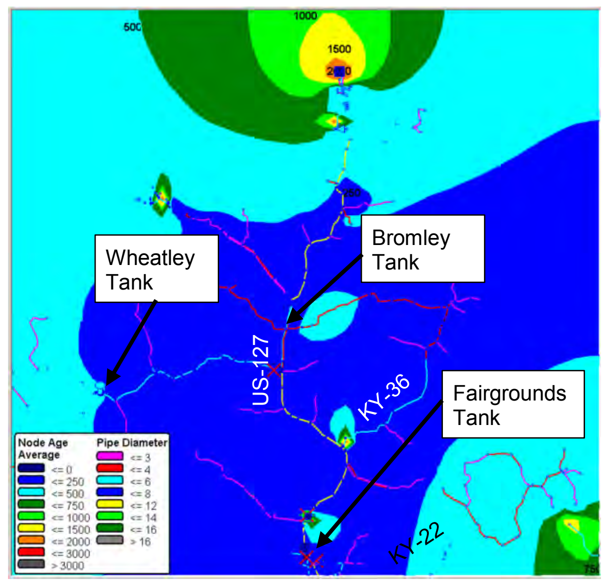


Figure 5.06-6 Northern Portion Average Water Age with Capital Improvements

C. Owenton Service Area

Figures 5.06-7 and 5.06-8 show the average water age results for the current minimum demand day model and the future minimum demand day model with capital improvements for the Owenton service area, respectively. Most of the improvements in this area were designed to address minimum pressures. As shown in Figure 5.06-8, the capital improvements also resulted in a slight decrease in water age at some locations.

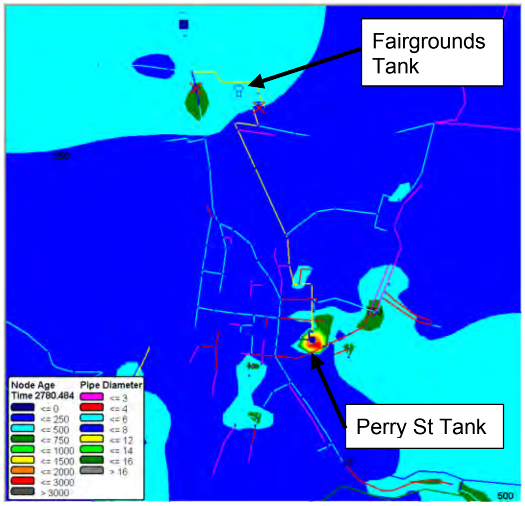


Figure 5.06-7 Owenton Average Water Age Without Capital Improvements

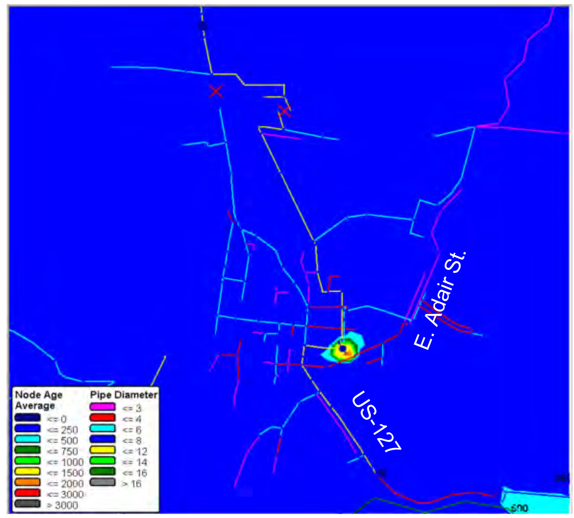


Figure 5.06-8 Owenton Average Water Age With Capital Improvements

5.07 FUTURE MAXIMUM DAY DEMAND SCENARIO RESULTS

Figures 5.07-1 and 5.07-2 show the overall minimum pressure results for the future demand day model without improvements and the future demand day model with capital improvements, respectively. All pressures presented in figures within this section are in psi.

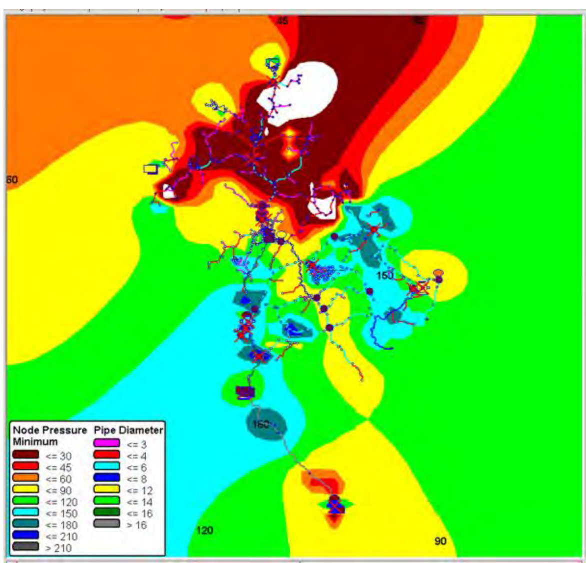


Figure 5.07-1 Northern Division Minimum Pressures Without Capital Improvements

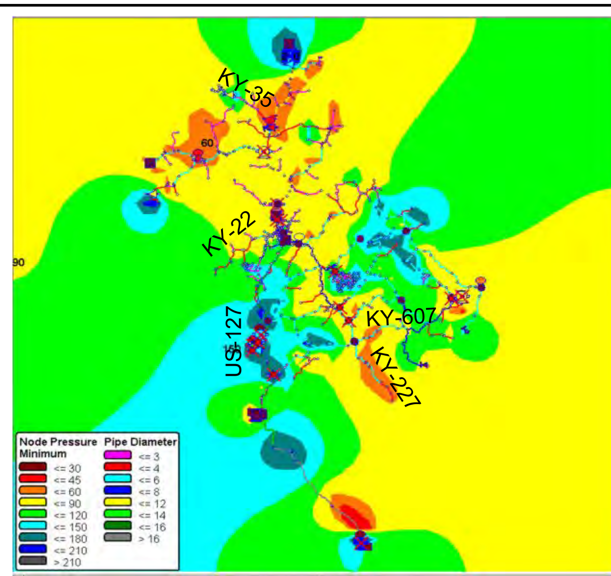


Figure 5.07-2 Northern Division Minimum Pressures With Capital Improvements

Figures 5.07-3 and 5.07-4 show the overall maximum pressure results for the future demand day model without improvements and the future demand day model with capital improvements, respectively.

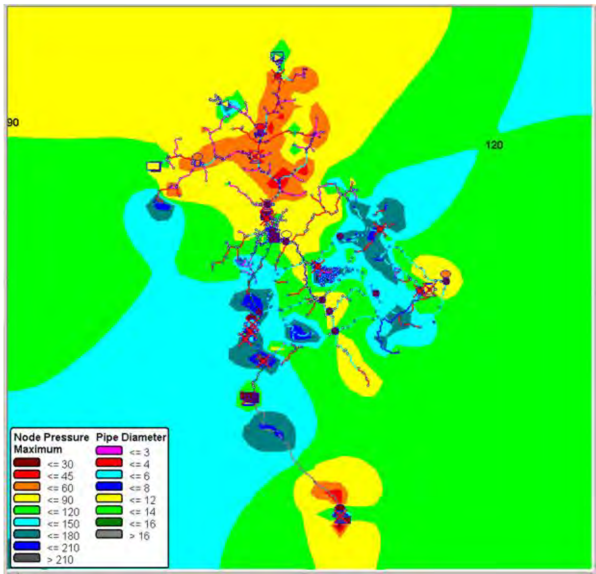


Figure 5.07-3 Northern Division Maximum Pressures Without Capital Improvements

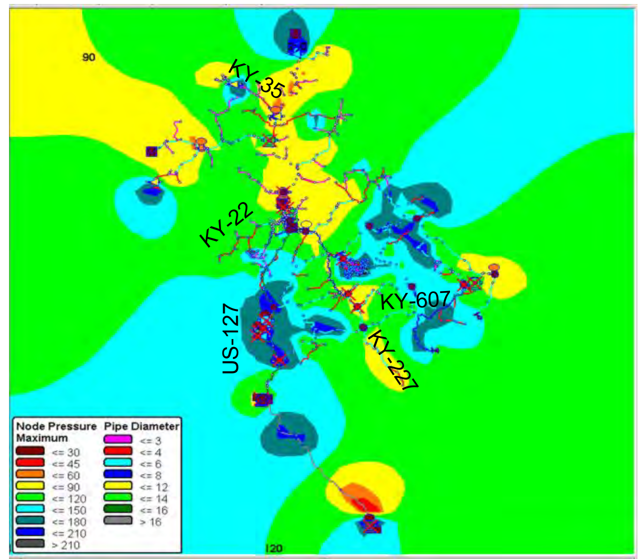


Figure 5.07-4 Northern Division Maximum Pressures With Capital Improvements

As Figure 5.07-1 through 5.07-4 show, overall the distribution system was able to provide adequate pressure after improvements were implemented. The following discussion focuses on the effects of capital improvements to address discreet areas within the distribution system.

A. <u>Northern Portion of the Northern Division</u>

Figures 5.07-5 and 5.07-6 show the minimum pressure results for the areas north of the Fairgrounds tank for the future demand day model without improvements and the future maximum demand day model with capital improvements, respectively.

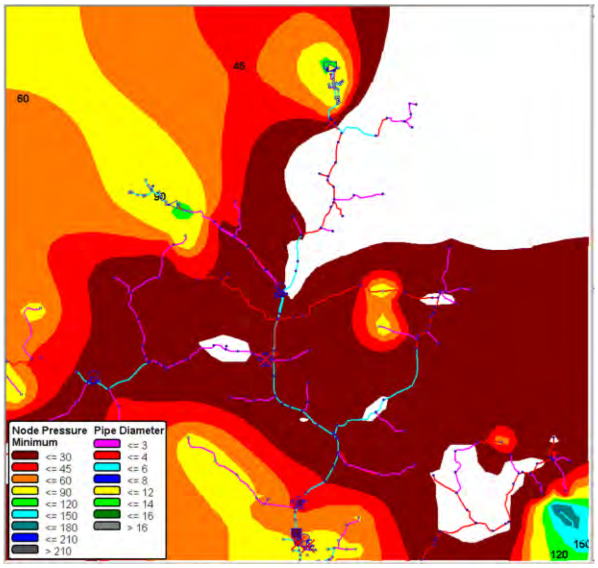


Figure 5.07-5 Northern Area Minimum Pressures Without Capital Improvements

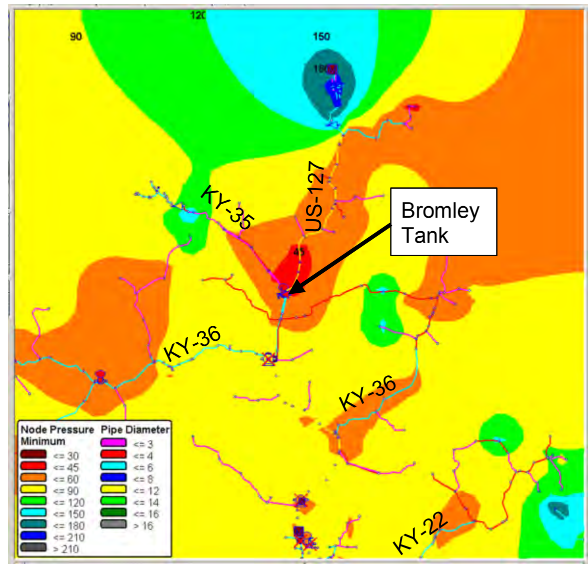


Figure 5.07-6 Northern Area Minimum Pressures With Capital Improvements

The increase in demand of a future maximum day demand and the need to supply water to areas currently served by CCWD No. 1 and GCWD through the existing small diameter piping cause significant head loss, which reduces pressures below 45 psi in some areas. Some areas experienced negative minimum pressures. As seen in Figure 5.07-6, proposed improvements increased minimum pressures above 45 psi for most areas except a few local areas with high elevation. Even for these areas, modeled minimum pressures were above 35 psi. Because of the elevation of these areas, even with Bromley tank at its overflow elevation, minimum pressures will still be below 45 psi.

The areas currently served by GCWD have lower elevations compared to the Bromley tank pressure zone. Modeling results indicated elevated pressures when this area is fed directly by the tank as seen in the northern portion of Figure 5.07-6, which is why a PRV is recommended for this area. The setting of the PRV can be adjusted to provide the desired pressures.

Modeled improvements resulted in increased flow to the Bromley tank. A flow control valve was then incorporated at the influent to the tank to reduce the flow into the Bromley that also allows flow into the Wheatley Tank Service Area.

Other alternatives considered to feed the Bromley tank included activating the pump station along US 127 near KY-978. Activating this pump station did not allow adequate system pressures to be provided.

The upsized pipes along KY-227 and KY-325 also resulted in minimum system pressures above 45 psi for the Wheatley Tank Service Area and areas currently served by CCWD No. 1. Other alternatives considered for the Wheatley Tank Service Area included activating the Wheatley pump station located on KY-227 and the intersection with US-127. However, for this alternative, minimum pressures below 45 psi were experienced in most areas because of significant head losses resulting from the small pipe sizes.

B. Areas Along KY-22

Figure 5.07-7 shows the minimum pressure results for the future maximum demand scenario without capital improvements. Figure 5.07-8 shows that upgrading this section of 4-inch main to 6-inch main increases minimum system pressures above 45 psi.

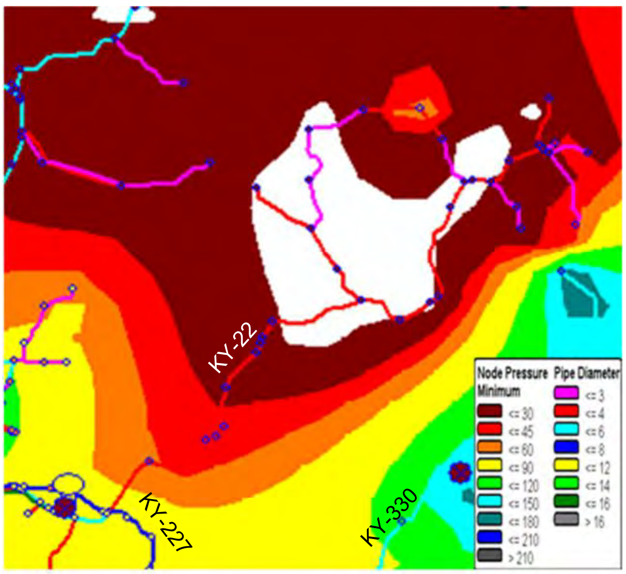


Figure 5.07-7 **Areas along KY-22 Minimum Pressures Without 6-inch Main Upgrade**

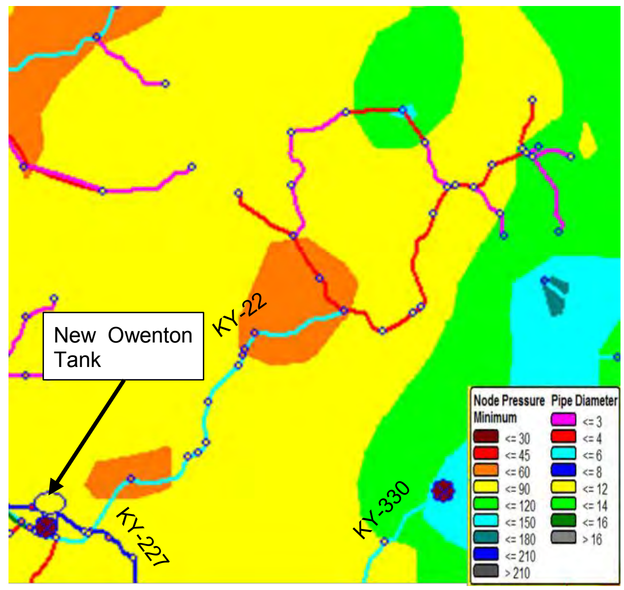


Figure 5.07-8 **Areas along KY-22 Minimum Pressures With 6-inch Main Upgrade**

C. Owenton

The Wheatley area and Glencoe zone are supplied by CCWD No. 1 and GCWD, respectively. Figure 5.07-9 displays minimum pressures in Owenton when supplied by them. Since KAW is interested in serving these areas from its own supply, modeling was conducted to determine the feasibility of this alternate supply. Preliminary results indicated that the Fairgrounds tank was unable to be refilled resulting in low pressures in areas being supplied by the Fairgrounds tank. Mains along Main Street were upsized to address these issues as mentioned previously. The minimum pressure results are displayed in Figure 5.07-10. The Fairgrounds tank was able to be refilled after these improvements. KAW noted water main breaks along additional portions of Main Street north of Seminary Street. Though modeled results were positive without the additional improvements, KAW may want to consider replacing this section of Main Street to address pipe condition deficiencies.

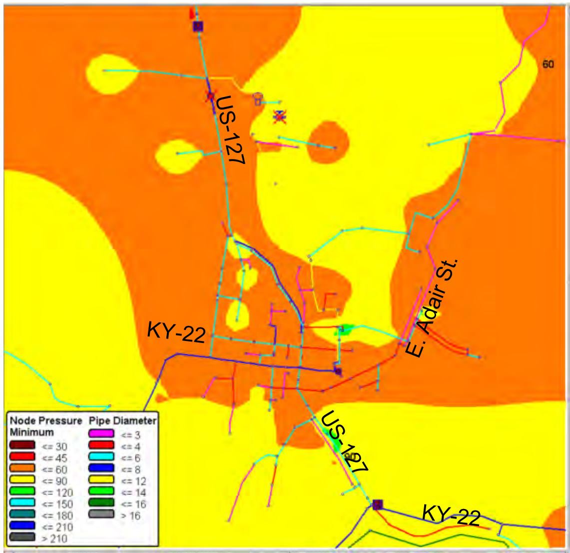


Figure 5.07-9 Owenton Minimum Pressures Without 12-inch Main Upgrade

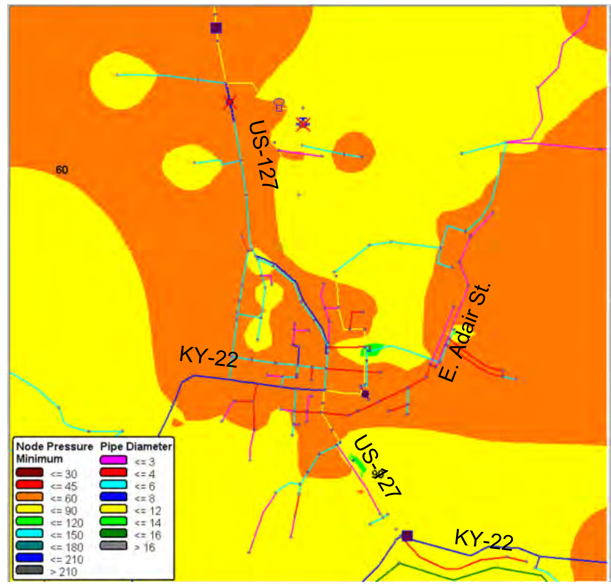


Figure 5.07-10 Owenton Minimum Pressures With 12-inch Main Upgrade

D. <u>Southern Portion of Distribution System</u>

Figures 5.07-11 and 5.07-12 show the minimum pressure results for the southern portion of the system for the future demand day model with two system connectivity alternatives. Under the first alternative, the southern portion of the system was shown to be supplied primarily by the new tank and pumping facilities. The second alternative involves the closing of valves left open in the first alternative. These valve closings are discussed in Section 5.05 G. The figures show that with the second alternate connectivity minimum pressures are slightly lower. The second alternate connectivity will allow most of these areas to be fed by the New Columbus tank to increase tank turnover. The water, therefore, travels a longer distance and does experience more head loss. This reduces the HGL and, hence, system pressures in the area. Modeled minimum pressures were still above 45 psi. To maintain pressures above 45 psi, the New Columbus tank was operated within the top 20 feet.

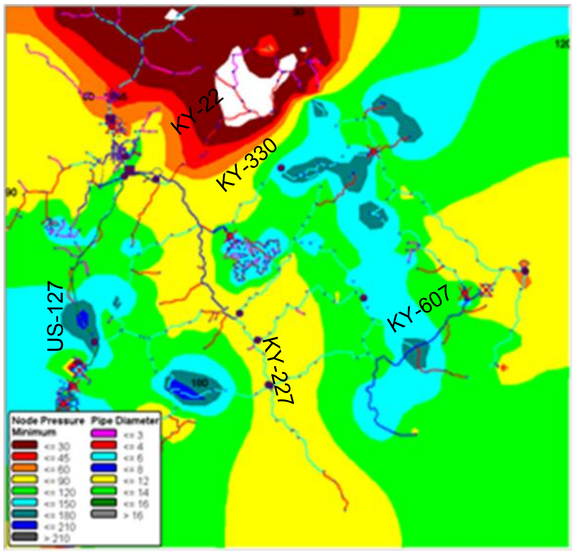


Figure 5.07-11 Southern Portion Minimum Pressures Without Improvements

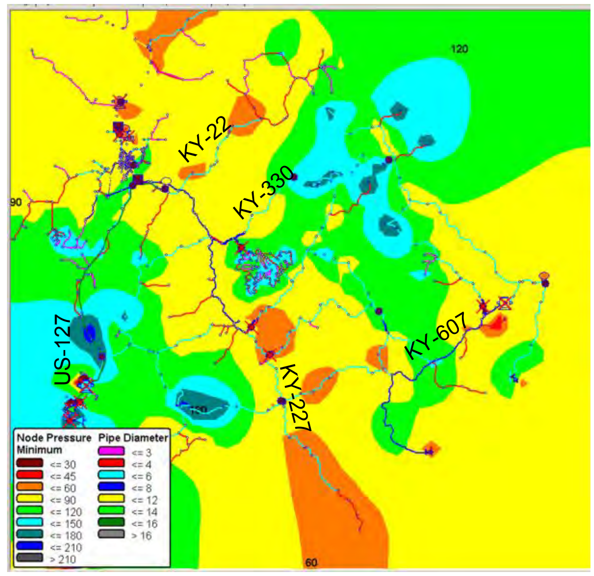


Figure 5.07-12 Southern Portion Minimum Pressures With Improvements

5.08 NORTHERN DIVISION CAPITAL IMPROVEMENT CONCLUSIONS

Overall modeling results indicate the current system cannot typically meet the projected future maximum demand and maintain a minimum pressure of 30 psi in most northern portions of the distribution system. Modeled capital improvements significantly reduced head losses and helped to maintain a minimum pressure of 45 psi in most areas of the system while not substantially increasing maximum pressures. Alternate operation of the New Columbus Tank Area should improve water age in areas served by this tank.

APPENDIX A
CENTRAL DIVISION AVERAGE DEMAND FACILITY OPERATIONS
(*PROVIDED ON CD)*

APPENDIX B
CENTRAL DIVISION MODEL PRESSURE PLOTS

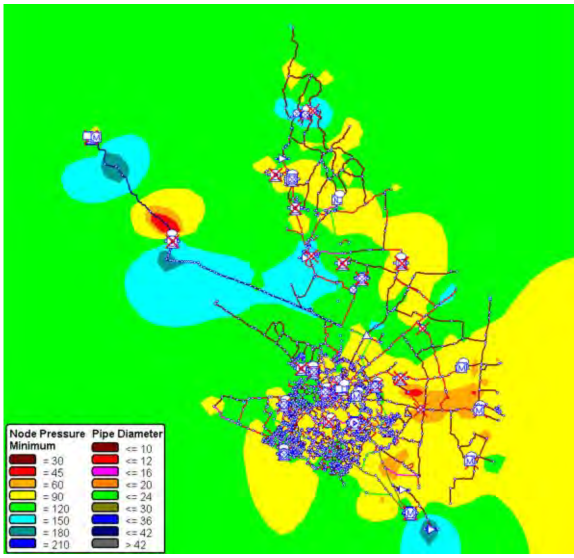


Figure 1 **Current Minimum Demand Day Minimum System Pressure**

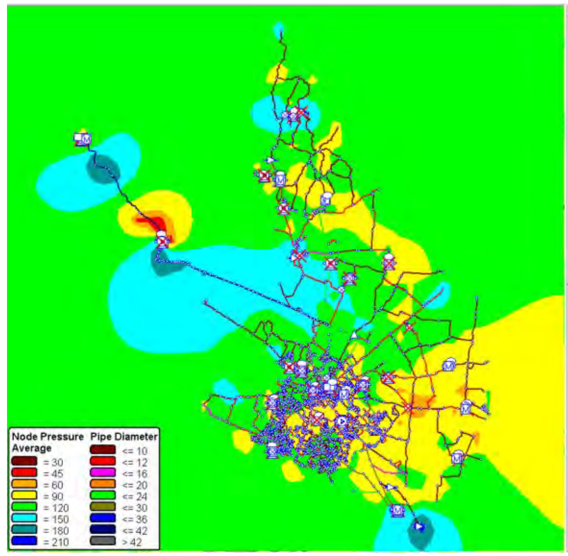


Figure 2 **Current Minimum Demand Day Average System Pressure**

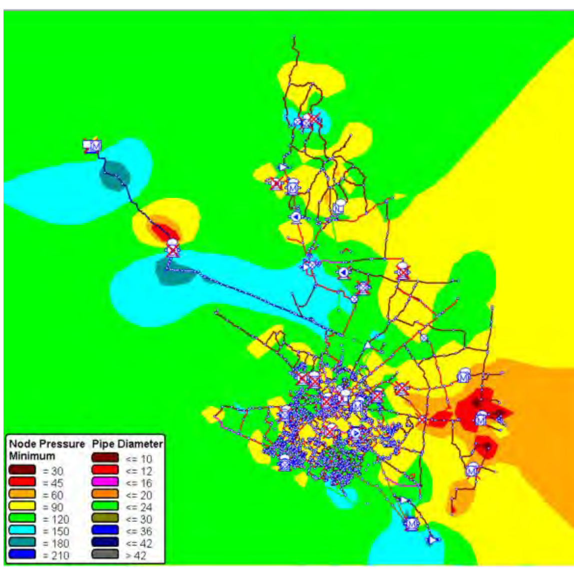


Figure 3 **Current Maximum Demand Day Minimum System Pressure**

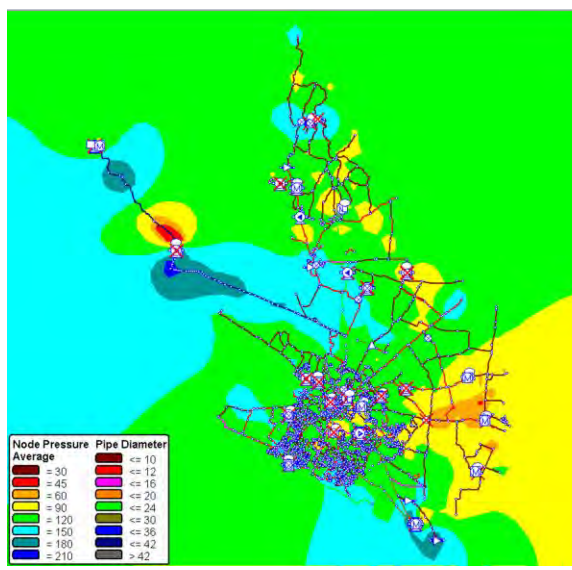


Figure 4 **Current Maximum Demand Day Average System Pressure**

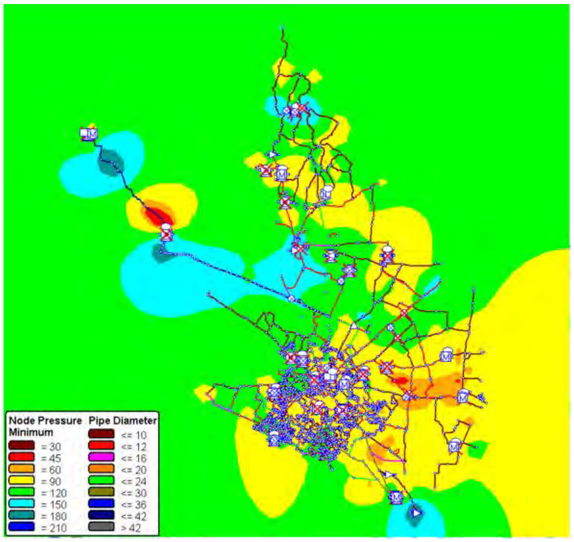


Figure 5 **Future Minimum Demand Day Minimum System Pressure**

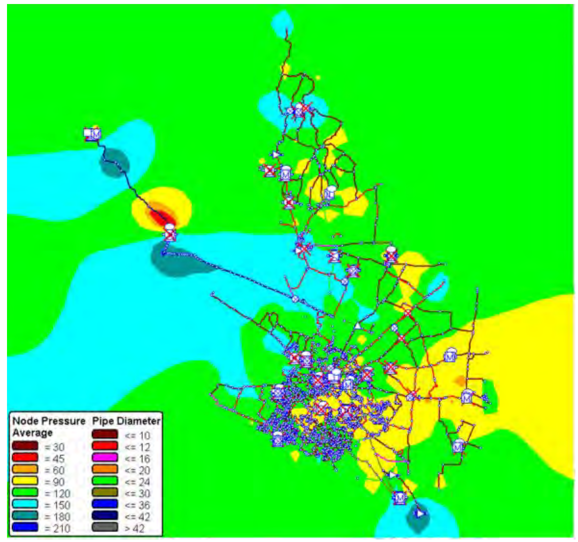


Figure 6 **Future Minimum Demand Day Average System Pressure**

APPENDIX C
CURRENT MINIMUM WATER AGE EXPORTS

Junction Name	Average Modeled Water Age (Hours)
c-6	0
Cox St Model Reservoir	0
Hall Model Reservoir	0
Hume Model Reservoir	0
J-968	0
KRS1	0
KRS-13	0
KRS2 Pool 3	0
Mercer Model Reservoir	0
RRS	0
York Model Reservoir	0
c-1	1
c-10	1
c-4	1
c-5	1
c-8	1
c-9	1
J-971	1
J-976	1
J-978	1
10607	2
10608	2
c-11	2
c-2	2
c-20	2
c-21	2
c-3	2
J-6126	2
J-7281	2
J-8515	2
J-977	2
J-983	2
KRS2 HS No. 2	2
RRS-9	2
10605	3
10606	3
5939	3
c-19	3

Junction Name	Average Modeled Water Age (Hours)
J-1046	3
J-7595	3
J-808	3
10588	4
5954	4
c-15	4
J-137	4
J-634	4
J-816	4
New No. 2	4
New No. 3	4
RRSa	4
5417	5
5550	5
5588	5
5933	5
5941	5
5947	5
c-12	5
c-13	5
c-14	5
J-636	5
J-755	5
J-811	5
J-815	5
4506	6
5268	6
5556	6
5606	6
6173	6
J-559	6
J-757	6
J-784	6
J-813	6
KRS-11	6
RRS-8	6
10261	7
4108	7
4117	7
4392	7

Junction Name	Average Modeled Water Age (Hours)
4813	7
5815	7
5829	7
6040	7
6401	7
6643	7
6645	7
6758	7
J-373	7
J-814	7
J-817	7
J-967	7
10314	8
4404	8
4759	8
5851	8
5853	8
6504	8
6622	8
6626	8
6765	8
9029	8
c-16	8
J-640	8
J-656	8
J-658	8
RRS-6	8
10061	9
10089	9
4111	9
4118	9
4125	9
4152	9
4180	9
4249	9
4356	9
4367	9
4397	9
5695	9
5794	9

Junction Name	Average Modeled Water Age (Hours)
6082	9
6138	9
6576	9
6598	9
7057	9
9813	9
9832	9
9921	9
9928	9
J-204	9
J-206	9
J-208	9
J-375	9
J-639	9
J-659	9
J-975	9
J-984	9
RV-6	9
10108	10
10112	10
10579	10
3754	10
3977	10
4527	10
4544	10
4752	10
4904	10
5669	10
5959	10
5964	10
6393	10
6554	10
6563	10
8124	10
8126	10
9458	10
9557	10
9595	10
9794	10
9834	10

Junction Name	Average Modeled Water Age (Hours)
J-124	10
J-205	10
J-207	10
J-210	10
J-212	10
J-221	10
J-436	10
J-548	10
J-660	10
J-985	10
10103	11
10148	11
10577	11
10578	11
3695	11
3706	11
3724	11
3735	11
3737	11
3740	11
3741	11
4129	11
4293	11
5769	11
5859	11
6383	11
6667	11
7428	11
7660	11
7684	11
9186	11
9338	11
9425	11
9487	11
9501	11
9664	11
9682	11
9686	11
9754	11
J-127	11

Junction Name	Average Modeled Water Age (Hours)
J-128	11
J-211	11
J-220	11
J-780	11
J-809	11
10217	12
10220	12
10273	12
10275	12
10562	12
3552	12
3568	12
3641	12
3643	12
3678	12
3682	12
3693	12
5356	12
5495	12
6443	12
6454	12
7230	12
7414	12
7420	12
7434	12
7500	12
9245	12
9398	12
9439	12
9516	12
9653	12
9678	12
9702	12
9799	12
J-1013	12
J-135	12
J-224	12
J-685	12
J-690	12
J-779	12

Junction Name	Average Modeled Water Age (Hours)
J-782	12
10100	13
10130	13
10296	13
10313	13
3550	13
3602	13
3616	13
3633	13
3692	13
3865	13
3950	13
5308	13
5494	13
6300	13
6384	13
6390	13
6394	13
6811	13
6899	13
6960	13
7004	13
7013	13
7139	13
7507	13
7604	13
7710	13
7743	13
8801	13
9185	13
9222	13
9246	13
9277	13
9334	13
9397	13
9562	13
9563	13
9655	13
9820	13
J-138	13

Junction Name	Average Modeled Water Age (Hours)
J-686	13
J-687	13
J-692	13
10300	14
3531	14
4780	14
5178	14
5357	14
5435	14
5803	14
6331	14
6473	14
6607	14
6708	14
6794	14
6873	14
6949	14
7484	14
7553	14
7802	14
8714	14
8798	14
9022	14
9143	14
9195	14
9232	14
9288	14
9291	14
9773	14
9777	14
9783	14
J-226	14
J-691	14
10252	15
10302	15
10346	15
3808	15
3903	15
5337	15
5688	15

Junction Name	Average Modeled Water Age (Hours)
5704	15
5923	15
6096	15
6121	15
6205	15
6491	15
6540	15
6579	15
6592	15
6706	15
7114	15
7492	15
7520	15
7986	15
8026	15
8691	15
9002	15
9015	15
9165	15
9182	15
9623	15
9641	15
9839	15
H368	15
H3784	15
J-129	15
J-688	15
J-698	15
J-878	15
10060	16
10201	16
10232	16
10340	16
10342	16
10350	16
10497	16
3566	16
4969	16
5707	16
6014	16

Junction Name	Average Modeled Water Age (Hours)
6044	16
6091	16
6103	16
6193	16
6236	16
6281	16
6299	16
6308	16
6348	16
6516	16
6678	16
6856	16
7002	16
7464	16
7467	16
8516	16
8684	16
8862	16
8870	16
9000	16
9310	16
9407	16
9624	16
9782	16
9846	16
9871	16
9886	16
J-131	16
J-693	16
J-694	16
J-695	16
J-783	16
10020	17
10396	17
10398	17
10403	17
3236	17
3544	17
3573	17
3680	17

Junction Name	Average Modeled Water Age (Hours)
3851	17
4153	17
4154	17
4575	17
5667	17
6285	17
6290	17
6361	17
7007	17
7079	17
7161	17
7167	17
7169	17
7275	17
7349	17
7645	17
7760	17
7857	17
7913	17
8837	17
8840	17
8868	17
8921	17
8951	17
8952	17
8966	17
9171	17
9305	17
9446	17
9479	17
H4347	17
J-1012	17
J-1061	17
J-130	17
J-217	17
J-663	17
J-683	17
J-987	17
10012	18
10013	18

Junction Name	Average Modeled Water Age (Hours)
10332	18
10333	18
10347	18
10404	18
10426	18
10504	18
3288	18
3384	18
3393	18
3696	18
4812	18
4855	18
4893	18
5622	18
5741	18
5826	18
5865	18
5892	18
5990	18
6110	18
6929	18
7264	18
7369	18
7939	18
7985	18
8454	18
8469	18
8539	18
8605	18
8619	18
8824	18
8828	18
8865	18
9034	18
9441	18
9734	18
H4348	18
J-168	18
J-196	18
J-654	18

Junction Name	Average Modeled Water Age (Hours)
J-661	18
J-846	18
J-847	18
10247	19
10297	19
10436	19
10445	19
10487	19
10488	19
10509	19
3338	19
3662	19
3677	19
3690	19
3734	19
4678	19
5115	19
5501	19
5587	19
5616	19
5679	19
5713	19
5755	19
6673	19
6818	19
6944	19
7156	19
7273	19
7360	19
7683	19
8201	19
8254	19
8396	19
8425	19
8542	19
8678	19
8727	19
8838	19
8958	19
8994	19

Junction Name	Average Modeled Water Age (Hours)
9105	19
9911	19
9977	19
J-1011	19
J-133	19
J-223	19
J-419	19
J-645	19
J-662	19
J-849	19
10121	20
10237	20
10278	20
3334	20
3607	20
3637	20
3661	20
3666	20
3709	20
4648	20
4905	20
5215	20
5223	20
5227	20
5299	20
5487	20
5498	20
5692	20
5791	20
5834	20
5839	20
6669	20
6872	20
7047	20
7277	20
7392	20
7408	20
7424	20
7437	20
7691	20

Junction Name	Average Modeled Water Age (Hours)
7886	20
7903	20
8034	20
8103	20
8214	20
8327	20
8467	20
8486	20
8720	20
8721	20
8751	20
8797	20
8836	20
8869	20
8987	20
9261	20
9975	20
9982	20
J-1010	20
PR-CHILESBURG	20
10002	21
10180	21
10212	21
10269	21
3221	21
3574	21
4036	21
4602	21
4624	21
4840	21
5083	21
5479	21
5497	21
5551	21
6313	21
6684	21
6863	21
7091	21
7101	21

Junction Name	Average Modeled Water Age (Hours)
7252	21
7400	21
7643	21
7877	21
7914	21
7962	21
7977	21
8110	21
8585	21
8606	21
8690	21
9459	21
CHIL	21
J-157	21
J-161	21
J-213	21
J-27	21
J-420	21
J-431	21
J-652	21
10166	22
3184	22
3472	22
4611	22
4984	22
5074	22
5831	22
6912	22
7024	22
7046	22
7131	22
7318	22
7545	22
7855	22
7984	22
8014	22
8084	22
8494	22
8572	22
8587	22

Junction Name	Average Modeled Water Age (Hours)
8651	22
8733	22
9292	22
9427	22
9454	22
9490	22
9575	22
9970	22
H4479	22
J-119	22
J-132	22
J-153	22
J-155	22
J-172	22
J-988	22
J-990	22
J-991	22
3224	23
4246	23
5419	23
7163	23
7205	23
7211	23
7276	23
7409	23
7537	23
7562	23
7723	23
8056	23
8270	23
9039	23
9685	23
J-105	23
J-121	23
J-29	23
J-429	23
J-430	23
J-479	23
J-595	23
J-643	23

Junction Name	Average Modeled Water Age (Hours)
J-758	23
J-840	23
J-992	23
10042	24
10083	24
3050	24
3372	24
3459	24
4613	24
4980	24
5019	24
6717	24
6915	24
6975	24
7220	24
7322	24
7367	24
7578	24
7682	24
7884	24
7908	24
8033	24
8294	24
9507	24
9856	24
H3821	24
H3936	24
J-122	24
J-150	24
J-171	24
J-378	24
J-379	24
J-416	24
10131	25
10464	25
2915	25
3199	25
3665	25
4452	25
4456	25

Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)
4978	25	7186	26	7005	27
5844	25	7223	26	7218	27
6702	25	7268	26	7278	27
6916	25	7287	26	7327	27
6945	25	7342	26	7508	27
7286	25	7790	26	7647	27
7357	25	8588	26	7848	27
7786	25	8653	26	8068	27
8387	25	9010	26	8297	27
8456	25	9406	26	8617	27
8583	25	9881	26	8884	27
9765	25	9913	26	9127	27
H3702	25	9918	26	9505	27
J-1018	25	J-114	26	9826	27
J-120	25	J-152	26	9958	27
J-417	25	J-162	26	9983	27
J-418	25	J-169	26	J-1009	27
J-644	25	J-175	26	J-170	27
J-751	25	J-422	26	J-376	27
PR-REYNOLDS	25	J-678	26	J-426	27
PR-STRADER	25	10048	27	J-705	27
10022	26	10133	27	10187	28
10221	26	10193	27	10317	28
10276	26	10414	27	10318	28
2824	26	2730	27	10328	28
2854	26	2826	27	10353	28
3098	26	3087	27	2592	28
3672	26	3256	27	2733	28
3701	26	3332	27	2883	28
4666	26	3668	27	3008	28
5088	26	4035	27	4601	28
5997	26	4895	27	4798	28
6561	26	5371	27	4839	28
6565	26	5500	27	5376	28
6587	26	5938	27	5592	28
6619	26	6689	27	6699	28
6697	26	6721	27	6876	28
6725	26	6749	27	7140	28
6909	26	6855	27	7247	28
6938	26	6871	27	8039	28

Junction Name	Average Modeled Water Age (Hours)
8049	28
8144	28
8153	28
8154	28
8187	28
8227	28
8670	28
9986	28
J-1016	28
J-198	28
J-209	28
J-259	28
J-382	28
J-424	28
J-681	28
J-704	28
J-715	28
RV-8	28
10050	29
10152	29
10234	29
10260	29
10266	29
10339	29
10343	29
10354	29
10358	29
3133	29
3565	29
4562	29
4612	29
5217	29
5547	29
5573	29
5583	29
6662	29
6672	29
6677	29
6709	29
6711	29

Junction Name	Average Modeled Water Age (Hours)
7361	29
7521	29
7829	29
7872	29
7920	29
7976	29
7987	29
8186	29
8322	29
8369	29
8437	29
8650	29
8858	29
8906	29
8912	29
9130	29
9724	29
J-1019	29
J-1040	29
J-421	29
J-881	29
J-882	29
RV-9	29
Tates Creek	29
2526	30
2892	30
3067	30
3121	30
3183	30
3368	30
3598	30
3710	30
4339	30
4485	30
5005	30
5309	30
6666	30
6963	30
7352	30
7456	30

Junction Name	Average Modeled Water Age (Hours)
8077	30
8137	30
9007	30
9035	30
9103	30
9110	30
9126	30
9420	30
9556	30
9887	30
J-1020	30
J-381	30
J-423	30
J-804	30
J-842	30
J-879	30
J-883	30
10037	31
10244	31
10406	31
2888	31
3122	31
3158	31
3171	31
4276	31
4884	31
5063	31
5289	31
6039	31
6175	31
6591	31
7491	31
7809	31
8041	31
8228	31
9106	31
9823	31
J-199	31
J-415	31
10125	32

Junction Name	Average Modeled Water Age (Hours)
10315	32
2392	32
2961	32
2987	32
3119	32
5552	32
5694	32
6038	32
6509	32
6776	32
7340	32
7481	32
7517	32
8007	32
9335	32
9735	32
J- 2317	32
J-1	32
J-222	32
J-802	32
J-993	32
Woodlake Tank No. 1	32
10017	33
10214	33
2258	33
2361	33
2388	33
2426	33
2732	33
3114	33
3157	33
3242	33
3640	33
5614	33
5763	33
5808	33
6310	33
6321	33
6327	33

Junction Name	Average Modeled Water Age (Hours)
6343	33
7014	33
7074	33
9163	33
9944	33
J-1006	33
J-1023	33
J-425	33
J-707	33
J-799	33
J-946	33
2287	34
2801	34
2820	34
3053	34
4183	34
4372	34
5647	34
5665	34
5714	34
6168	34
6231	34
6781	34
7416	34
7461	34
J-356	34
J-427	34
J-433	34
J-805	34
J-961	34
Woodlake No. 2	34
3487	35
3615	35
4326	35
4946	35
5643	35
6771	35
6859	35
9619	35
9720	35

Junction Name	Average Modeled Water Age (Hours)
H4040	35
J-1044	35
J-115	35
J-357	35
J-957	35
J-994	35
POOL3 NODE	35
10216	36
2841	36
3716	36
4267	36
5285	36
5863	36
6025	36
6076	36
6438	36
6477	36
6797	36
6865	36
9432	36
9716	36
9728	36
9732	36
H4041	36
J-1041	36
J-1042	36
J-2	36
J-942	36
J-943	36
2901	37
4330	37
4390	37
4589	37
5166	37
5396	37
5522	37
5946	37
6171	37
6868	37
7359	37

Junction Name	Average Modeled Water Age (Hours)
9626	37
9715	37
J-1045	37
J-166	37
J-353	37
J-939	37
J-940	37
J-941	37
2291	38
3685	38
4123	38
5960	38
6143	38
6379	38
6595	38
9191	38
J-107	38
J-123	38
J-125	38
J-597	38
J-936	38
J-937	38
J-938	38
10306	39
10459	39
2983	39
3089	39
4235	39
4850	39
5034	39
5686	39
5782	39
6081	39
6215	39
6457	39
6655	39
6777	39
9625	39
H2339	39
H3242	39

Junction Name	Average Modeled Water Age (Hours)
J-1024	39
J-158	39
J-354	39
J-754	39
J-933	39
J-934	39
J-935	39
2384	40
2909	40
4290	40
4498	40
4642	40
5132	40
5648	40
5660	40
5790	40
5804	40
5849	40
6613	40
7646	40
J-1008	40
J-1025	40
J-109	40
J-111	40
J-598	40
J-930	40
J-932	40
5075	41
5654	41
7092	41
7094	41
7118	41
7666	41
9257	41
H2338	41
J-1058	41
J-163	41
J-350	41
J-752	41
J-928	41

Junction Name	Average Modeled Water Age (Hours)
J-929	41
J-997	41
5529	42
5627	42
5687	42
6541	42
6812	42
6816	42
J-151	42
J-360	42
J-701	42
J-798	42
J-925	42
J-926	42
J-927	42
2177	43
4122	43
4303	43
5545	43
5664	43
5904	43
5924	43
6906	43
J-1043	43
J-1064	43
J-352	43
J-428	43
J-753	43
J-922	43
J-924	43
2741	44
5445	44
5481	44
6163	44
7510	44
7797	44
8712	44
8969	44
9207	44
9229	44

Junction Name	Average Modeled Water Age (Hours)
J-919	44
J-920	44
J-921	44
J-996	44
2012	45
2162	45
2939	45
2985	45
2993	45
4192	45
5441	45
5927	45
6668	45
7720	45
8590	45
8859	45
8973	45
9616	45
J-364	45
J-796	45
J-916	45
J-918	45
J-963	45
Woodlake No. 1	45
4250	46
4762	46
5008	46
5858	46
5993	46
6117	46
6691	46
6914	46
9020	46
9355	46
J-1065	46
J-159	46
J-351	46
J-405	46
J-915	46
J-917	46

Junction Name	Average Modeled Water Age (Hours)
3430	47
4164	47
4374	47
5464	47
6835	47
7044	47
8804	47
9073	47
9107	47
J-102	47
J-1026	47
J-113	47
J-407	47
J-409	47
J-911	47
J-912	47
J-914	47
2695	48
4065	48
4101	48
4797	48
5403	48
6940	48
7854	48
8724	48
8750	48
J-411	48
J-684	48
J-910	48
2769	49
4093	49
5362	49
5966	49
6903	49
6935	49
9135	49
J-126	49
J-908	49
J-909	49
10066	50

Junction Name	Average Modeled Water Age (Hours)
10455	50
1937	50
2178	50
2823	50
4144	50
4223	50
5331	50
7059	50
8384	50
8759	50
9412	50
J-771	50
J-906	50
J-907	50
1906	51
2931	51
4263	51
4764	51
5323	51
6639	51
6741	51
7037	51
7192	51
J-1005	51
J-343	51
J-408	51
J-412	51
1922	52
4901	52
6534	52
6824	52
6989	52
7672	52
7959	52
J-1022	52
J-1027	52
J-410	52
J-904	52
J-905	52
2829	53

Junction Name	Average Modeled Water Age (Hours)
2893	53
5264	53
5281	53
7071	53
7332	53
9025	53
J-903	53
J-999	53
2376	54
3086	54
4037	54
5341	54
5835	54
6399	54
6866	54
7701	54
8896	54
9225	54
9400	54
J-370	54
J-902	54
2873	55
5359	55
6389	55
6762	55
6763	55
7236	55
7243	55
7579	55
7705	55
7820	55
8004	55
J-367	55
J-368	55
J-900	55
J-901	55
1886	56
3044	56
4317	56
6937	56
7165	56
7429	56
7532	56
8218	56
9094	56
EASTLAND	56
J-1060	56
J-406	56
J-897	56
J-899	56
4631	57
5104	57
5172	57
5255	57
7258	57
7466	57
7856	57
8028	57
8072	57
8476	57
9082	57
9442	57
H3115	57
J-156	57
J-896	57
1512	58
1667	58
2764	58
2941	58
4043	58
4105	58
7246	58
8545	58
J-160	58
J-895	58
1506	59
2137	59
2338	59
2383	59
2602	59
2929	59
3848	59
4076	59
4411	59
5057	59
5415	59
5719	59
7109	59
7597	59
7927	59
9118	59
9467	59
H2730	59
J-894	59
2058	60
2062	60
2296	60
2344	60
9639	60
H2768	60
J-112	60
J-366	60
J-891	60
J-944	60
1647	61
2044	61
2091	61
4007	61
9343	61
9564	61
J-349	61
J-414	61
1585	62
1681	62
1689	62
2181	62
2956	62
3645	62
4768	62
5106	62

Junction Name	Average Modeled Water Age (Hours)
5235	62
7814	62
8591	62
9339	62
9416	62
9608	62
J-948	62
J-949	62
RV-10	62
1800	63
4006	63
5130	63
6474	63
7863	63
8087	63
9290	63
J-950	63
MARIA	63
2061	64
2212	64
2382	64
7875	64
8333	64
8812	64
J-1001	64
J-365	64
J-778	64
J-893	64
J-951	64
J-952	64
10618	65
1492	65
3162	65
7926	65
8168	65
9303	65
J-1056	65
J-108	65
J-954	65
J-955	65

Junction Name	Average Modeled Water Age (Hours)
10263	66
1413	66
1416	66
1823	66
2104	66
2299	66
2355	66
7924	66
7942	66
9230	66
9279	66
10661	67
1468	67
1482	67
2156	67
3831	67
5011	67
5032	67
7242	67
7331	67
7468	67
J-413	67
J-956	67
J-969	67
J-974	67
1552	68
1832	68
2134	68
2213	68
498	68
500	68
5068	68
5107	68
5476	68
8430	68
J-945	68
J-947	68
1702	69
1775	69
2014	69

Junction Name	Average Modeled Water Age (Hours)
3498	69
3588	69
488	69
505	69
506	69
5591	69
7995	69
8216	69
J-1053	69
J-733	69
474	70
8202	70
9167	70
9218	70
J-1002	70
J-735	70
J-776	70
1723	71
2821	71
3056	71
7070	71
7097	71
J-380	71
J-736	71
1325	72
1765	72
2485	72
593	72
7245	72
7554	72
8378	72
8638	72
J-139	72
J-347	72
J-398	72
J-740	72
J-742	72
1370	73
1371	73
1424	73

Junction Name	Average Modeled Water Age (Hours)
1457	73
1910	73
2132	73
3063	73
3226	73
540	73
541	73
9155	73
9296	73
J-1003	73
J-22	73
J-24	73
J-966	73
J-99	73
2386	74
7115	74
7300	74
7430	74
9138	74
9451	74
J-1038	74
J-507	74
J-511	74
J-739	74
1545	75
2125	75
2233	75
377	75
5047	75
671	75
7248	75
7556	75
9037	75
J-1004	75
J-1037	75
J-345	75
J-402	75
J-403	75
J-513	75
J-514	75
J-515	75
J-591	75
J-738	75
J-743	75
J-749	75
J-750	75
J-769	75
1244	76
2066	76
2245	76
374	76
3946	76
806	76
8080	76
851	76
J- 5945	76
J-1063	76
J-594	76
J-732	76
1195	77
1227	77
1229	77
1517	77
1678	77
2330	77
2374	77
665	77
7628	77
8121	77
898	77
9074	77
9192	77
BH No. 1	77
J-1062	77
J-117	77
J-18	77
J-23	77
J-516	77
J-517	77
J-518	77
J-599	77
10149	78
1106	78
1221	78
1223	78
1261	78
1303	78
1306	78
1555	78
2115	78
355	78
7341	78
748	78
831	78
8382	78
931	78
962	78
J-11	78
J-229	78
J-28	78
J-737	78
J-970	78
10281	79
1172	79
1184	79
1187	79
1209	79
1302	79
1305	79
1553	79
2000	79
2235	79
2278	79
2308	79
2676	79
2759	79
3083	79
309	79
317	79
3939	79

Junction Name	Average Modeled Water Age (Hours)
404	79
788	79
822	79
9123	79
J- 4305	79
J-30	79
J-362	79
J-43	79
J-775	79
J-958	79
1156	80
1163	80
1455	80
2335	80
282	80
2822	80
312	80
3937	80
401	80
7356	80
7509	80
J-280	80
J-286	80
J-329	80
J-330	80
J-335	80
J-34	80
J-363	80
J-45	80
J-520	80
J-555	80
J-602	80
J-649	80
J-650	80
J-730	80
J-97	80
1132	81
1144	81
1165	81
2013	81

Junction Name	Average Modeled Water Age (Hours)
2962	81
3052	81
3402	81
7290	81
7572	81
8100	81
837	81
8386	81
9008	81
J-266	81
J-294	81
J-32	81
J-325	81
J-348	81
J-37	81
J-44	81
J-519	81
J-522	81
J-601	81
1260	82
1472	82
1504	82
2159	82
8167	82
838	82
J-298	82
J-346	82
J-523	82
J-553	82
J-554	82
J-557	82
J-633	82
J-646	82
J-647	82
RV-5	82
10429	83
10662	83
1121	83
1264	83
2927	83

Junction Name	Average Modeled Water Age (Hours)
797	83
J-20	83
J-281	83
J-331	83
J-41	83
J-57	83
J-638	83
J-648	83
J-964	83
J-980	83
1567	84
2059	84
215	84
216	84
2665	84
756	84
8778	84
J-227	84
J-296	84
J-31	84
J-324	84
J-361	84
J-38	84
J-40	84
J-521	84
J-525	84
J-600	84
J-767	84
J-839	84
1171	85
1219	85
203	85
2192	85
3107	85
6778	85
7596	85
800	85
J-1054	85
J-1057	85
J-230	85

Junction Name	Average Modeled Water Age (Hours)
J-231	85
J-300	85
J-509	85
J-765	85
RV-13	85
200	86
202	86
2960	86
4139	86
7535	86
9141	86
9404	86
J-290	86
J-33	86
J-39	86
J-58	86
313	87
318	87
4767	87
8030	87
820	87
J-1051	87
J-232	87
J-234	87
J-326	87
J-42	87
10654	88
193	88
7323	88
7784	88
7807	88
872	88
J-101	88
J-344	88
J-47	88
J-508	88
10656	89
1152	89
2466	89
689	89

Junction Name	Average Modeled Water Age (Hours)
7544	89
7727	89
8105	89
880	89
890	89
9295	89
J-100	89
J-262	89
J-762	89
J-768	89
J-98	89
2719	90
3383	90
741	90
7626	90
863	90
J-328	90
J-400	90
J-48	90
J-528	90
J-550	90
J-551	90
J-655	90
J-664	90
J-744	90
J-760	90
1039	91
1193	91
2238	91
3781	91
700	91
886	91
9187	91
J-195	91
J-327	91
J-512	91
J-549	91
J-651	91
J-772	91
J-773	91

Junction Name	Average Modeled Water Age (Hours)
1059	92
1294	92
179	92
5062	92
979	92
J-201	92
J-52	92
J-546	92
J-55	92
J-561	92
J-666	92
J-667	92
1000	93
1089	93
1090	93
1105	93
3109	93
764	93
7764	93
777	93
781	93
7970	93
8114	93
867	93
J-12	93
J-248	93
J-49	93
J-53	93
J-564	93
J-568	93
J-668	93
J-731	93
J-745	93
New No. 1	93
1007	94
1017	94
1021	94
1082	94
2813	94
706	94

Junction Name	Average Modeled Water Age (Hours)
8044	94
833	94
989	94
J-194	94
J-235	94
J-236	94
J-292	94
J-747	94
J-761	94
218	95
2394	95
6017	95
7758	95
7868	95
810	95
8601	95
J-14	95
J-359	95
J-759	95
J-824	95
PR-MARIA	95
1211	96
3130	96
586	96
722	96
7972	96
J-186	96
J-187	96
J-284	96
J-533	96
165	97
3060	97
611	97
7752	97
7891	97
8013	97
843	97
892	97
AV-2	97
J-285	97

Junction Name	Average Modeled Water Age (Hours)
J-4	97
J-529	97
J-77	97
1989	98
8132	98
J-237	98
J-530	98
J-547	98
J-7	98
RV-12	98
1086	99
1295	99
1346	99
164	99
2202	99
2265	99
3447	99
8241	99
9086	99
H3959	99
J-297	99
J-322	99
J-766	99
J-792	99
J-841	99
167	100
288	100
J-249	100
J-307	100
J-397	100
J-578	100
RV-11	100
1288	101
2204	101
2714	101
8050	101
8133	101
8344	101
8749	101
J-16	101

Junction Name	Average Modeled Water Age (Hours)
J-245	101
J-536	101
J-556	101
J-795	101
J-821	101
10652	102
10659	102
1180	102
143	102
2150	102
219	102
289	102
3155	102
5145	102
J-283	102
J-532	102
J-534	102
J-818	102
J-823	102
J-844	102
J-953	102
1112	103
1293	103
140	103
141	103
3473	103
H3958	103
J-265	103
J-299	103
J-510	103
J-552	103
J-790	103
J-825	103
1939	104
4300	104
8352	104
9311	104
2381	105
8784	105
J-268	105

Junction Name	Average Modeled Water Age (Hours)
10631	106
10653	106
10655	106
1153	106
2994	106
3172	106
8166	106
J-247	106
J-358	106
J-819	106
1133	107
1160	107
3176	107
5026	107
J- 4309	107
J-17	107
J-270	107
J-577	107
J-61	107
J-8	107
10660	108
2077	108
J-15	108
J-291	108
J-537	108
J-848	108
J-995	108
10632	109
1983	109
8236	109
J-188	109
J-239	109
J-489	109
J-59	109
J-791	109
8574	110
8756	110
J-485	110
J-56	110
10619	111

Junction Name	Average Modeled Water Age (Hours)
2437	111
3595	111
4843	111
8658	111
J-233	111
J-238	111
J-535	111
J-543	111
J-545	111
J-569	111
J-832	111
J-9	111
2349	112
2757	112
8744	112
J-154	112
J-399	112
J-567	112
J-669	112
J-711	112
J-826	112
PR-TOYOTA	112
2100	113
3774	113
7991	113
8408	113
8680	113
J-287	113
J-619	113
2809	114
3031	114
8362	114
8498	114
J-190	114
J-267	114
J-303	114
PR-JIMTOWN	114
2316	115
8450	115
8687	115

Junction Name	Average Modeled Water Age (Hours)
J-889	115
315	116
3632	116
8410	116
8462	116
8552	116
8694	116
J-54	116
J-794	116
10658	117
3211	117
8764	117
228	118
J-293	118
J-51	118
J-787	118
J-793	118
1289	119
1462	119
J-197	119
J-241	119
J-487	119
J-304	120
J-491	120
J-542	120
J-713	120
10657	121
2122	121
2434	121
J-191	121
J-302	121
J-305	121
J-308	121
J-504	121
J-538	121
J-723	121
2473	122
J-19	122
J-240	122
J-503	122

Junction Name	Average Modeled Water Age (Hours)
10629	123
10634	123
8210	123
Hall Tank	123
J-214	123
J-257	123
J-4213	123
J-716	123
10633	124
10635	124
136	124
Hall No. 2	124
J-10	124
J-612	124
J-718	124
J-721	124
3165	125
4210	125
4903	125
4940	125
4994	125
J-21	125
J-497	125
J-803	125
95	126
J-189	126
J-834	126
94	127
J-323	127
J-501	127
J-83	127
225	128
3041	128
J-272	128
J-391	128
J-527	128
J-539	128
J-679	128
J- 4292	129
J-1015	129
J-255	129
J-583	129
J-589	129
524	130
J-582	130
J-588	130
J-729	130
J-581	131
J-585	131
J-587	131
J-786	131
3040	132
3667	132
J-254	132
J-807	132
4739	133
8634	133
8685	133
J-173	133
J-371	133
4778	134
544	134
8992	134
9156	134
J-584	134
J-614	134
J-677	134
2910	136
J-26	136
J-674	136
J-675	136
J-709	136
10625	137
2337	137
3660	137
4834	137
J-392	137
RV-7	137
J-134	138
J-258	138
J-295	138
J-708	138
J-830	138
J-856	138
J-859	138
3025	139
4886	139
4971	139
J-1033	139
2214	140
8626	140
8679	140
J-865	140
J-866	140
3478	141
J-827	141
110	142
J-372	142
J-502	142
J-572	142
J-869	142
10639	143
10627	144
4369	144
8536	144
J-242	144
J-321	144
J-369	144
J-871	144
J-875	144
Clays Mill Tank	145
J-46	145
J-845	145
2520	147
J-319	147
J-320	147
J-860	147
10641	148
1274	148
8581	148

Junction Name	Average Modeled Water Age (Hours)
J-316	148
J-785	148
J-876	148
CM No. 1	149
J-243	149
J-858	149
J-888	149
J- 4297	150
J- 4298	150
J- 4302	150
J-60	150
J-864	150
J-870	150
J- 4299	151
J-192	151
J-576	151
J-861	151
J-862	151
US 60 No. 1	151
US 60 No. 2	151
2130	152
3587	152
J-340	152
J-609	152
J-829	152
10640	153
J-672	153
J-873	153
1639	156
J-442	156
J-253	157
J-475	157
106	158
J- 2086	158
J-439	158
J-676	158
3474	160
J-252	160
J-311	160
J-384	160

Junction Name	Average Modeled Water Age (Hours)
1262	161
4988	161
J-250	161
J-310	161
J-383	161
J-473	161
8188	163
J-607	163
J-472	164
J-608	164
J-613	165
J-615	165
J-618	165
J-728	165
J-1032	166
J-1036	166
J-610	166
J-617	166
J-874	167
493	170
8794	170
J-393	170
J-560	170
J-868	170
J-562	171
J-877	171
J-490	173
J-604	174
J-256	175
J-836	176
5001	177
J-273	177
J-251	178
J-260	178
J-269	178
J-275	178
Parkers Mill	179
5020	180
J-558	180
J-828	180

Junction Name	Average Modeled Water Age (Hours)
J-857	180
J-390	181
J-468	182
1242	183
J-1034	184
J-394	184
J-835	184
J-389	185
J-579	185
J-853	186
J-606	188
J-261	189
J-852	190
J-246	191
J-789	191
J-279	193
J-1048	194
PM Booster	194
653	195
Hume Road	195
J-387	195
J-388	195
J-854	195
J-855	197
8523	198
J-1047	198
J-492	198
J-571	198
J-580	198
J-850	198
682	199
Hume No. 1	199
J-526	199
1220	200
J-1052	200
J-7620	200
1055	201
1098	201
1100	201
10638	202

Junction Name	Average Modeled Water Age (Hours)
J-563	202
1241	203
8457	203
703	204
J-673	204
J-76	204
J-144	205
J-81	205
J-1049	206
J-278	207
2810	209
J-264	209
J-63	209
J-75	209
J- 3048	211
J- 5787	211
J-1050	211
J-25	211
York Street	212
J-314	214
J-570	217
702	218
J-263	218
1213	220
1222	220
1240	220
2558	220
565	222
J-277	222
J-499	222
4985	223
J-566	223
712	226
8715	226
4989	228
770	228
J-85	228
1479	229
J-276	229
J-149	230

Junction Name	Average Modeled Water Age (Hours)
J-84	230
York Booster	230
1248	233
774	233
J-440	233
J-441	233
1215	234
913	234
1035	235
923	236
J-680	237
J-395	238
2715	239
10621	242
2751	242
J-301	242
657	243
641	245
J-148	245
J-336	245
J-445	245
J-461	245
35	246
36	246
Briar Hill Tank	246
J-443	247
J-459	247
J-466	247
J-396	248
J-462	250
986	251
J-68	251
1214	253
998	253
J-271	253
1014	255
PR-LEESTOWN	256
1476	257
J-78	259
746	260

Junction Name	Average Modeled Water Age (Hours)
J-457	260
J-727	260
J-67	263
1143	264
637	264
J-62	265
J-726	265
J-333	266
J-147	269
J-494	269
J-65	269
J-91	270
J-448	271
J-87	271
J-334	272
J-450	273
J-590	275
J-455	276
J-90	276
3502	277
J- 2085	277
J-486	277
2411	278
J-622	278
J-627	278
10642	279
J-454	279
J-458	279
J-603	279
J-630	279
J-631	279
642	280
719	280
J-452	280
J-626	280
J-451	281
J-483	281
J-86	281
50	282
J-460	282

Junction Name	Average Modeled Water Age (Hours)
J-464	282
J-465	282
J-621	282
J-623	282
J-632	282
J-456	283
10645	285
1469	285
J-88	285
812	286
8755	287
1466	288
J-332	289
2686	290
J-342	290
697	291
J- 2087	292
801	294
J-447	294
2458	295
J-965	297
10644	298
J-386	301
670	302
J-471	304
J-446	306
J-481	311
J-573	314
10643	316
J-142	321
J-972	321
9133	322
616	323
J-74	326
J-385	327
J-463	327
699	330
J-337	330
763	333
2684	334

Junction Name	Average Modeled Water Age (Hours)
5	335
710	335
696	336
J-432	336
6	337
J-885	337
714	340
J-341	341
2498	342
J-624	342
J-13	343
J-70	343
10646	344
185	344
J-596	345
734	351
J-484	354
J-71	357
3	358
J-635	360
10239	361
J-64	363
79	370
10620	372
625	372
J-92	375
J-437	377
620	379
3274	381
1536	382
J-434	383
9316	385
J-72	388
2634	389
J-274	389
J-476	391
184	395
629	395
J-183	398
J-9272	404

Junction Name	Average Modeled Water Age (Hours)
J-884	413
J-118	416
J-482	417
J-4883	418
J-103	419
J-477	422
J-467	426
J-496	426
80	431
82	432
2564	433
J-574	438
J-480	441
2618	442
J-575	442
J-495	443
J-145	445
J-438	448
J-478	449
J-338	452
3271	457
9423	462
3273	463
J-339	471
J-73	473
J-89	474
J-444	478
2612	486
1752	489
J-146	489
9050	496
Muddy Ford	509
J-2654	513
J-180	519
J-69	522
1837	523
J-66	523
1833	524
1884	524
1883	525

Junction Name	Average Modeled Water Age (Hours)
J-2614	527
AV-1	542
CoxG Tank	552
Mercer Booster	553
Mercer Road	553
J-1030	557
J-93	557
2777	559
2603	564
CoxG Booster	564
J-306	582
J-96	595
J-116	601
10636	602
2832	622
J-176	646
2028	655
10647	656
2060	658
2774	674
J-244	681
J-104	690
J-1031	699
J-1029	704
J-470	719
J-469	723
J-1028	739
8787	751
J-182	752
J-178	757
2226	772
J-960	1027
J-177	1195
J-174	1219
J-592	1263
KRS-12	1269
2623	1650
9429	1678
2626	1697
J- 2617	1708

Junction Name	Average Modeled Water Age (Hours)
CoxE Tank	1772
CoxE Booster	1783
J-435	2251
RCave1	2662
RCave2	2730
BH No. 2	3039
KRS-15	3128
J-717	3324
10630	3370
KRS2 HS No. 1	3796
J-720	3848
MH No. 1	3971
MH No. 2	3971
Delaplain	3977
Lee No. 1	3977
Lee No. 2	3977
MP1	3977
MP2	3977
RV-1	3977
RV-2	3977
RV-3	3977
Hall No. 1	3980
RV-4	3980
J-181	4394
RCave3	5564
RRS-10	7959
0J-725	7960
AV-3	7960
Becknerville Tank	7960
Clintonville Tank	7960
CM No. 2	7960
CMILL T2	7960
Hume No. 2	7960
Hume No. 3	7960
J-449	7960
J-453	7960
J-531	7960
J-605	7960
J-628	7960

Junction Name	Average Modeled Water Age (Hours)
J-724	7960
J-725	7960
J-887	7960
J-959	7960
J-973	7960
J-981	7960
J-982	7960
KRS-10	7960
KRS-14	7960
KRS2 HS No. 3	7960
KRS2 HS No. 4	7960
RRS-11	7960
RRS-7	7960
RUSSELL CAVE	7960
Sadieville Standpipe	7960
Woodlake No. 3	7960

APPENDIX D
FUTURE MINIMUM WATER AGE EXPORTS

Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)
c-6	0	J-634	3	5853	6
Cox St Reservoir	0	J-7281	3	5964	6
Hall Model Reservoir	0	J-7595	3	6082	6
Hume Reservoir	0	J-8515	3	6401	6
J-968	0	RRSa	3	6504	6
KRS1	0	5417	4	6626	6
KRS2 Pool 3	0	5550	4	6765	6
Mercer Reservoir	0	5588	4	c-11	6
PM Reservoir	0	5606	4	c-13	6
RRS	0	5941	4	c-14	6
York Reservoir	0	5947	4	J-373	6
c-5	1	6173	4	J-636	6
c-8	1	10605	4	J-658	6
c-9	1	10606	4	J-811	6
J-971	1	c-15	4	J-815	6
J-976	1	J-755	4	3977	7
J-978	1	J-757	4	4152	7
KRS-13	1	J-808	4	4180	7
New No. 2	1	MH No. 1	4	4356	7
New No. 3	1	MH No. 2	4	4367	7
RRS-7	1	New No. 1	4	4397	7
5939	2	4392	5	4752	7
c-1	2	4506	5	4904	7
c-19	2	5268	5	5669	7
c-20	2	5556	5	5794	7
c-4	2	5829	5	5959	7
J-137	2	6040	5	6598	7
J-981	2	6643	5	6622	7
J-983	2	6645	5	c-12	7
KRS2 HS No. 1	2	6758	5	c-21	7
5933	3	10588	5	J-124	7
5954	3	J-559	5	J-375	7
10607	3	J-816	5	J-656	7
10608	3	4108	6	J-659	7
c-10	3	4117	6	J-813	7
c-2	3	4404	6	J-967	7
c-3	3	4759	6	3682	8
J-1046	3	4813	6	3693	8
J-6126	3	5815	6	3706	8
		5851	6	3754	8

Junction Name	Average Modeled Water Age (Hours)
4111	8
4118	8
4125	8
4249	8
4527	8
4544	8
5695	8
6138	8
6331	8
6393	8
6554	8
6563	8
6576	8
6678	8
10261	8
c-16	8
J-127	8
J-128	8
J-436	8
J-660	8
J-784	8
J-814	8
J-817	8
3531	9
3550	9
3678	9
3695	9
3724	9
3735	9
3737	9
3740	9
3741	9
3903	9
3950	9
4129	9
4293	9
4780	9
5308	9
5688	9
5769	9

Junction Name	Average Modeled Water Age (Hours)
6383	9
6667	9
7057	9
9029	9
10314	9
J-129	9
J-135	9
J-548	9
J-639	9
J-640	9
J-975	9
J-984	9
KRS-11	9
KRS-15	9
RV-6	9
3552	10
3568	10
3602	10
3633	10
3641	10
3643	10
5356	10
5435	10
5495	10
5859	10
6300	10
6443	10
6454	10
9832	10
9921	10
9928	10
10061	10
J-131	10
J-138	10
J-206	10
J-208	10
J-7620	10
J-985	10
KRS-12	10
KRS-14	10

Junction Name	Average Modeled Water Age (Hours)
3566	11
3573	11
3616	11
3692	11
3851	11
3865	11
4969	11
5337	11
5494	11
5713	11
6390	11
6394	11
6706	11
6856	11
7004	11
7013	11
8124	11
8126	11
9458	11
9557	11
9595	11
9754	11
9794	11
9813	11
9834	11
10089	11
10108	11
AV-1	11
Delaplain	11
J-204	11
J-205	11
J-207	11
J-210	11
J-212	11
J-220	11
J-221	11
RV-2	11
3544	12
4812	12
4855	12

Junction Name	Average Modeled Water Age (Hours)
5178	12
5357	12
5803	12
6193	12
6236	12
6384	12
6473	12
6579	12
6708	12
7428	12
7660	12
7684	12
9338	12
9425	12
9487	12
9501	12
9655	12
9682	12
9686	12
9702	12
10103	12
10112	12
10148	12
J-130	12
J-211	12
J-224	12
J-226	12
J-780	12
J-809	12
3384	13
3393	13
3808	13
4153	13
4575	13
5923	13
6044	13
6290	13
6516	13
6818	13
7002	13
7230	13
7414	13
7420	13
7434	13
7500	13
9186	13
9245	13
9334	13
9397	13
9398	13
9439	13
9516	13
9653	13
9664	13
9678	13
9783	13
9799	13
9846	13
10217	13
10273	13
10275	13
J-1013	13
J-685	13
J-686	13
J-690	13
J-779	13
J-782	13
J-878	13
3338	14
3690	14
3696	14
3734	14
4154	14
5704	14
6091	14
6096	14
6121	14
6285	14
6348	14
6811	14
6899	14
7007	14
7079	14
7114	14
7139	14
7167	14
7169	14
7507	14
7604	14
7710	14
7743	14
8801	14
9165	14
9185	14
9222	14
9232	14
9246	14
9277	14
9562	14
9820	14
9871	14
10220	14
10577	14
10578	14
10579	14
H4347	14
J-687	14
J-692	14
3334	15
3574	15
3637	15
3662	15
3677	15
3709	15
5865	15
6014	15
6103	15
6205	15
6299	15
6607	15

Junction Name	Average Modeled Water Age (Hours)
6794	15
6873	15
6949	15
6960	15
7161	15
7484	15
7802	15
7986	15
8691	15
8714	15
8798	15
8951	15
9015	15
9022	15
9143	15
9195	15
9288	15
9291	15
9563	15
9777	15
9886	15
10100	15
10130	15
10296	15
10313	15
10562	15
H4348	15
J-379	15
J-654	15
J-663	15
J-688	15
J-698	15
3221	16
3288	16
3472	16
3607	16
3680	16
4602	16
5115	16
5667	16

Junction Name	Average Modeled Water Age (Hours)
5755	16
6361	16
6491	16
6540	16
6592	16
6929	16
7273	16
7277	16
7492	16
8026	16
8865	16
8868	16
8870	16
8921	16
8966	16
9000	16
9002	16
9171	16
9182	16
9310	16
9407	16
9623	16
9624	16
9641	16
9734	16
9773	16
9839	16
10300	16
10302	16
H368	16
H3784	16
J-1061	16
J-133	16
J-196	16
J-652	16
J-691	16
J-693	16
J-694	16
J-695	16
J-847	16

Junction Name	Average Modeled Water Age (Hours)
3050	17
3184	17
3661	17
3666	17
5083	17
5707	17
5791	17
5839	17
6281	17
6308	17
6872	17
7360	17
8516	17
8684	17
8862	17
8952	17
9034	17
10060	17
10232	17
10252	17
10346	17
J-1012	17
J-168	17
J-378	17
J-683	17
J-684	17
J-846	17
J-849	17
J-987	17
3459	18
4893	18
5892	18
5990	18
7408	18
7553	18
7645	18
7760	18
7857	18
7913	18
7939	18

Junction Name	Average Modeled Water Age (Hours)
7985	18
8486	18
8605	18
8619	18
8840	18
9105	18
9305	18
9441	18
9446	18
9479	18
9782	18
9911	18
10201	18
10340	18
10342	18
10497	18
J-1040	18
J-1060	18
J-217	18
J-376	18
J-662	18
PR-STRADER	18
2824	19
2915	19
4613	19
5074	19
5587	19
5616	19
5622	19
5826	19
6110	19
6313	19
6944	19
7046	19
7252	19
8201	19
8396	19
8425	19
8454	19
8469	19

Junction Name	Average Modeled Water Age (Hours)
8828	19
8837	19
8838	19
9292	19
10020	19
10247	19
10396	19
10398	19
10403	19
J-645	19
J-661	19
J-783	19
2854	20
3224	20
4648	20
4678	20
5215	20
5227	20
5487	20
5498	20
5501	20
5679	20
5692	20
5834	20
6673	20
7131	20
7322	20
7392	20
7520	20
7545	20
7643	20
7683	20
7691	20
7886	20
8103	20
8254	20
8327	20
8539	20
8727	20
8824	20

Junction Name	Average Modeled Water Age (Hours)
8994	20
9427	20
9459	20
10012	20
10013	20
10347	20
10350	20
CHIL	20
J-1010	20
J-1011	20
J-170	20
J-223	20
J-419	20
J-431	20
PR-CHILESBURG	20
2733	21
3372	21
4666	21
4905	21
5019	21
5223	21
5299	21
5479	21
5497	21
5551	21
5741	21
7318	21
7537	21
7562	21
7903	21
7977	21
8034	21
8214	21
8467	21
8542	21
8678	21
8720	21
8836	21
8958	21

Junction Name	Average Modeled Water Age (Hours)
9261	21
9575	21
9977	21
9982	21
10332	21
H4479	21
J-1063	21
J-153	21
J-157	21
J-161	21
J-420	21
J-704	21
2730	22
4624	22
4984	22
5831	22
6561	22
6669	22
6689	22
6702	22
6717	22
7205	22
7464	22
7467	22
7578	22
7682	22
7855	22
7877	22
7914	22
7962	22
7984	22
8110	22
8270	22
8494	22
8585	22
8587	22
8721	22
8797	22
8869	22
8987	22

Junction Name	Average Modeled Water Age (Hours)
9039	22
9454	22
9490	22
9975	22
10002	22
10504	22
J-1062	22
J-119	22
J-132	22
J-155	22
J-213	22
J-429	22
J-707	22
J-751	22
J-988	22
J-990	22
J-991	22
2592	23
3332	23
3565	23
4246	23
4840	23
4980	23
6565	23
6619	23
6684	23
6697	23
6709	23
6912	23
7268	23
7349	23
7723	23
7786	23
8014	23
8056	23
8084	23
8572	23
8606	23
8651	23
8751	23

Junction Name	Average Modeled Water Age (Hours)
10121	23
10297	23
10426	23
10509	23
J-121	23
J-150	23
J-172	23
J-27	23
J-430	23
J-643	23
J-992	23
2526	24
2883	24
4978	24
5419	24
6595	24
6863	24
7508	24
7790	24
7908	24
8033	24
8690	24
8733	24
9685	24
9856	24
9970	24
10042	24
10333	24
10464	24
H3936	24
J-1018	24
J-114	24
J-122	24
J-171	24
J-198	24
J-29	24
J-416	24
J-595	24
J-758	24
3098	25

Junction Name	Average Modeled Water Age (Hours)
3256	25
3665	25
3672	25
3701	25
4601	25
5088	25
5371	25
5844	25
5997	25
6945	25
7024	25
7367	25
8068	25
8294	25
8456	25
10022	25
10166	25
10237	25
10269	25
10278	25
10445	25
10487	25
H3821	25
J-1069	25
J-120	25
J-173	25
J-644	25
2826	26
3008	26
3668	26
4895	26
5500	26
5547	26
6749	26
6909	26
7352	26
7357	26
7369	26
7437	26
7521	26
7884	26
8387	26
8583	26
9507	26
10131	26
10180	26
10212	26
10404	26
10436	26
10488	26
H3702	26
J-1016	26
J-1081	26
J-152	26
J-162	26
J-381	26
J-417	26
J-418	26
J-705	26
J-798	26
PR-REYNOLDS	26
Tates Creek	26
3133	27
3199	27
3710	27
4611	27
4612	27
4839	27
5573	27
5583	27
5938	27
6776	27
6916	27
7047	27
7163	27
7275	27
7278	27
7409	27
7424	27
7647	27
8039	27
8049	27
8297	27
8653	27
8884	27
9406	27
9765	27
9881	27
10048	27
10083	27
10133	27
10221	27
10276	27
10414	27
J-1019	27
J-1020	27
J-174	27
J-479	27
J-678	27
2732	28
2892	28
2961	28
3053	28
3067	28
3087	28
3114	28
3119	28
3121	28
3122	28
3171	28
3183	28
3368	28
3598	28
5592	28
6672	28
6711	28
6871	28
6915	28
7140	28
7218	28

Junction Name	Average Modeled Water Age (Hours)
7220	28
7223	28
7264	28
7276	28
7286	28
7287	28
7342	28
7829	28
7848	28
7872	28
7920	28
7987	28
8144	28
8153	28
8154	28
8227	28
8588	28
8617	28
8650	28
8670	28
9913	28
9918	28
10193	28
10317	28
10328	28
10353	28
J-169	28
J-209	28
J-357	28
J-422	28
J-424	28
2258	29
2388	29
5309	29
5694	29
6587	29
6591	29
6855	29
7186	29
7361	29

Junction Name	Average Modeled Water Age (Hours)
8187	29
8437	29
8858	29
8912	29
9958	29
9986	29
10318	29
10339	29
10354	29
10406	29
J-1023	29
J-175	29
J-356	29
J-426	29
J-715	29
J-796	29
J-881	29
RV-8	29
3615	30
3640	30
5005	30
5289	30
5665	30
5763	30
5808	30
6038	30
6175	30
6655	30
6699	30
6721	30
6725	30
6938	30
7156	30
8137	30
8186	30
8322	30
8369	30
9010	30
9127	30
9505	30

Junction Name	Average Modeled Water Age (Hours)
9826	30
9983	30
10187	30
10234	30
10244	30
10266	30
10315	30
10343	30
10358	30
J-1084	30
J-115	30
J-382	30
J-423	30
J-681	30
J-804	30
J-879	30
J-882	30
J-883	30
RV-9	30
2384	31
2987	31
3236	31
3716	31
5217	31
5522	31
5552	31
6321	31
6662	31
6876	31
7809	31
7976	31
8041	31
8077	31
9035	31
9106	31
9335	31
9724	31
10050	31
10125	31
10152	31

Junction Name	Average Modeled Water Age (Hours)
10214	31
10260	31
J-1024	31
J-1025	31
J-199	31
2392	32
3158	32
4764	32
4797	32
4884	32
5063	32
5614	32
6039	32
6677	32
7101	32
7247	32
7400	32
7491	32
8007	32
8228	32
8906	32
9007	32
9823	32
10037	32
J-1042	32
J-105	32
J-158	32
J-259	32
J-350	32
J-360	32
J-415	32
Woodlake Tank No. 1	32
2801	33
3157	33
3685	33
4768	33
5687	33
6771	33
6781	33
7014	33
7481	33
9163	33
9420	33
9556	33
10017	33
J- 2317	33
J-222	33
J-427	33
J-433	33
J-752	33
J-754	33
J-802	33
J-946	33
J-993	33
2901	34
4276	34
4562	34
4798	34
5166	34
5714	34
6666	34
7327	34
9103	34
9887	34
J-1026	34
J-166	34
J-425	34
J-799	34
J-805	34
MARIA	34
5643	35
6509	35
6613	35
6777	35
7416	35
7461	35
7517	35
9432	35
9735	35
H2339	35
J-753	35
J-957	35
J-961	35
POOL3 NODE	35
2162	36
2177	36
2769	36
2820	36
4183	36
4456	36
4946	36
6231	36
6310	36
6327	36
6343	36
6541	36
6963	36
7211	36
9110	36
9130	36
9944	36
10216	36
10306	36
J-1005	36
J-1022	36
J-1027	36
J-941	36
J-942	36
J-943	36
1922	37
4485	37
4589	37
4642	37
6076	37
6168	37
6975	37
7005	37
7359	37
9191	37

Junction Name	Average Modeled Water Age (Hours)
9720	37
9732	37
10459	37
H2338	37
H4040	37
J-1	37
J-1057	37
J-597	37
J-840	37
J-938	37
J-939	37
J-940	37
4372	38
4452	38
5647	38
7091	38
7118	38
9126	38
9619	38
9728	38
H3242	38
H4041	38
J-1044	38
J-421	38
J-936	38
J-937	38
Woodlake No. 2	38
4303	39
5285	39
5376	39
5648	39
6025	39
6171	39
9716	39
J-353	39
J-933	39
J-934	39
J-935	39
3430	40
4498	40

Junction Name	Average Modeled Water Age (Hours)
4850	40
5529	40
5627	40
5654	40
5660	40
5664	40
5863	40
6143	40
6438	40
6477	40
7092	40
7456	40
9257	40
9626	40
J-598	40
J-930	40
J-932	40
5034	41
5396	41
5545	41
5686	41
5782	41
5790	41
5804	41
5849	41
5946	41
5960	41
6215	41
6379	41
6691	41
6859	41
6865	41
7074	41
9715	41
J-151	41
J-927	41
J-928	41
J-929	41
J-963	41
RV-13	41

Junction Name	Average Modeled Water Age (Hours)
Woodlake No. 1	41
2012	42
2939	42
5132	42
6868	42
J-1053	42
J-364	42
J-701	42
J-924	42
J-925	42
J-926	42
2888	43
2985	43
2993	43
5075	43
5445	43
5481	43
5858	43
6081	43
6457	43
6797	43
7340	43
9625	43
J-1064	43
J-354	43
J-428	43
J-920	43
J-922	43
J-994	43
2741	44
5441	44
J-2	44
J-352	44
J-918	44
J-919	44
J-921	44
2287	45
2291	45
2829	45
2931	45

Junction Name	Average Modeled Water Age (Hours)
5464	45
9050	45
J-1045	45
J-1065	45
J-916	45
J-917	45
3089	46
4330	46
6816	46
8794	46
9412	46
J-1080	46
J-351	46
J-405	46
J-912	46
J-914	46
J-915	46
2426	47
5362	47
5403	47
5904	47
5924	47
6812	47
7094	47
7510	47
8216	47
J-1043	47
J-159	47
J-407	47
J-911	47
2181	48
2695	48
4036	48
4762	48
5008	48
6163	48
7646	48
7701	48
7942	48
8591	48

Junction Name	Average Modeled Water Age (Hours)
9616	48
10066	48
J-102	48
J-909	48
J-910	48
1937	49
2013	49
4374	49
5331	49
5927	49
6668	49
7044	49
7666	49
7720	49
9355	49
10455	49
J-907	49
J-908	49
2602	50
3086	50
5323	50
5993	50
6117	50
7797	50
7854	50
8100	50
9133	50
9207	50
9229	50
J-180	50
J-409	50
J-906	50
2059	51
2125	51
2132	51
2929	51
2941	51
4223	51
6906	51
7672	51

Junction Name	Average Modeled Water Age (Hours)
8333	51
8545	51
8744	51
9156	51
J-1004	51
J-176	51
J-362	51
J-411	51
J-905	51
1512	52
1667	52
1800	52
1886	52
2134	52
2361	52
2376	52
5281	52
5966	52
8028	52
8679	52
8685	52
8969	52
9295	52
9311	52
J-1079	52
J-163	52
J-177	52
J-370	52
J-771	52
J-904	52
2062	53
5264	53
5341	53
6835	53
6914	53
8382	53
8430	53
8476	53
8523	53
8626	53

Junction Name	Average Modeled Water Age (Hours)
8755	53
J-902	53
J-903	53
1506	54
2156	54
2909	54
5359	54
6399	54
7959	54
8202	54
8581	54
8973	54
9316	54
9400	54
J-367	54
J-368	54
J-412	54
J-901	54
1647	55
2058	55
2137	55
2245	55
4901	55
5415	55
6940	55
8536	55
8712	55
J-1002	55
J-1003	55
J-897	55
J-899	55
J-900	55
2178	56
2212	56
4290	56
5172	56
5255	56
5719	56
5835	56
6903	56

Junction Name	Average Modeled Water Age (Hours)
6935	56
7037	56
8210	56
J-1041	56
J-343	56
J-896	56
1585	57
4339	57
4631	57
7192	57
8087	57
8218	57
J-1001	57
J-410	57
J-895	57
J-999	57
1906	58
2061	58
2983	58
4267	58
4317	58
6639	58
6741	58
6989	58
8457	58
8590	58
J-160	58
J-894	58
1492	59
2235	59
5057	59
5104	59
6824	59
7429	59
7579	59
8638	59
J-406	59
J-891	59
J-944	59
1482	60

Junction Name	Average Modeled Water Age (Hours)
1689	60
1702	60
2014	60
2104	60
2265	60
2299	60
6534	60
6866	60
7071	60
8750	60
8804	60
8859	60
9020	60
9073	60
9639	60
H2768	60
J-1074	60
J-408	60
1468	61
1552	61
1832	61
2382	61
2383	61
4122	61
4326	61
4989	61
5001	61
5130	61
5145	61
6762	61
6763	61
8384	61
9423	61
J-1075	61
J-156	61
J-363	61
J-750	61
J-948	61
J-949	61
2213	62

Junction Name	Average Modeled Water Age (Hours)
2485	62
4192	62
4250	62
4411	62
9564	62
J-950	62
J-958	62
1371	63
1416	63
1681	63
1823	63
2091	63
2330	63
2386	63
4985	63
6389	63
9608	63
J- 4305	63
J-594	63
J-893	63
J-951	63
1413	64
4263	64
5107	64
5235	64
6937	64
7059	64
7243	64
7332	64
J- 5945	64
J-1009	64
J-349	64
J-366	64
J-749	64
J-952	64
1325	65
1424	65
1457	65
2308	65
8724	65
J-792	65
J-954	65
J-964	65
RV-10	65
1775	66
2278	66
4101	66
5011	66
5106	66
7466	66
J-591	66
J-600	66
J-955	66
J-980	66
2044	67
2296	67
2344	67
4390	67
7165	67
7236	67
10263	67
J-345	67
J-348	67
J-365	67
J-956	67
J-969	67
500	68
1517	68
1555	68
1765	68
2338	68
5476	68
5591	68
J-1056	68
J-1087	68
J-402	68
J-778	68
J-947	68
J-974	68
J-99	68
498	69
505	69
1553	69
2374	69
6474	69
7258	69
8072	69
9442	69
9467	69
J-346	69
J-358	69
J-532	69
J-733	69
J-945	69
488	70
506	70
2233	70
5068	70
10281	70
10659	70
J-123	70
J-725	70
J-735	70
J-790	70
2335	71
4037	71
7109	71
7246	71
9339	71
9416	71
10660	71
EASTLAND	71
J-359	71
J-398	71
J-736	71
J-776	71
593	72
5032	72
7532	72
7875	72

Junction Name	Average Modeled Water Age (Hours)
9290	72
9429	72
9451	72
J-966	72
5047	73
9225	73
10618	73
J-1038	73
J-22	73
J-24	73
J-384	73
J-599	73
355	74
671	74
1910	74
4035	74
7070	74
8004	74
9107	74
9343	74
10658	74
10661	74
BH No. 1	74
J-380	74
J-413	74
J-414	74
J-740	74
J-742	74
309	75
540	75
541	75
806	75
7242	75
7300	75
10657	75
J-139	75
J-385	75
J-775	75
J-842	75
282	76

Junction Name	Average Modeled Water Age (Hours)
312	76
317	76
851	76
898	76
1723	76
7331	76
9025	76
10429	76
J-23	76
J-739	76
931	77
962	77
1567	77
4235	77
5026	77
7115	77
7597	77
J-1037	77
J-1051	77
J-280	77
J-286	77
J-294	77
J-361	77
J-507	77
J-511	77
J-513	77
J-533	77
J-738	77
J-743	77
J-996	77
377	78
1106	78
1172	78
1678	78
4767	78
7097	78
7705	78
8378	78
8896	78
9094	78

Junction Name	Average Modeled Water Age (Hours)
9135	78
J-100	78
J-18	78
J-266	78
J-281	78
J-514	78
J-515	78
J-633	78
J-638	78
J-732	78
RV-5	78
474	79
1184	79
1187	79
1209	79
1472	79
2000	79
3226	79
3645	79
7924	79
7926	79
8759	79
10149	79
J-403	79
J-516	79
J-517	79
J-518	79
J-791	79
PR-MARIA	79
203	80
215	80
216	80
374	80
1163	80
2159	80
2192	80
7245	80
7248	80
7863	80
7995	80

Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)
H2730	80	9279	83	8167	87
J-1076	80	J-1077	83	9218	87
J-126	80	J-1085	83	9296	87
J-229	80	J-262	83	H3115	87
J-737	80	J-28	83	J-1006	87
J-795	80	J-519	83	J-325	87
200	81	J-602	83	J-509	87
202	81	748	84	J-647	87
831	81	3383	84	J-648	87
1132	81	7430	84	837	88
1144	81	7468	84	1504	88
2066	81	J-11	84	J-125	88
2115	81	J-522	84	J-201	88
2238	81	J-555	84	J-529	88
3848	81	J-601	84	J-530	88
7814	81	J-649	84	J-534	88
9082	81	J-650	84	J-747	88
9118	81	J-769	84	J-766	88
9303	81	J-970	84	219	89
J-101	81	179	85	838	89
J-531	81	822	85	5062	89
193	82	1244	85	6778	89
318	82	4843	85	7556	89
665	82	10662	85	7572	89
7820	82	J-30	85	9167	89
7927	82	J-557	85	J-97	89
9230	82	J-767	85	1303	90
10619	82	J-768	85	2665	90
J-20	82	788	86	2676	90
J-290	82	7290	86	7323	90
J-520	82	9155	86	7554	90
J-730	82	9404	86	7727	90
J-98	82	J-32	86	J-117	90
218	83	J-330	86	J-292	90
313	83	J-34	86	J-31	90
401	83	J-553	86	J-508	90
404	83	J-554	86	756	91
1455	83	J-646	86	1227	91
7341	83	J-731	86	1229	91
7356	83	7856	87	1305	91

Junction Name	Average Modeled Water Age (Hours)
1306	91
2927	91
3242	91
7807	91
8386	91
8812	91
J-284	91
J-344	91
J-745	91
J-760	91
165	92
797	92
1156	92
1165	92
1221	92
1223	92
1302	92
7509	92
7596	92
9138	92
AV-4	92
J-512	92
J-773	92
1261	93
4903	93
7535	93
J-285	93
1121	94
7628	94
9037	94
9074	94
J-324	94
J-33	94
J-37	94
J-38	94
J-40	94
J-772	94
800	95
1171	95
1195	95

Junction Name	Average Modeled Water Age (Hours)
2719	95
AV-2	95
J-326	95
J-41	95
164	96
167	96
7784	96
9008	96
9192	96
10654	96
J-234	96
J-331	96
J-39	96
J-823	96
J-839	96
4139	97
4994	97
8121	97
9123	97
9141	97
J-232	97
J-528	97
143	98
820	98
7626	98
7764	98
J-227	98
J-230	98
J-265	98
J-283	98
J-43	98
J-762	98
141	99
689	99
1370	99
2355	99
4065	99
7544	99
10656	99
J-231	99

Junction Name	Average Modeled Water Age (Hours)
J-327	99
J-744	99
140	100
1021	100
1059	100
6017	100
8030	100
8778	100
J-329	100
J-759	100
RV-15	100
700	101
1039	101
1090	101
J-195	101
J-268	101
J-44	101
J-45	101
J-556	101
J-953	101
979	102
1089	102
1105	102
1983	102
2714	102
3487	102
7891	102
8168	102
J-48	102
J-803	102
1939	103
3063	103
9187	103
J-291	103
J-306	103
J-832	103
764	104
1000	104
2841	104
7970	104

Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)
J-194	104	J-761	108	J-293	114
J-510	104	722	109	1545	115
J-765	104	2202	109	2150	115
777	105	3044	109	8132	115
781	105	8114	109	8749	115
2204	105	J-527	109	J-49	115
7758	105	J-547	109	1153	116
8013	105	1007	110	3130	116
J-328	105	4123	110	8050	116
J-485	105	J-154	110	J-57	116
J-489	105	315	111	J-711	116
J-523	105	872	111	PR-TOYOTA	116
J-821	105	886	111	1133	117
J-825	105	1112	111	J-235	117
1017	106	1346	111	1160	118
1082	106	9086	111	8574	118
3107	106	J-186	111	J-239	118
8044	106	J-187	111	J-307	118
8105	106	J-52	111	8133	119
J-236	106	J-848	111	8344	119
J-347	106	863	112	H3958	119
J-47	106	1086	112	J-303	119
J-53	106	1211	112	J-826	119
J-818	106	8601	112	J-118	120
J-824	106	J-397	112	J-238	120
4164	107	J-487	112	J-55	120
7752	107	J-491	112	1260	121
7868	107	J-819	112	2437	121
7972	107	880	113	5020	121
J-400	107	1180	113	7991	121
J-525	107	4144	113	8236	121
J-889	107	8241	113	8352	121
810	108	H3959	113	Hall Tank	121
833	108	RV-12	113	J-1073	121
989	108	2100	114	J-214	121
3083	108	2466	114	J-58	121
4940	108	3109	114	J-834	121
J-237	108	4300	114	8462	122
J-287	108	8080	114	8756	122
J-521	108	J-245	114	8784	122

Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)
10634	122	4739	128	J-862	132
10635	122	4971	128	J-997	132
10652	122	8408	128	3660	133
Hall No. 2	122	J-844	128	10632	133
J- 4309	122	4886	129	0J-725	133
J-233	122	J-1008	129	J-109	133
J-42	122	J-241	129	J-295	133
PR-JIMTOWN	122	J-860	129	J-51	133
136	123	J-876	129	J-550	133
2394	123	RV-16	129	J-858	133
3155	123	288	130	J-869	133
4007	123	289	130	4093	134
8362	123	1219	130	10653	134
10633	123	2381	130	J-189	134
J-567	123	3667	130	J-26	134
J-569	123	4778	130	J-545	134
J-577	123	4988	130	J-861	134
8410	124	10631	130	J-865	134
J-188	124	J-197	130	J-866	134
J-247	124	J-536	130	J-874	134
8166	125	J-543	130	2349	135
8450	125	J-578	130	4006	135
8658	125	J-864	130	J- 4292	135
8680	125	J-873	130	J-134	135
J-1066	125	J-875	130	J-677	135
J-190	125	1152	131	J-856	135
94	126	2473	131	J-859	135
95	126	4210	131	890	136
8498	126	8634	131	J-614	136
8552	126	J-107	131	3165	137
8687	126	J-191	131	J-612	137
J-1067	126	J-240	131	J-619	137
2122	127	J-807	131	J-675	137
2956	127	J-868	131	J-258	138
3172	127	J-870	131	J-568	138
8694	127	J-871	131	J-59	138
8764	127	J-877	131	J-674	138
J-323	127	3176	132	J-709	138
J-679	127	4834	132	RV-7	138
1264	128	J-399	132	J-270	139

Junction Name	Average Modeled Water Age (Hours)
110	140
1193	140
3025	140
J-1068	140
J-551	140
J-561	140
J-708	140
J-827	140
2910	141
J-111	141
J-113	141
J-8	141
4105	142
J-16	142
J-549	142
J-564	142
York Street	142
3041	143
4369	143
J-4213	143
J-112	144
J-56	144
J-61	144
1989	145
2316	145
3040	145
J-321	145
2434	146
8992	146
J-319	146
J-320	146
J-54	146
J-60	146
10655	147
J-316	147
741	148
10629	148
J-830	148
706	149
1294	149

Junction Name	Average Modeled Water Age (Hours)
2873	149
J-12	149
J-785	149
867	150
2823	150
J-793	150
1288	152
2893	152
J-829	153
RV-11	153
2337	154
4076	154
J-178	154
J-46	154
J-546	154
J-77	154
4043	155
J-83	155
J-841	155
York Booster	155
J-15	156
J-552	156
106	157
J-17	157
J-490	157
843	158
892	158
1639	158
J-192	158
J-669	158
J-676	158
J-21	159
J-242	159
J-335	159
J-794	159
1462	160
J-272	160
J-713	161
2077	162
J-582	162

Junction Name	Average Modeled Water Age (Hours)
J-243	163
Clays Mill 2	164
J-723	164
RV-14	164
J-267	165
J-308	165
J-7	165
10639	166
J-888	166
CM No. 1	167
J-442	167
J-537	167
J-542	167
10625	168
J-1072	168
J-475	168
228	169
J-716	169
J-718	169
J-721	169
J-4	170
611	171
10641	171
225	172
2764	172
J-535	172
J-14	173
J-572	173
10640	174
J-539	175
586	176
1289	176
J-260	176
1295	177
J-305	177
J-787	178
J-497	179
J-501	180
1293	183
3946	183

Junction Name	Average Modeled Water Age (Hours)
J-257	183
J-439	183
J-492	183
J-828	183
J-391	185
J-392	185
J-261	187
J-538	187
J-472	188
J-322	189
J-473	189
J-584	190
J-585	190
J-672	190
Parkers Mill	190
J-254	191
J-581	191
J-587	191
J-786	191
2759	192
J-255	192
J-583	192
J-588	192
J-589	192
J-1048	193
J-304	193
J-845	195
J-1047	196
1274	197
J-576	198
J-1052	199
RUSSELL CAVE	199
J-19	200
10627	201
3056	202
2821	203
J- 2086	203
J-264	203
PM Booster	203
J-1049	205
J-724	207
3162	208
J-25	208
RCave1	208
J-1050	209
J-269	209
Hume Road	210
J-275	210
3052	211
J-273	211
2757	212
2813	215
8715	215
Hume No. 1	215
J-253	215
J-570	215
J-729	215
J-263	216
J-995	217
524	219
J-393	220
1262	225
544	226
J-279	227
J-615	228
J-617	228
J-618	228
J-63	228
J-394	229
J-613	229
J-571	232
3588	233
J-108	234
J-789	234
2520	235
2130	236
J-680	236
2214	237
2809	237
J-278	237
J-673	238
J-389	241
2822	242
J-1015	243
J-1070	243
J-314	243
J-390	245
J-340	247
3498	248
J-853	250
J-85	251
J-855	251
1241	252
2994	252
J-277	253
493	254
2962	254
1220	255
1242	256
3831	256
J-369	256
J-526	256
J-854	256
1240	263
J-276	264
1098	266
1222	266
J-728	266
36	267
1100	268
J-852	268
J-850	269
J-562	270
1055	271
1213	271
J-181	272
J-395	273
J-560	273
J- 3048	274
J- 5787	274

Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)	Junction Name	Average Modeled Water Age (Hours)
J-388	275	2411	316	J-457	365
J-1058	276	3939	316	J-75	365
J-372	276	J-78	316	1469	366
35	277	637	317	J-148	366
1248	277	J-252	318	3781	367
2960	277	J-371	318	2498	368
J-271	277	J-563	325	J-251	368
J-396	277	J-144	326	986	369
J-558	277	J-311	327	1014	369
3402	278	1214	335	998	370
J-387	278	697	336	J-466	370
641	279	2458	337	J-450	371
2558	280	J-566	337	J-459	371
J-76	280	8188	338	913	372
1215	281	J-607	339	J-445	372
J-81	281	565	340	923	373
J-836	281	J-596	342	J-443	373
J-84	282	670	345	J-451	374
J-579	286	1035	346	J-461	374
770	288	J-386	347	J-465	374
1479	291	J-608	348	J-142	375
703	292	2810	349	J-448	375
J-336	293	J-502	350	PR-LEESTOWN	375
620	294	1476	351	710	377
702	294	J-256	353	629	378
774	294	J-145	354	J-447	378
10621	294	719	355	J-455	378
682	298	J-149	355	616	381
712	298	J-310	355	J-454	381
J-182	298	J-250	356	J-458	381
J-835	298	3031	357	J-622	381
J-383	300	J-13	357	696	382
J-857	301	10239	358	J-334	382
Briar Hill Tank	304	J-440	361	J-452	382
657	306	J-441	361	J-627	382
653	307	J-606	363	J-727	382
642	308	J-65	363	1143	383
3473	308	J-9	363	J-456	383
3060	310	J-462	364	J-68	384
3937	313	J-494	364	812	385

Junction Name	Average Modeled Water Age (Hours)
3774	385
J-631	385
699	386
1466	386
J-630	386
801	387
J-62	387
J-726	387
J-965	387
J-972	387
714	388
J-446	388
J-460	388
J-464	388
J-603	388
J-67	388
J-88	389
734	390
J-621	390
746	391
763	391
J-86	391
J-147	392
J-623	392
J-626	392
J-632	392
10646	396
J-90	396
3211	397
J-468	397
3447	399
J-10	402
J-91	404
J-300	406
J-87	406
3595	407
1884	408
J-503	409
3632	410
J-333	414

Junction Name	Average Modeled Water Age (Hours)
J-1033	417
625	418
3478	420
J-463	420
J-74	420
J-274	421
3587	424
J- 2085	429
J-481	429
10642	430
J-666	432
10645	433
J- 2087	433
J-667	433
82	436
J-64	436
J-668	436
6	438
J-248	438
J-720	439
80	441
10643	443
J-342	443
10644	444
J-651	444
J-624	445
J-70	445
5	446
J-249	446
J-432	446
J-71	447
J-1036	451
J-655	453
J-664	453
J-580	454
J-504	457
3	461
2564	462
J-635	462
J-332	464

Junction Name	Average Modeled Water Age (Hours)
J-246	465
J-72	468
J-116	480
79	484
J-341	484
3474	486
J-296	489
2715	495
J-885	498
J-604	499
3502	501
J-299	501
50	502
J-297	506
J-337	510
J-302	513
J-434	513
J-467	517
2686	522
J-590	527
1536	529
J-73	530
J-1032	538
J-338	539
RV-1	539
J-482	540
J-66	544
J-1030	545
J-499	547
Mercer Road	548
J-884	549
Mercer Booster	549
2684	551
3274	554
J-592	561
3271	562
3273	563
J-69	564
J-483	567
J-484	568

Junction Name	Average Modeled Water Age (Hours)
J-1034	575
10647	576
J-298	580
J-103	587
2777	588
J-92	590
2603	591
J-717	592
J-1083	596
2634	597
J-437	597
J-486	597
2751	598
2612	604
J-339	605
185	608
2618	608
J-301	608
2832	609
1752	625
J-1031	629
2774	631
J-573	634
J-575	635
J-89	636
J-9272	636
J-574	637
J-478	639
184	641
J-1071	641
J-476	642
J-496	642
J-495	644
10620	651
J-104	651
J- 4298	660
J-480	665
RV-3	666
8787	667
J-477	669

Junction Name	Average Modeled Water Age (Hours)
2060	670
J-1028	672
J- 4297	674
J- 4299	675
J-4883	679
J-146	680
J- 4302	682
J-244	685
US 60 No. 2	686
J-438	687
US 60 No. 1	687
2626	693
1833	696
J-183	700
J-1029	701
1837	706
1883	709
J-96	712
2028	715
J-93	720
J-444	727
2226	729
J-609	736
J-2614	779
Muddy Ford	821
J-469	831
J-470	832
J-471	911
J-610	948
CoxE Booster	1088
J- 2617	1090
CoxE Tank	1092
Hall No. 1	1127
J-960	1447
CoxG Booster	1595
Cox Ground	1601
RCave2	2273
J-435	2459
10630	2592
BH No. 2	2660

Junction Name	Average Modeled Water Age (Hours)
10638	2688
MP1	2978
MP2	2978
RCave3	2984
J-2654	2995
2623	3086
RV-4	3200
Lee No. 1	3432
Lee No. 2	3432
J-628	3921
10636	5495
RRS-9	5764
RRS-6	5885
AV-3	5961
Becknerville Tank	5961
Clintonville Tank	5961
CM No. 2	5961
CMILL T2	5961
Hume No. 2	5961
Hume No. 3	5961
J-1054	5961
J-1078	5961
J-449	5961
J-453	5961
J-605	5961
J-887	5961
J-959	5961
J-973	5961
J-977	5961
J-982	5961
KRS-10	5961
KRS2 HS No. 2	5961
KRS2 HS No. 3	5961
KRS2 HS No. 4	5961
RRS-10	5961
RRS-11	5961
RRS-8	5961
Sadieville Standpipe	5961
Woodlake No. 3	5961

APPENDIX E
NORTHERN DIVISION FACILITY OPERATIONS
(PROVIDED ON CD)

APPENDIX F
NORTHERN DIVISION BASELINE PRESSURE AND WATER AGE PLOTS

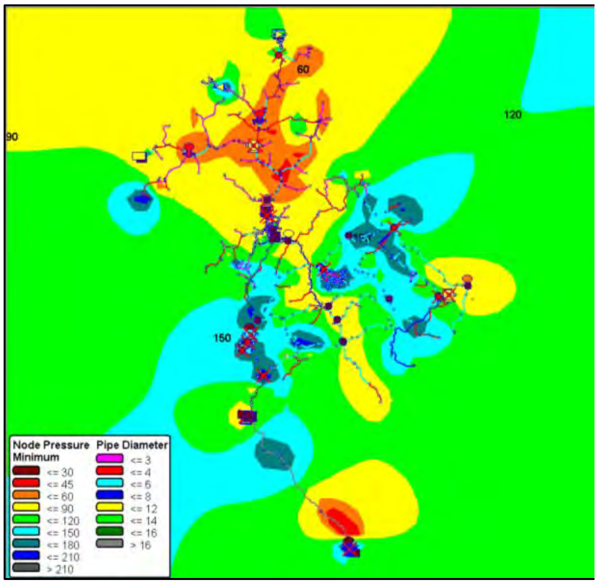


Figure 1 **Current Minimum Day Demand Minimum System Pressure**

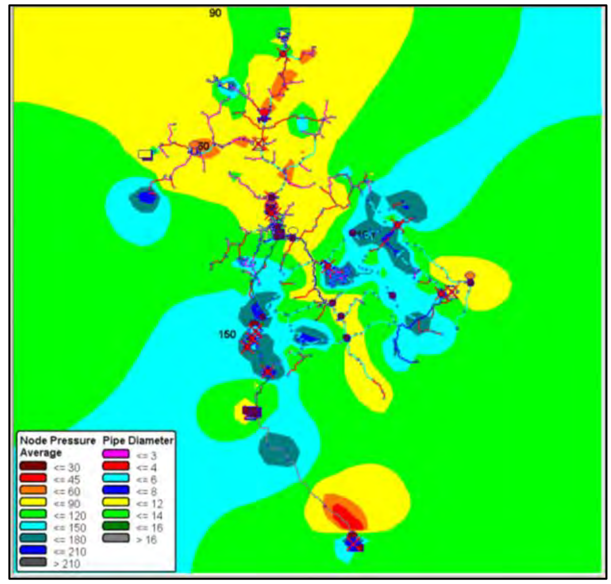


Figure 2 **Current Minimum Day Demand Average System Pressure**

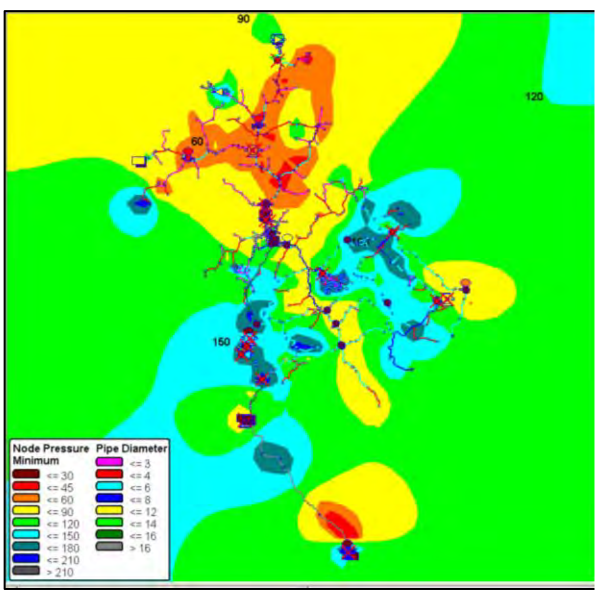


Figure 3 **Future Minimum Day Demand Minimum System Pressure**

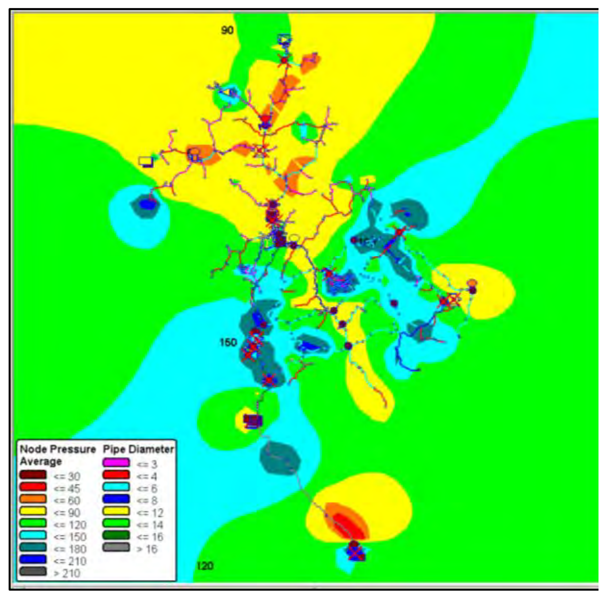


Figure 4 **Future Minimum Day Demand Average System Pressure**

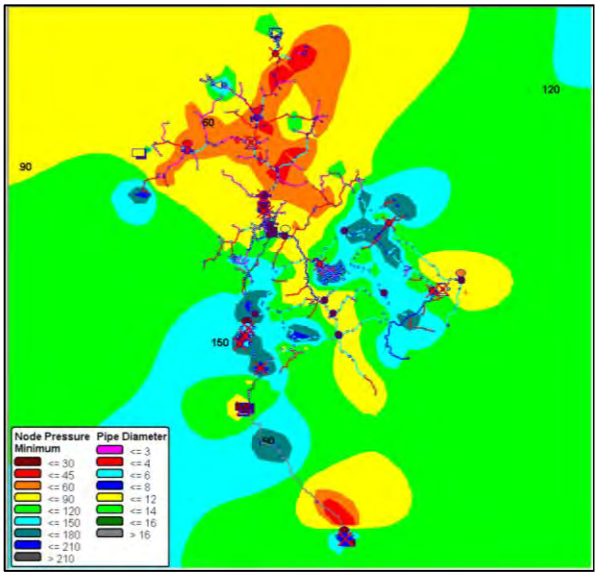


Figure 5 **Current Average Day Demand Minimum System Pressure**

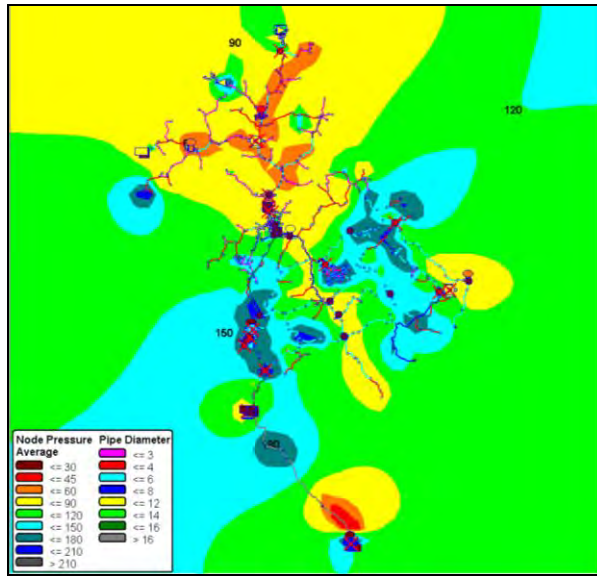


Figure 6 **Current Average Day Demand Average System Pressure**

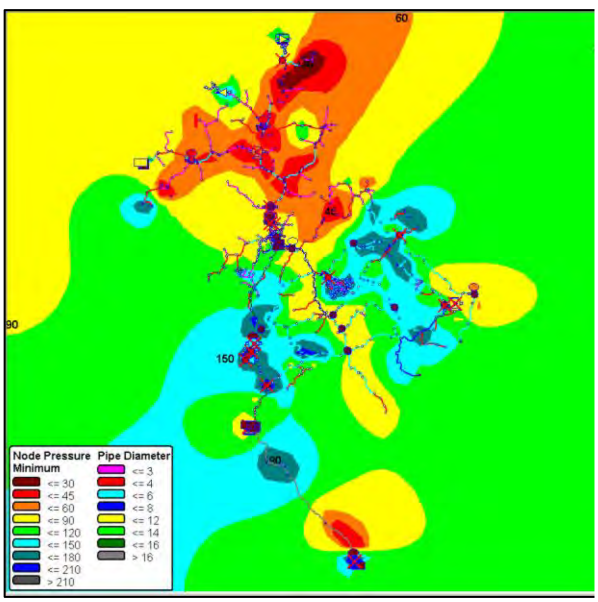


Figure 7 **Future Average Demand Day Minimum System Pressure**

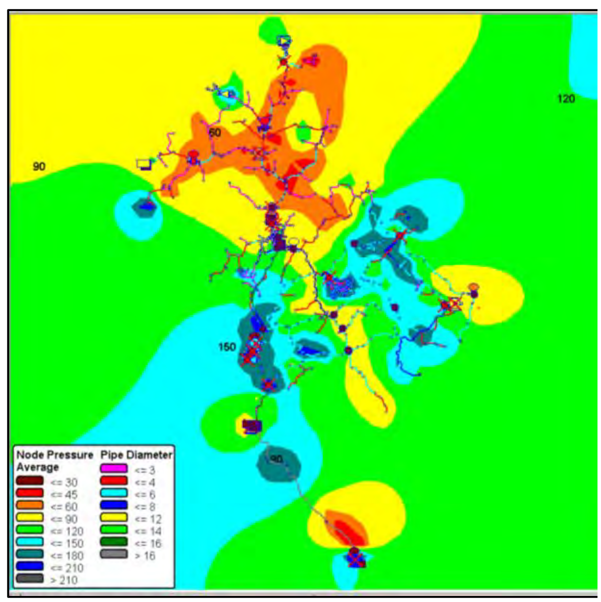


Figure 8 **Future Average Demand Day Average System Pressure**

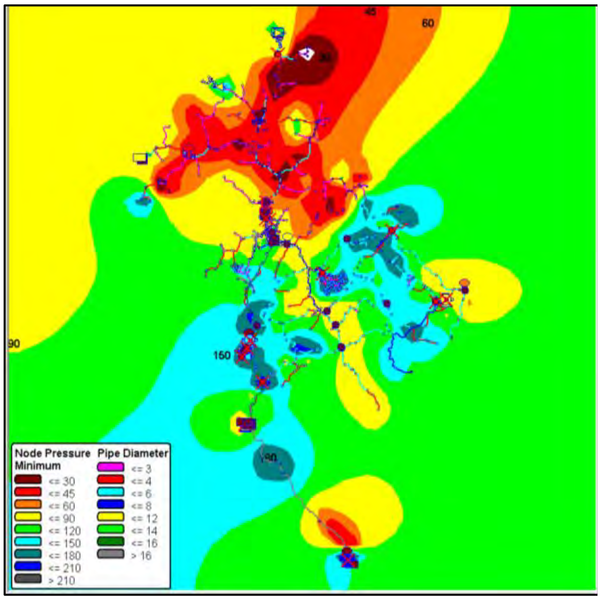


Figure 9 **Current Maximum Day Demand Minimum System Pressure**

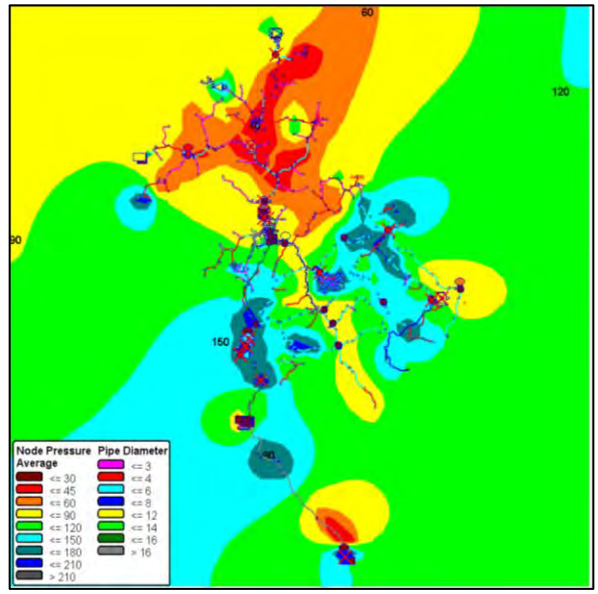


Figure 10 **Current Maximum Day Demand Average System Pressure**

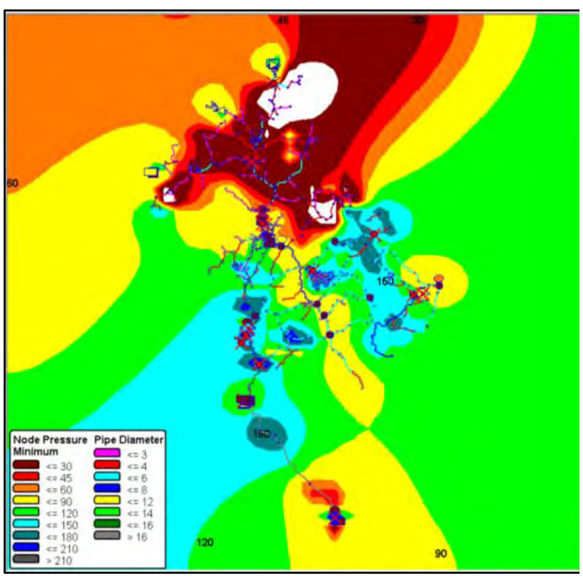


Figure 11 **Future Maximum Day Demand Minimum System Pressure**

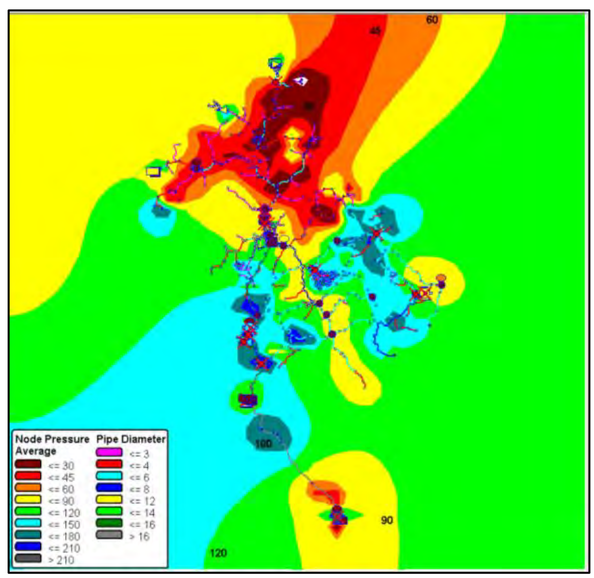


Figure 12 **Future Maximum Day Demand Average System Pressure**

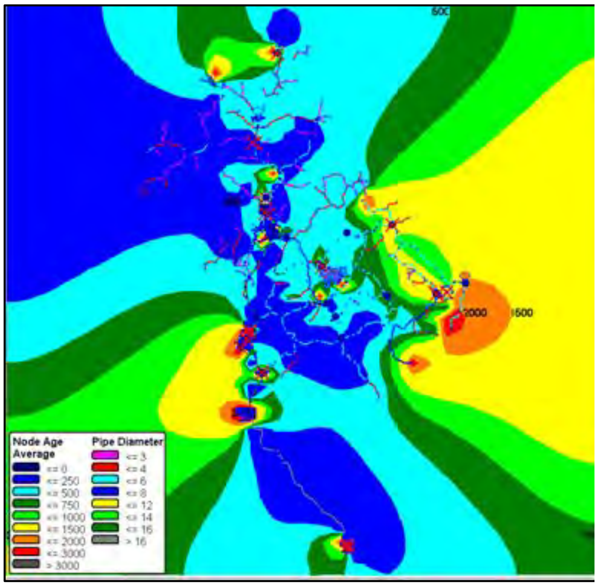


Figure 13 **Current Minimum Demand Day Average System Water Age**

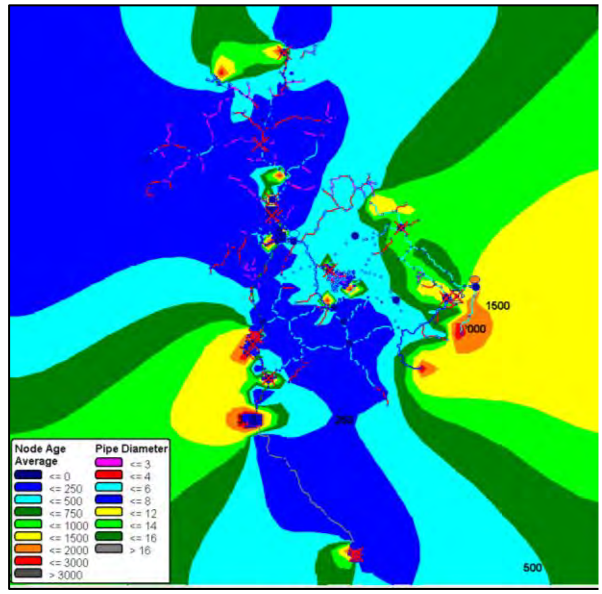


Figure 14 **Future Minimum Demand Day Average System Water Age**

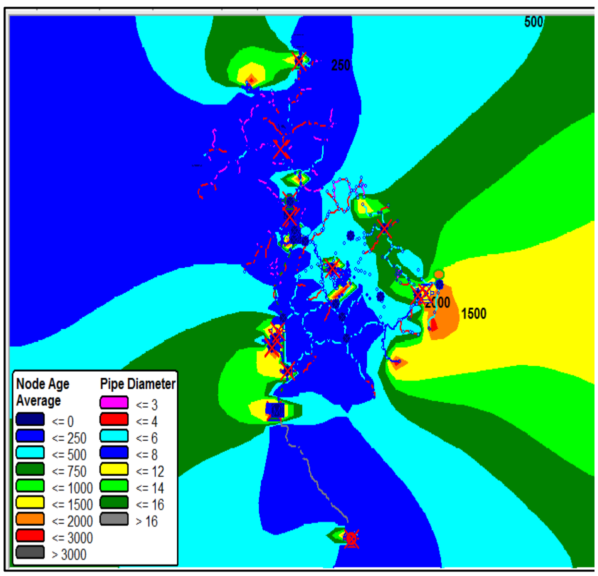


Figure 15 **Current Average Demand Day Average System Water Age**

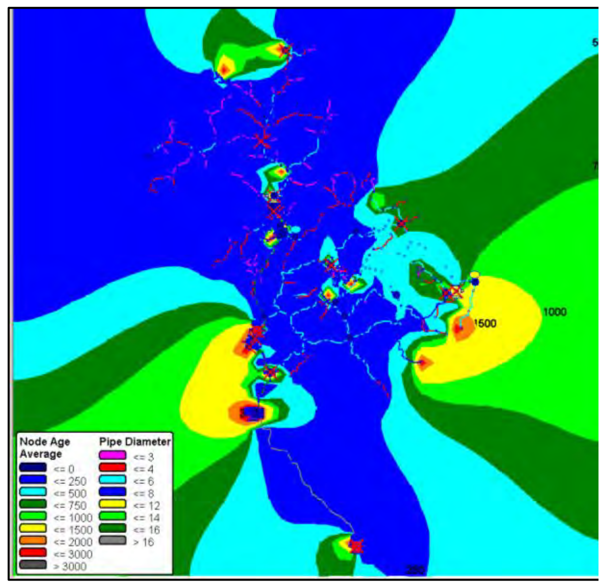


Figure 16 **Future Average Demand Day Average System Water Age**

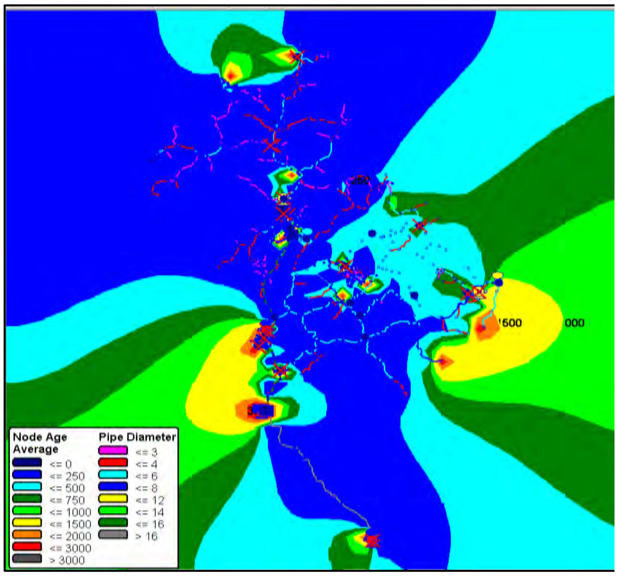


Figure 17 **Current Maximum Demand Day Average System Water Age**

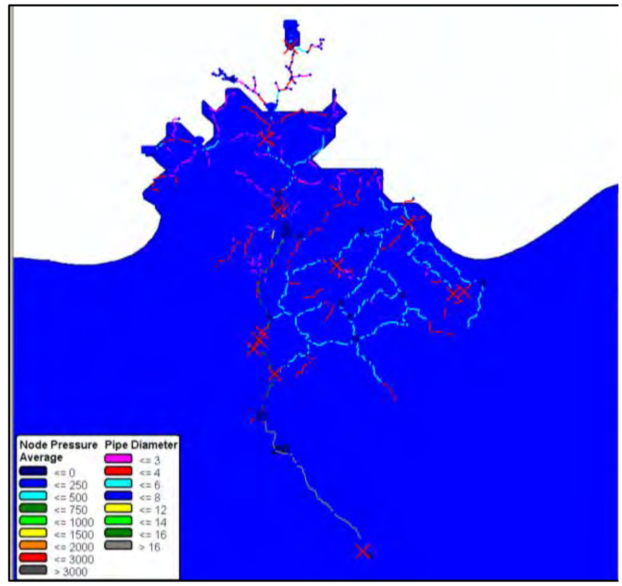


Figure 18 **Future Maximum Demand Day Average System Water Age**