Chapter 1 – Introduction

1.1 Purpose of Study

Princeton Water and Wastewater (PW&W), located in Caldwell County in western Kentucky, is the water service provider for the City of Princeton. They also provide wholesale water to Caldwell County Water District and Lyon County Water District.

Hethcoat and Davis, Inc. (H&D) prepared a hydraulic model of the existing water system for Princeton Water and Wastewater. In conjunction with the hydraulic model, H&D identified deficiencies within the system and has made recommendations to overcome them.

The model's purpose is to replicate, as close as practical, the tendencies of the existing water system. A water system is dynamic throughout the day, as pressures and demands are in continuous change. However, patterns do develop and can be mimicked. A good water model can simulate, fairly accurately, the pressure and demand patterns of the water system. It allows improvements to be tried before implementing. If the model accurately reflects the patterns and tendencies in the existing water system, water system officials can be confident that future changes/modifications/additions to the model will reflect the same results in the system.

A hydraulic model can help properly control development by cost effectively identifying appropriate infrastructure improvements necessary to adequately provide service.

This plan is a direct result of the hydraulic water model. The model identified existing system deficiencies. Proposed improvements were built into the model and the resulting benefits were assessed. Recommendations and priorities were established by ranking the benefits of each tried improvement.

1.2 Scope of Study

Within the Princeton water system, the following items were analyzed and prepared to determine system deficiencies and align them with corresponding recommended improvements.

- Computer model of the existing water distribution system
- Assessment of the existing water distribution system operation and infrastructure
- Computer model scenarios to simulate the water system's response to normal water usage distribution
- Existing System under average demands used for model calibration
- Future System under average demands after each priority is implemented
- Model runs to simulate extreme water usage events, i.e. flushing, at strategic locations to determine the system's response to stressed conditions, which include:
  - -Existing System under flushing conditions

        -Future System under flushing conditions after each priority is implemented

- Problem identification for the existing system
- Recommendations to maximize system performance
- Cost estimate of recommended improvements
- Final report

Design Guidance is provided by the Kentucky Division of Water. Any portion of the water system that cannot satisfy the following minimum standards is considered underserved.

- A minimal pressure of 30 psi must be available on the discharge side of all meters.
- Water lines must be able to be flushed at a minimum of 2.5 fps while maintaining 20 psi throughout the system.
- Calculate whether 30 psi can be maintained at the discharge side of the meter under the peak domestic demand using the formula: 10 gpm x (No. of Customers)$^{1/2}$.

Chapter 2 – Existing Water Distribution System

2.1 General

The Princeton water system consists of a 3.0 MGD Water Treatment Plant with two high service pumps, three water storage tanks, one booster pump station and a distribution network serving 2,900 customers. The system has connections to Caldwell County Water District and Lyon County Water District to supply wholesale water. Figures 1 and 2 (end of Chapter 2) display the existing distribution system. This chapter provides detailed information on each of the key components as well as current operating conditions throughout the system.

2.2 System Components

2.2.1 Water Supply (Reservoir)

Raw (untreated) surface water is supplied to the Water Treatment Plant from the Cumberland River (Lake Barkley) in Eddyville. Lake Barkley is a 57,900 acre lake with 1,004 shoreline miles. Lake Barkley provides electric power, flood control, navigation, recreation and water supply to surrounding areas including Princeton. Its supply is essentially unlimited for the area.

2.2.2 Water Treatment Plant

The Water Treatment Plant (WTP) owned and operated by the City of Princeton is a surface water treatment plant and treats raw water from Lake Barkley. The WTP is rated at 3.0 MGD. The water treatment plant is a conventional type process utilizing the Actiflo[1] process manufactured by Kruger, Inc. for floc formation and clarification. Dual media filters follow the Actiflo process. Filtered water is fluoridated and chlorinated and stored in a 400,000 gallon clearwell. The clearwell volume is 13 percent of the plant rating.

2.2.3 Pumps

The Princeton Water Treatment Plant operates two high service pumps. Pumps are Fairbanks Morse 13H-7000AW with 10 stages. The design point for the pump is 2,100 gpm at 600 ft TDH. One pump is controlled by a variable-frequency-drive (VFD), and one pump is controlled with a flow control valve. It appears that if uncontrolled, the pumps would run closer to 2,400 gpm at 480 ft TDH. Operations generally have the speed controlled pump operating at a reduced rate. Operating the 1,785 rpm pump at 1,400 rpm creates a pump curve of approximately 1,700 gpm at 380 ft TDH. Static head is 282 feet (Linton Hill Tank overflow elev. = 662 ft; Clearwell water elev. = 380 ft).

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[1] Actiflo is promoted as a compact, conventional-type water clarification system used in drinking water applications that utilizes microsand as a seed for floc formation. The microsand provides surface area that enhances flocculation and acts as a ballast or weight. The resulting sand ballasted floc display unique settling characteristics, which allow for clarifier designs with high overflow rates and short retention times.

Courtesy of Fairbanks Morse on-line pump selection program, we have graphed the system curve over the pump curve at various pump speeds (see Figure 3 below).

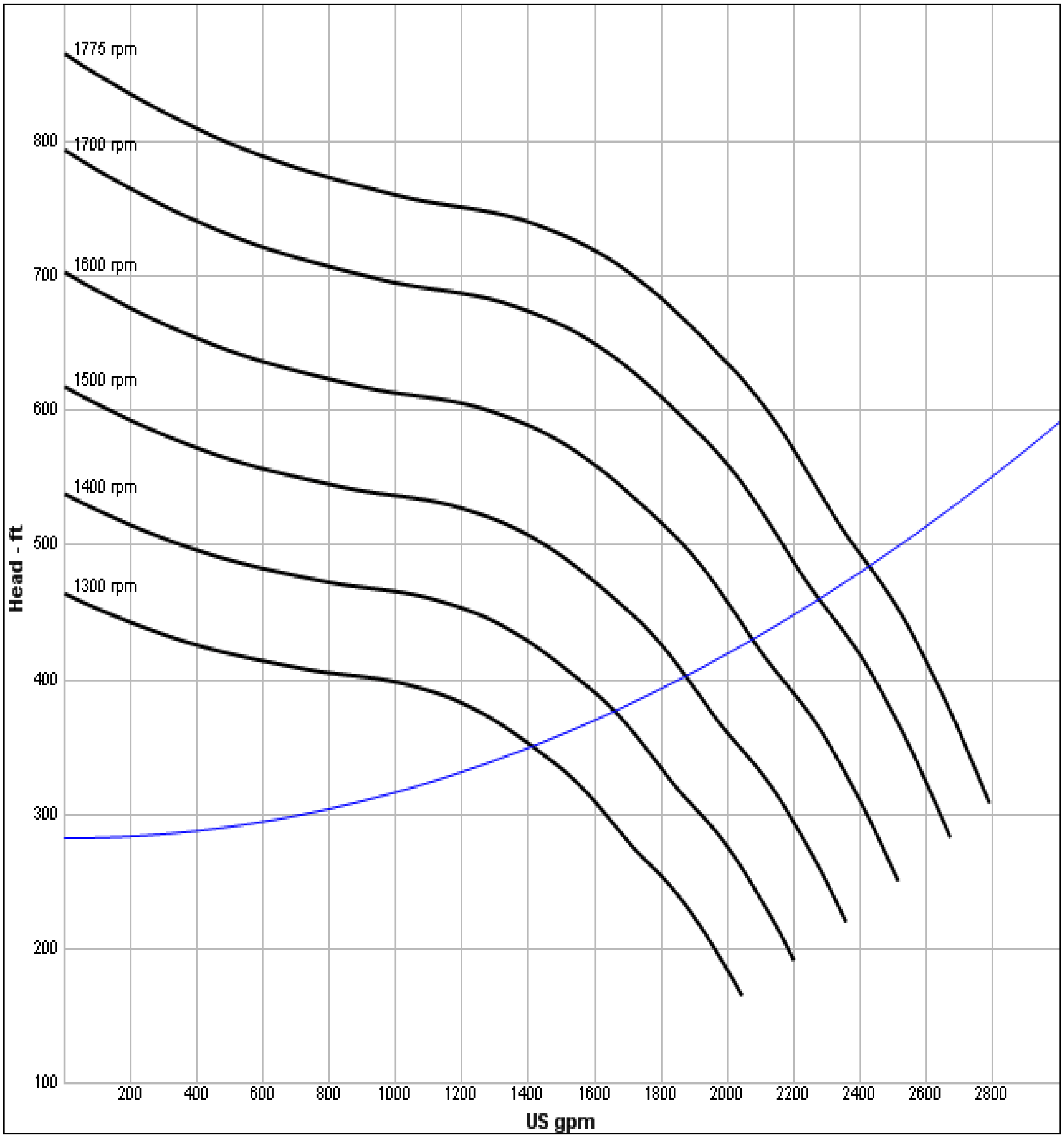

Figure 3: High Service Pump Curve at Various Speeds
Fairbanks Morse 13H.4+ (10 stage)

The distribution system is divided into two distinct pressure zones; Low Service (Zone 1) and High Service (Zone 2). These zones are illustrated on Figure 1 (end of Chapter 2). The Low Service zone is served by the High Service Pumps and supported and maintained by the Linton Hill Water Tank and the Industrial Park Water Tank. This pressure zone operates at a water level of 662 feet MSL. The High Service zone is served by the Linton Hill Booster Pump Station and supported and maintained by the Skyline Water Tank. This pressure zone operates at a water level of 730 feet MSL. The booster pump station, located at the base of the Linton Hill Tank, pumps approximately 1,000 gpm into the High Service zone and increases the hydraulic grade line by 68 feet.

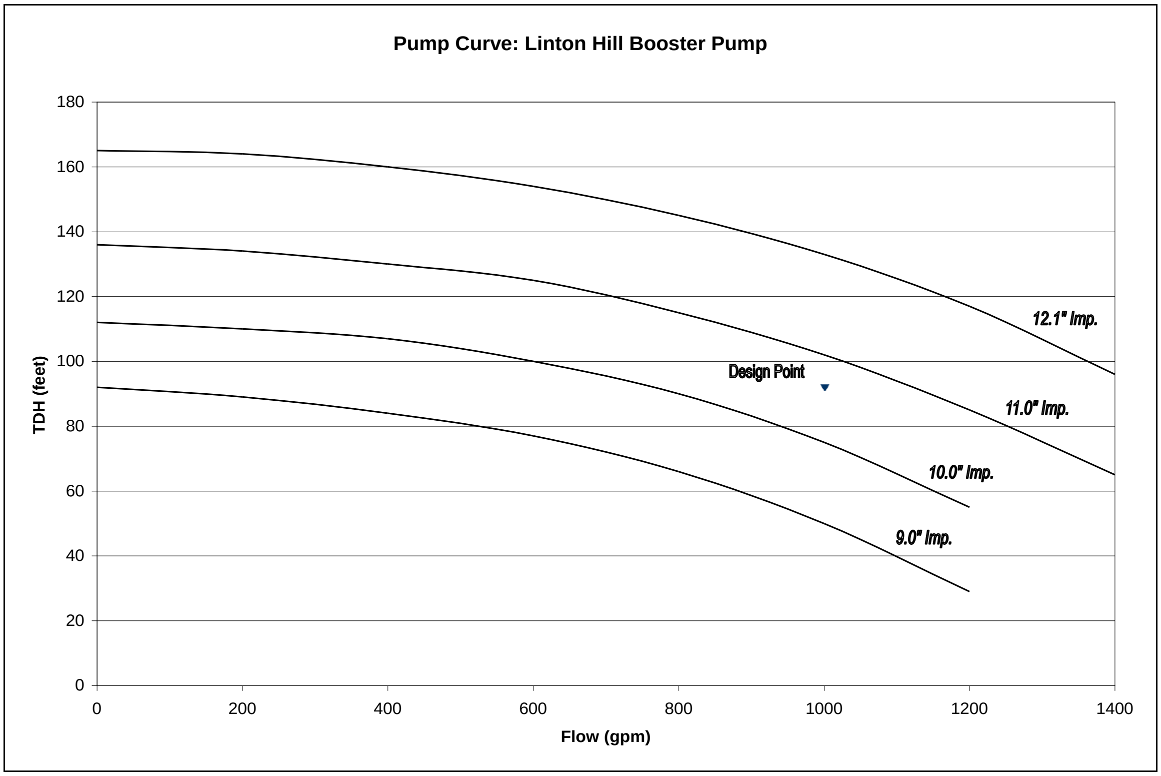

Figure 4: Linton Hill Booster Pump Curve

2.2.4 Pipes

The PW&W water system provides safe and reliable drinking water to its customers through approximately 89 miles of transmission and distribution lines ranging in size from 2-inch to 16-inches. There exists one 16-inch transmission main approximately 76,000 LF in length from the WTP to the Linton Hill Water Tank. This line is adequately sized and the High Service Pumps are capable of filling the tank. Other lines, smaller in size, branch off and create loops around this line. The majority of the pipes in the water system are 6-inch diameter. A complete listing of the pipe information modeled for the Princeton Water System can be found on Figure 1 (end of Chapter 2).

2.2.5 Valves

A number of devices used to stop or control flow in the pipelines such as gate and butterfly valves are present throughout the Princeton distribution system. Generally, these valves are assumed open during all flow scenarios with no associated headloss, unless otherwise specified by observed field data or information provided by the water utility. Specific valve closures are present in the existing distribution system to create pressure zone boundaries between Zone 1 and Zone 2. These valves are accurately represented in the model at locations of pressure boundaries provided by PW&W.

2.2.6 Storage

The ability of water distribution systems to adequately store treated water is essential to meet daily demands of the system, as well as provide water for fire protection and other emergency demands. The Princeton water system has three water storage tanks. Two tanks are elevated, and one tank is ground storage. Two tanks, the Linton Hill Tank (ground storage) and the Industrial Park Tank (elevated storage), are located in the low service pressure zone. One tank, the Skyline Tank (elevated storage), is located in the high service pressure zone. Table 1 displays the characteristics of all three tanks.

Tank Name	Ground Elevation (ft)	Overflow Elevation (ft)	Diameter (ft)	Size (gallons)	Method of Control
Linton Hill	620	662	50	600,000	Altitude Valve
Industrial Park	532	662	70	1,000,000	Telemetry
Skyline	603	730	56	500,000	Telemetry

Table 1 – Water Storage Tank Summary

According to Kentucky Division of Water criteria, a water system shall have 24 hours of useful storage. Additionally, the storage shall be cycled completely within 72 hours (1/3 of the tank cycled per day). The current system demand ranges between 1.6 MGD and 2.1 MGD on peak demand days. The water treatment plant is rated at 3.0 MGD, and there is 2.1 MG of available storage. Therefore, the system has sufficient storage at existing conditions.

2.3 Operation Schedule

Two high service pumps are housed at the WTP to supply treated water to the distribution system. Only one of these pumps operates at a time. Generally, operation follows the same daily pattern. Pumps operate for 16 to 20 hours each day. Four times a day the pump in service is shut down to allow for a filter backwash cycle. The pump may be shut down from 30 minutes to one hour for each of the four filters. The selected high service pump is generally started between 4:00 AM and 6:00 AM each morning. Pump shutdown is generally between 10:00 PM and 12:00 AM (midnight) each night.

The high service pumps are not operated at 100 percent capability. One pump has a VFD. This pump at various speeds was shown graphically in Figure 3. The other high service pump has a flow control valve that effectively reduces the pump output and head.

Chapter 3 – Methodology and Model Development

3.1 Design Criteria

The analysis of the Princeton water distribution system for existing system deficiencies and corresponding recommendations was based on the Kentucky Division of Water Design Criteria. This document includes criteria for all aspects of public water system design, including source and treatment standards. This analysis however, was only concerned with the requirements relating to pumping facilities, finished water storage and distribution systems. This study did not include a water treatment plant evaluation. The design criteria applicable to the PW&W water system on which the analysis was based follows.

3.1.1 System Storage

The total system storage should be sufficient to provide adequate flow and pressure throughout the water network under peak demand, as well as provide sufficient water for fire-flow emergencies. The minimum acceptable total system storage is that which provides at least the average 24-hour system demand. However, the ideal system storage should exceed the projected 24-hour system demand and account for fire-flow emergencies. Distribution storage guidelines require demonstration that the storage water is completely turned over (cycled) in a 72-hour period, or 33 percent each day.

3.1.2 Pressure Constraints

A minimal pressure of 30 psi must be available on the discharge side of all meters throughout the entire distribution system.

3.1.3 Flushing

A minimum velocity of 2.5 fps must be achieved during flushing while maintaining a minimum pressure of 20 psi at all points in the distribution system. For this analysis, the flushing was conducted on all dead-end lines in the system.

3.1.4 Fire Protection

Fire hydrants for use in fire protection shall not be connected to water mains smaller than 6-inches in diameter. In addition, the minimum pressure at all points in the distribution system must be at least 20 psi while providing a minimum fire-flow of 500 gpm for a period of two hours. Note: At the request of PW&W, the goal was to achieve 1,000 gpm of fire-flow for two hours while maintaining a minimum residual pressure of 20 psi throughout the system. However, for the purpose of this study, only nodes that could not supply 500 gpm of fire-flow were considered deficient.

The fire-flow analysis was conducted on all lines with a diameter of 6-inches or greater. Smaller lines are considered underserved for fire protection even if they could adequately deliver water at the minimum accepted standards. A 6-inch water line is able to deliver 500 gpm with a head loss of 20 feet per 1,000 feet. A 4-inch water line can only deliver

175 gpm with the same resulting head loss. The water system has approximately 53,000 LF of looped 4-inch line.

3.2 Model Selection

Water system modeling is a critical part of the design and operation of water distribution systems that are capable of serving customers reliably, efficiently and safely. Hydraulic models enable one to identify weaknesses in the hydraulics of existing systems and assist in the design and operation of a more efficient system in terms of daily operation and emergency situations. The benefit of computer models is that they are able to mathematically approximate the behavior of a distribution system by iteratively solving thousands of flow continuity and energy equations that would otherwise be impractical. Several hydraulic modeling programs for water system evaluations are available. Hethcoat and Davis, Inc. has chosen Infowater, which is developed, owned, distributed and supported by MWH Soft, Inc., to model the Princeton water distribution system. This program allows for the construction of a user friendly, graphical representation of the water distribution system and can be integrated with other software such as Microstation and ArcGIS.

3.2.1 Model Methodology

The main objectives of water system modeling are to evaluate flow and pressure characteristics, storage capacity and tank level fluctuations to ensure the applicable water system criteria are met, and ultimately to determine if the system provides satisfactory service to its customers. These items can be analyzed for a specific time of day or over an extended period of time. At a specific time, network parameters are analyzed against constant boundary conditions to give a single set of flow and pressure results. Boundary conditions include set pumping rates, set tank levels, constant water demand, etc. Alternatively, variable flow and pressure results, tank levels, pumping rates and water demand can be analyzed over an extended period of time.

Analysis of distribution performance depends on the calculation of the hydraulic grade lines (HGL) throughout the system. The HGL is the sum of the elevation head and the pressure head at a specific junction for any given time. The difference in HGL between two points is what drives the flow from a higher HGL to a lower HGL. Pumps are added to systems to increase hydraulic grade lines, thus enabling the system to fill tanks and supply water to customers at high elevations.

The difference in HGL between an upstream and downstream junction is a combination of the elevation difference and head loss through the connecting pipes. Head loss represents the amount of energy used when water moves from one point to another and is comprised of friction losses and minor losses. Friction losses occur between the pipe wall and water surface. Minor losses occur through pipe fittings such as bends, reducers and tees. Minor losses in distribution systems are typically ignored, as their contributing head losses are negligible. Friction losses, however, are considered significant, especially for smaller pipe sizes and under high flow conditions.

Infowater calculates the headloss due to pipe friction for every pipe section under all flow conditions and subtracts this loss from the upstream HGL. The software uses the Hazen-Williams formula to calculate headloss, which accounts for the friction losses through the assigned Roughness Coefficient, C. The C factors, which are unique to each type of pipe material, are high for new, smooth pipes that contribute low friction loss (C = 140), and are low for old, rough pipes that contribute high friction loss (C = 80). The Princeton Water and Wastewater hydraulic model has been constructed using a C factor of 100 for every pipe in the network.

3.3 Model Development

3.3.1 Data Collection

The Princeton Water and Wastewater water distribution system was previously mapped by Quest Engineers, Inc. PW&W provided Hethcoat and Davis, Inc. with copies of these maps, which served as the basis of our network model. Once pipes were drawn into the model from these maps, copies were sent to PW&W to be reviewed for locations and sizes of system pipes, and to identify missing components. Once the revisions were returned to Hethcoat & Davis, Inc., all corrections identified by PW&W were made to the model.

Besides physical system data, information to define the water system operation was also needed. This data was provided by PW&W, and included pump curves, tank level records and operations data and controls. Total system demands, including residential, commercial, and wholesale demand, were also provided by PW&W, and will be discussed in Section 3.4.

3.4 Demand Allocation

3.4.1 Spatial Distribution

PW&W provided total system water demand data from November 2005 to April 2008. This data included total water sold to residential, commercial, and wholesale (Caldwell County and Lyon County) customers. For calibration purposes, the total demand provided for November 2007 was input into the model because physical data was collected during this period which could be compared to the modeled results. Further explanation is provided in Section 3.5. For analysis of the system at existing peak demands, the average demand over the months of July 2007 to September 2007 was calculated and input into the model.

The total demand provided by PW&W represents the amount of water sold to customers in the entire system. However, there is an additional demand that must be added to the system to quantify unaccounted water. This demand accounts for water lost during flushing, fire-training, backwashing of filters at the water treatment plant, overfilling of tanks, leaks, inaccurate meters, etc. The amount of water loss should reflect the amount of raw water treated at the plant minus the amount of water sold. Once this value was established (gpm), the demand was divided evenly over all nodes in the system.

3.4.2 Temporal Distribution

Water demand normally varies throughout the 24-hour day and can follow different patterns based on the type of customer being supplied. However, demand is typically low over night and high during the morning and evening hours. Three diurnal demand distribution patterns were defined for this analysis: residential, commercial and water loss. Residential and wholesale customers were assigned the residential pattern, commercial customers were assigned the commercial pattern, and the water loss demand was assigned the water loss pattern. Residential customers and commercial customers have different water usage needs throughout the day. Following in Figures 5, 6 and 7 are the diurnal demand patterns assigned to each customer / demand type defined in the model. Initially, the base demand (gpm) for each individual customer was defined in the model with its corresponding demand pattern. The demand was placed on nodes that corresponded with the actual geographical location of the customer being served. The base demand was then updated at each time step based on the corresponding multiplier provided in the diurnal demand pattern.

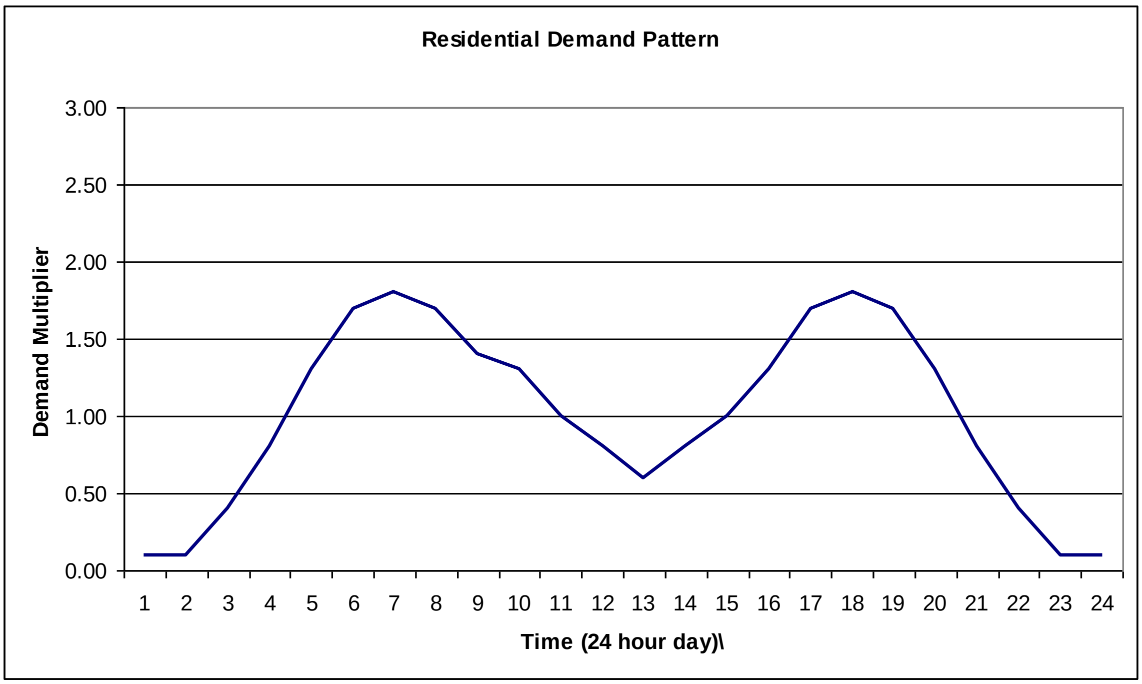

Figure 5

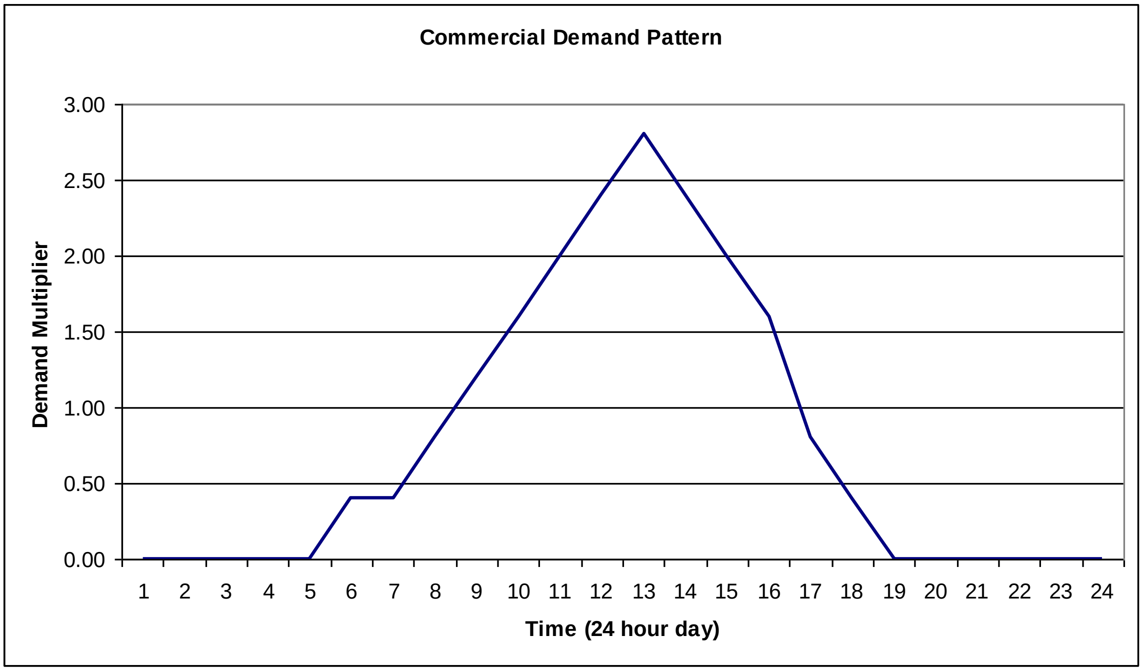

Figure 6

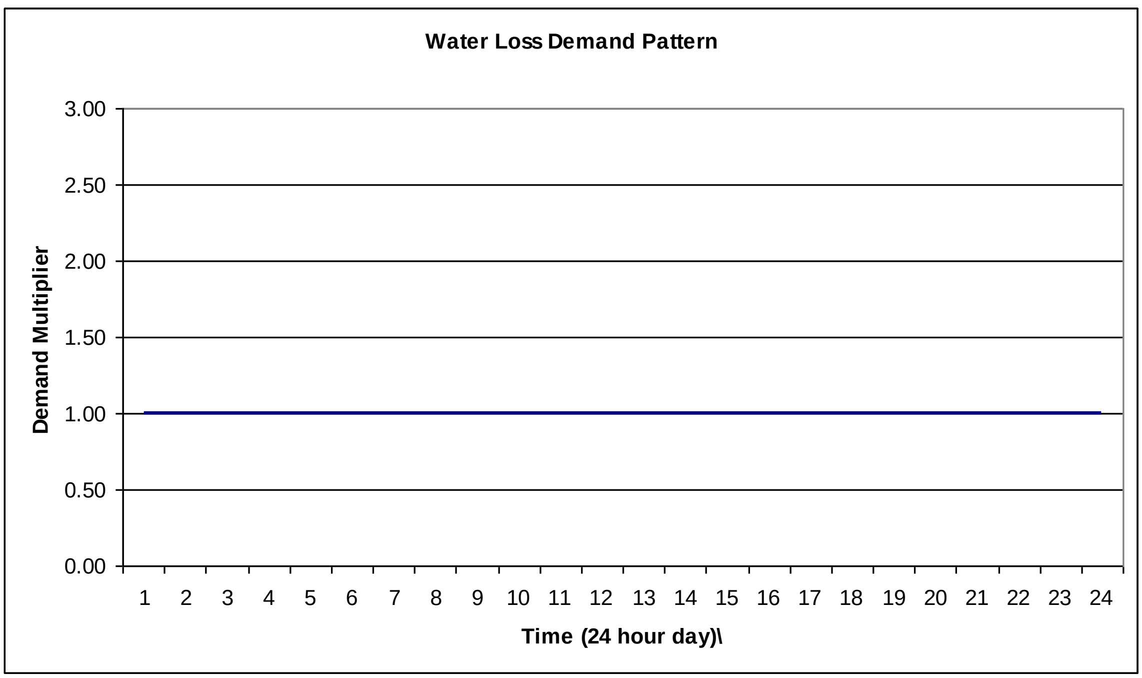

Figure 7

3.4.3 Projected Demands

Based on research from the University of Louisville Urban Studies Institute, the City of Princeton's population is expected to decline over the next 12 years (projections not available past 2020). Caldwell County's population is expected to decline by the year 2030, while that of Lyon County is expected to slightly increase by the year 2030. In total, the population of Princeton, Caldwell County and Lyon County is expected to decline by the year 2020. Figure 8 illustrates these projections.

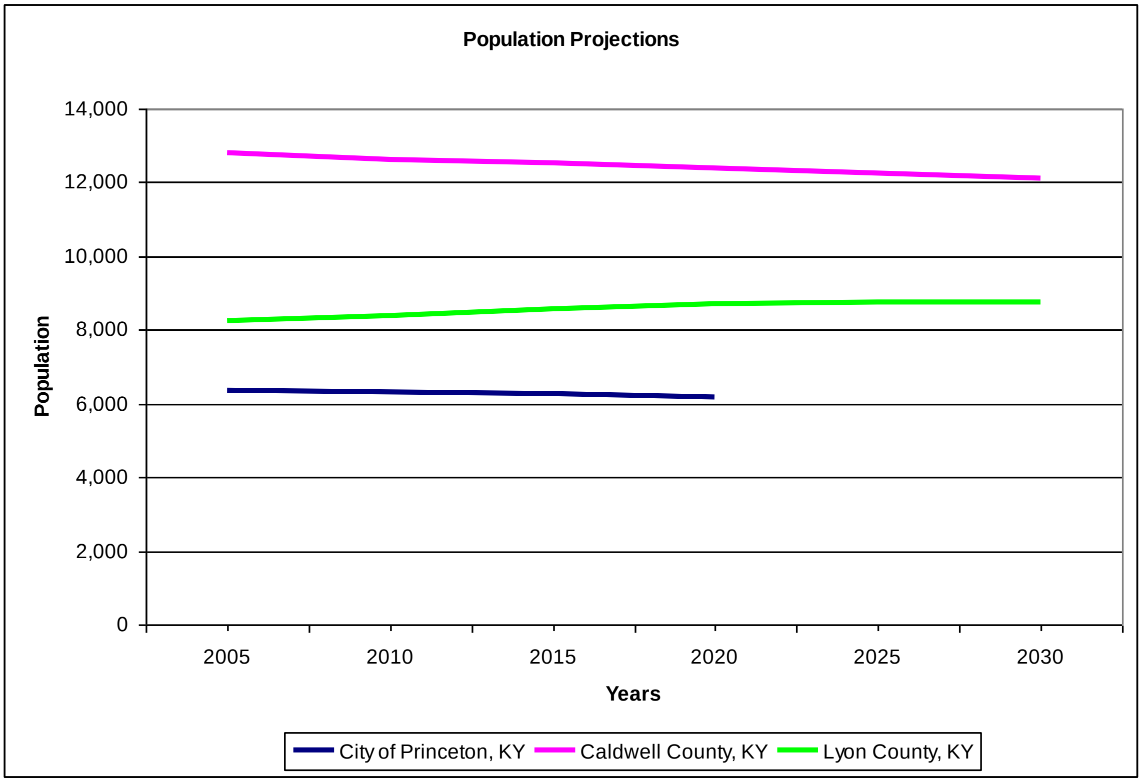

Figure 8

Considering these findings, it was decided to maintain the average peak demand from July 2007 to September 2007 in all future scenarios to identify improvements, as this is the highest demand the PW&W water system will experience based on the projections. In addition, PW&W informed Hethcoat and Davis, Inc. that a Super Wal-Mart (currently under construction), a new hospital and a new strip mall would be constructed in the near future near the Industrial Park Water Tank on Highway 62. Projected demands for each were included in our future scenarios under the commercial demand pattern. Table 2 displays these projected demands.

Customer	Demand (GPM)
Hospital	10.40
Super Wal-Mart	20.20
Strip Mall	30.00

Table 2

3.4.4 Flushing Analysis

To determine pressure deficiencies throughout the distribution system caused by flushing, flow was added to a single dead-end node at a time to achieve a minimum of 2.5 fps in the dead-end main. The amount of flow added was based on that required to maintain 2.5

fps in the corresponding pipe size. Table 3 displays the added flows for each dead-end node.

Line Size at Dead-end Node (inches)	Flow Added (gpm)
2	26
3	55
4	98
6	220
8	392

Table 3

A flushing demand pattern was added to each corresponding dead-end node to represent flushing one hour a day at 5 AM. This time was chosen as tanks were at the lowest levels and the high service pumps were not on, thus system pressures were at their lowest values. The flushing pattern is illustrated in Figure 9. Pressures throughout the entire system were analyzed during the time of flushing to identify any node with a pressure below 20 psi, which would reflect a deficient (or underserved) location in the system.

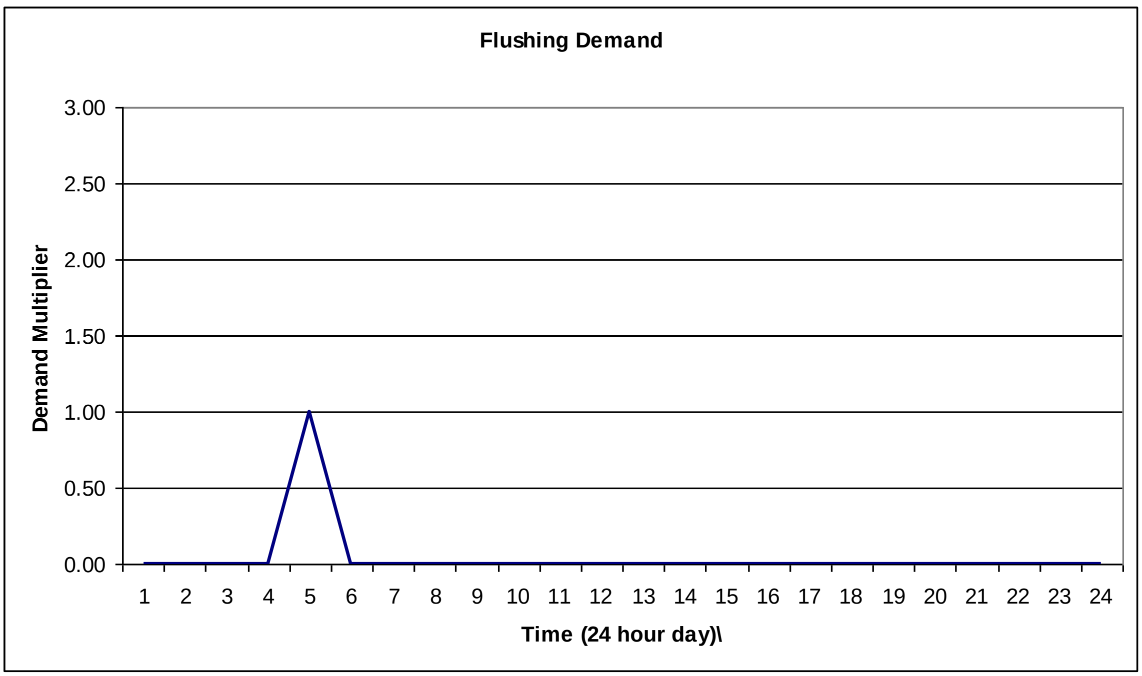

Figure 9

3.4.5 Fire-Flow Analysis

To determine water distribution system deficiencies under stressed conditions, the system was analyzed under fire-flow conditions. Infowater has a fire-flow analysis tool to determine if the system can meet user-specified fire-flow demands while maintaining the minimum design pressure constraint of 20 psi. A fire-flow analysis was performed individually on all system nodes connected to water mains with diameters 6-inch or greater, and the analysis was performed at 5 AM (see Section 3.4.4 for time selection).

The program then reported the available flow for each such node so that every node in the entire network could maintain a minimum pressure of 20 psi.

3.5 Model Calibration

3.5.1 Calibration Philosophy

Before a model can be accurately used to model system behavior and make decisions for design purposes, the model must first be calibrated to simulate field-observed pressure and flow values. Calibration involves iterations of adjusting model parameters so that actual pressure and flow values are reflected in the model. The parameters that can be adjusted during this process include open or closed valves, pipe diameters, pipe roughness coefficients (C values), elevations, demands, and operations data such as tank elevations and pump rates.

In order to verify model calibration, actual field-observed data must be obtained for comparison. This information was provided by placing a series of digital pressure recorders at various locations throughout the distribution system. Each recorder was placed on a particular hydrant for six to nine days at a time from November 6, 2007 to December 6, 2007. Recorded results were downloaded to the computer and compared with modeled results.

3.5.2 Calibration Results

The first step in the calibration process involved setting the high service pump in the model to match existing pump characteristics. This was achieved with help from PW&W, who provided the telemetry data recorded for the high service pump. With the flow information and run times provided, we were able to manually input the information for the pumps in the hydraulic model. Similarly, the Linton Hill Booster Pump telemetry data was inserted into the model.

The next step involved matching the tank conditions in the model to the existing tank conditions in the system. PW&W provided H&D with records of the tank levels from January 2006 to March 2008. Considering that the demand used to calibrate the model was based on historical records for the month of November 2007, the modeled tank behavior had to match historical tank records for November 2007. Therefore, a 72-hour span of modeled tank levels was graphed with a 72-hour span of historical tank records from November 13 to November 15, 2007 (Tuesday – Thursday). Weekday records were chosen in order to avoid the hydraulic inconsistencies encountered over a weekend. The graphs of each tank are shown in Figures 10, 11 and 12.

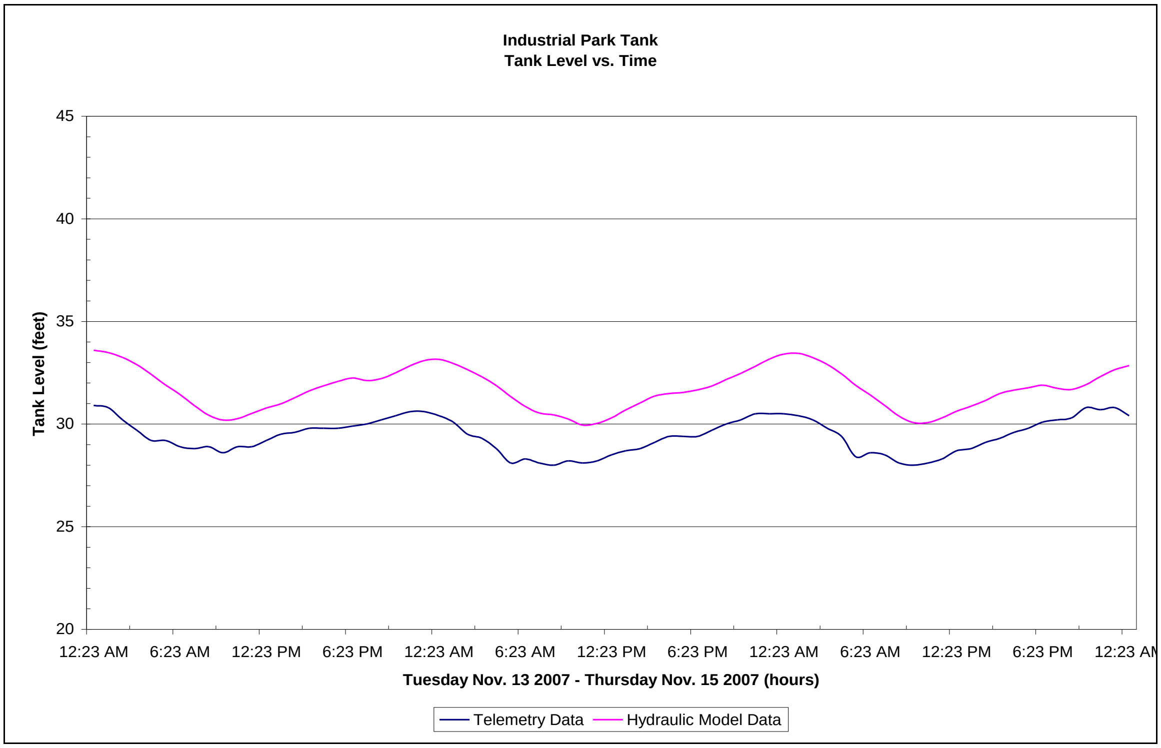

Figure 10: Industrial Park Tank

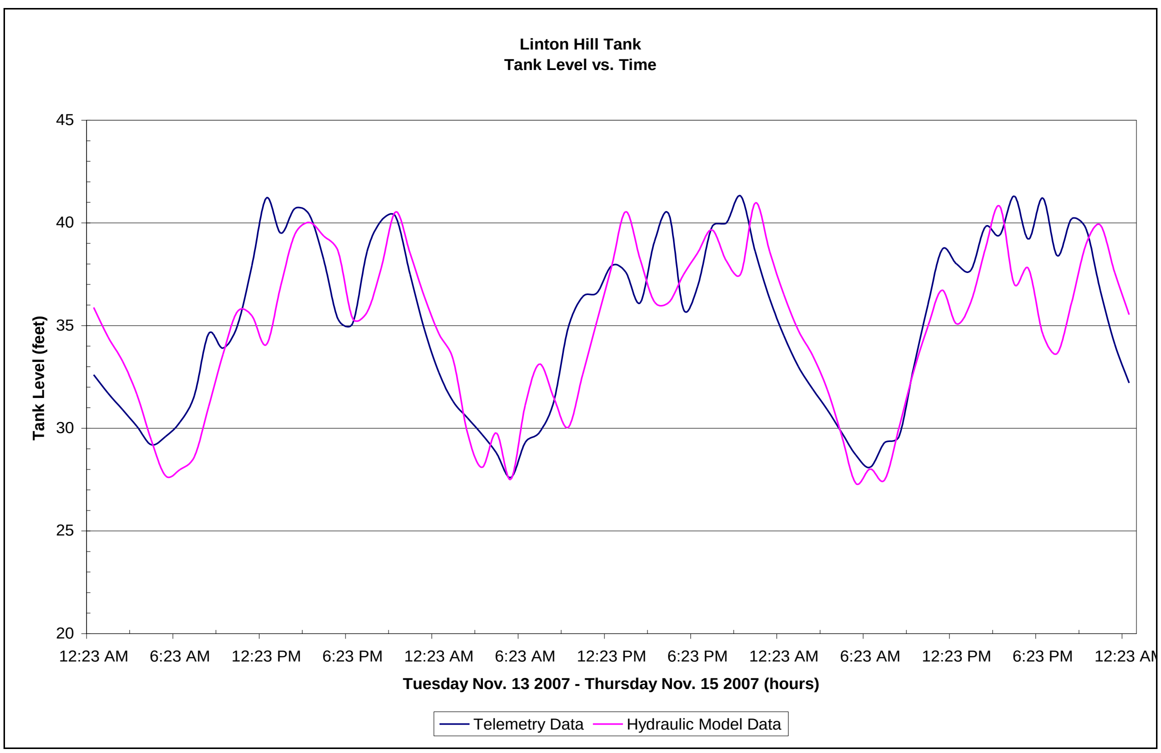

Figure 11: Linton Hill Tank

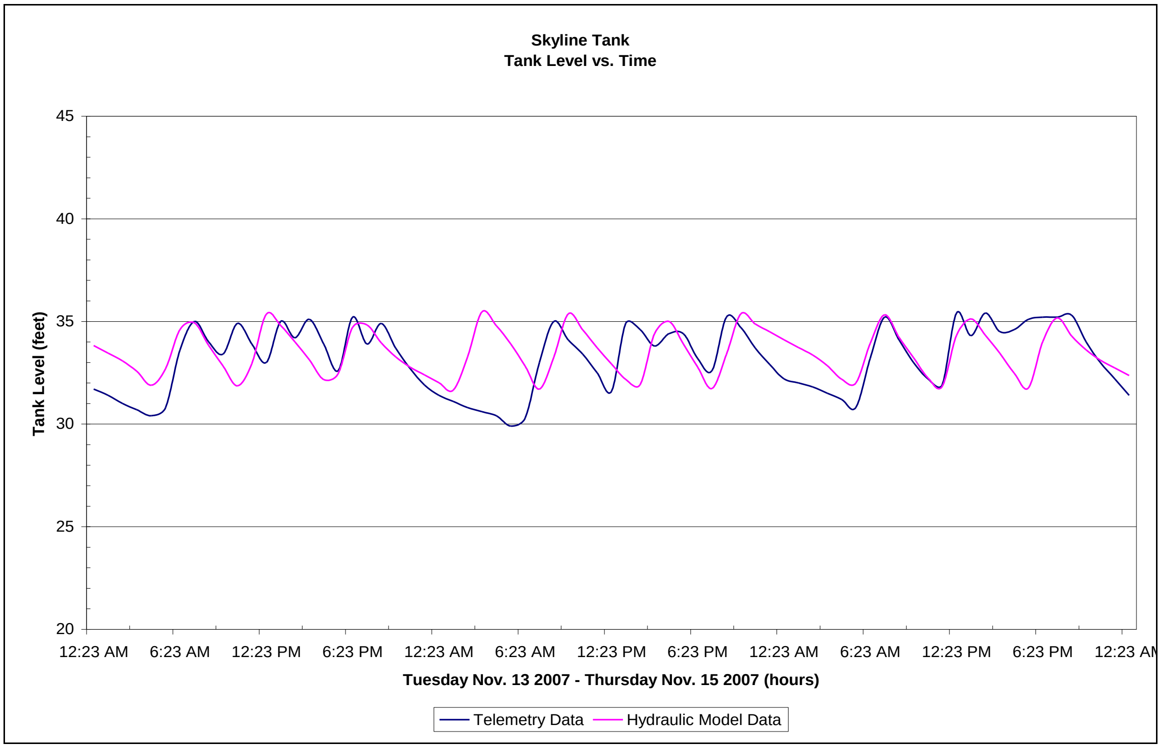

Figure 12: Skyline Tank

The final step of the calibration process involved comparing pressures of the field-observed data with pressures of the corresponding nodes in the model. With the help of PW&W, Hethcoat and Davis, Inc. was able to identify the exact location of a hydrant in the field where a pressure recorder had been placed. H&D then matched that hydrant to a node at the same location in the model. The model was run for 144 hours (six days), and pressure results for the corresponding node from hours 72 to 144 (three days) were copied into an excel spreadsheet. These hours were selected as the system was most balanced during these times.

Field-recorded pressure results were then observed for the corresponding hydrant, and the most consistent three-day span of results beginning at 12:00 a.m. were selected. In this manner, pressure fluctuations caused by flushing, fire flows, etc. were ignored as they represent unforeseen circumstances that Infowater cannot simulate. Results taken over the weekend were also ignored as they do not reflect a consistent pattern of water usage.

The 72-hour field-observed results were also copied into the same excel spreadsheet, and actual field pressures were graphed versus modeled pressures to ensure accuracy of the model. In total, field-observed pressure recordings at ten locations were compared to corresponding nodes in the model to ensure that the model was calibrated. Figures 13 through 22 illustrate these results.

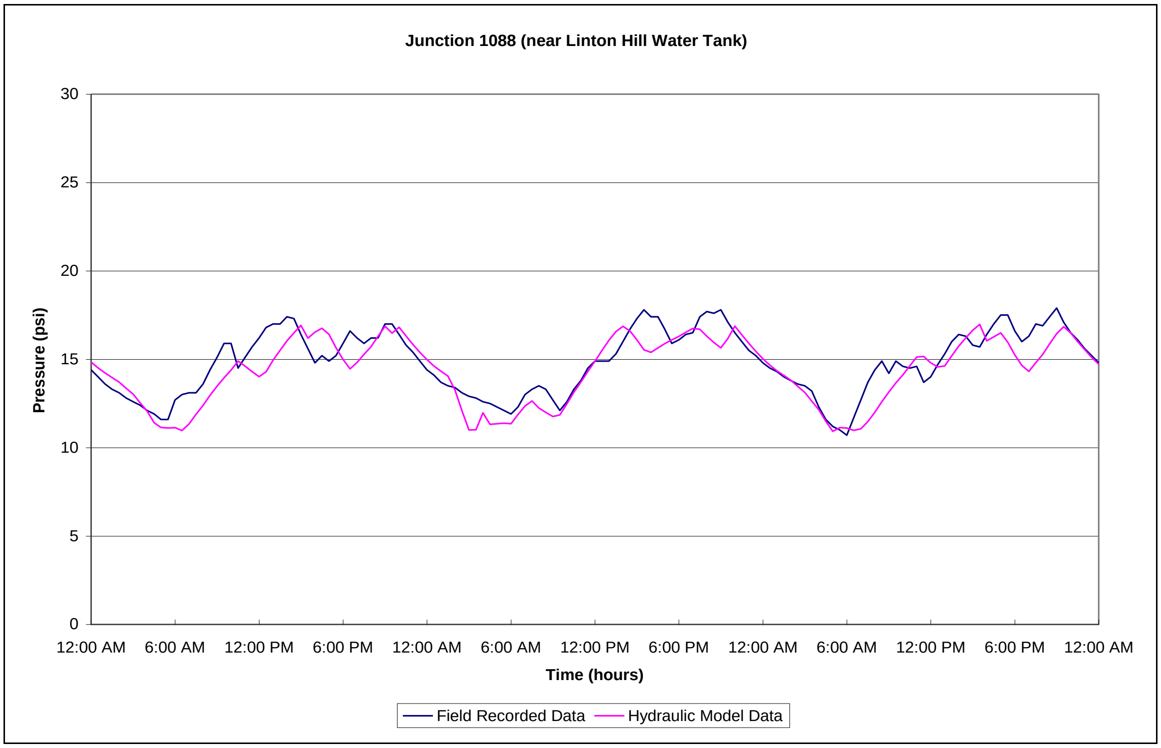

Figure 13

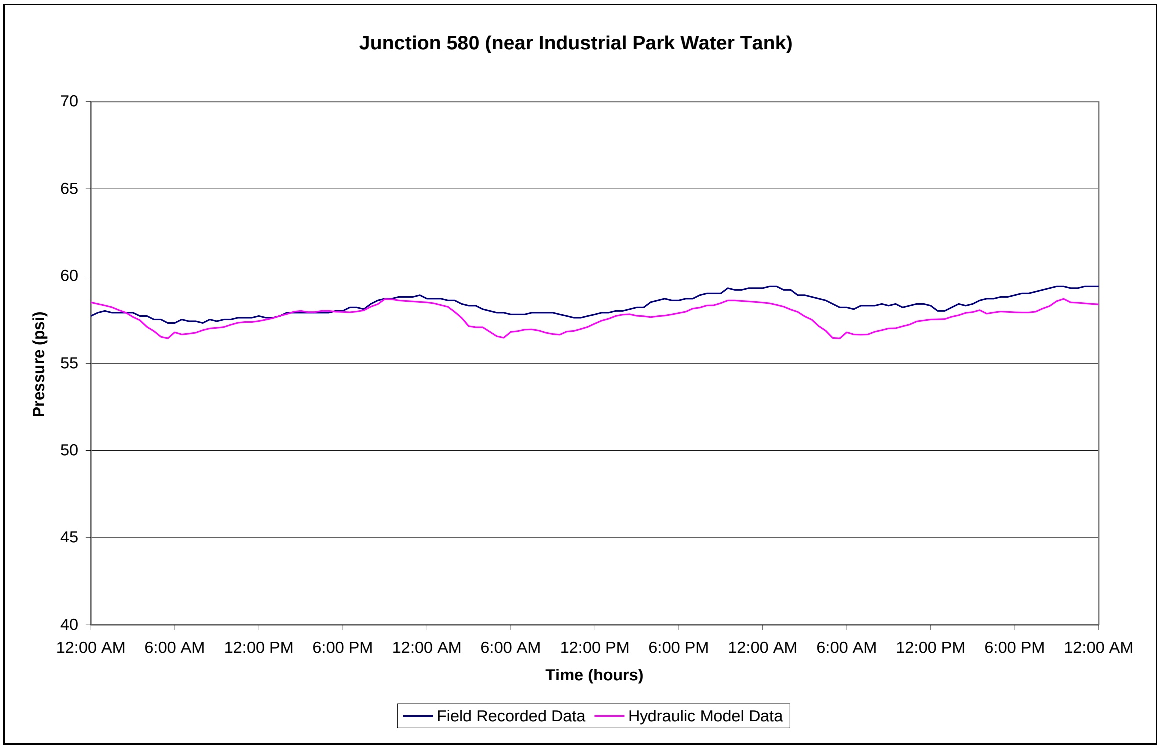

Figure 14

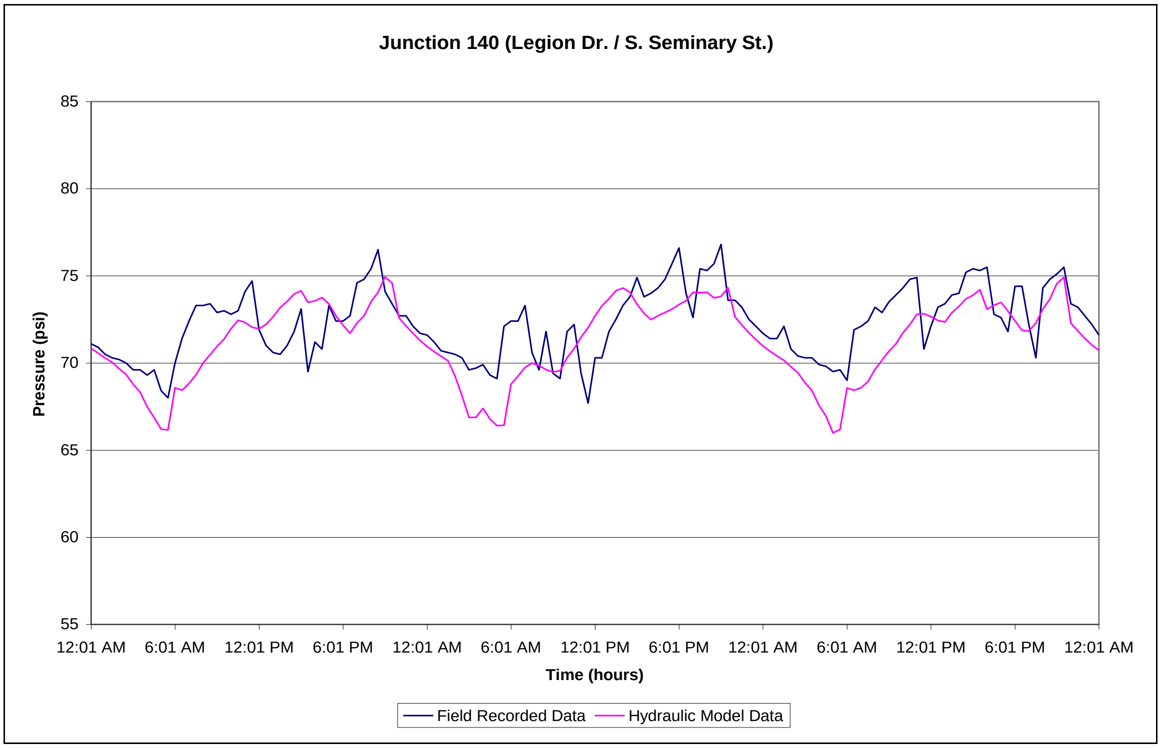

Figure 15

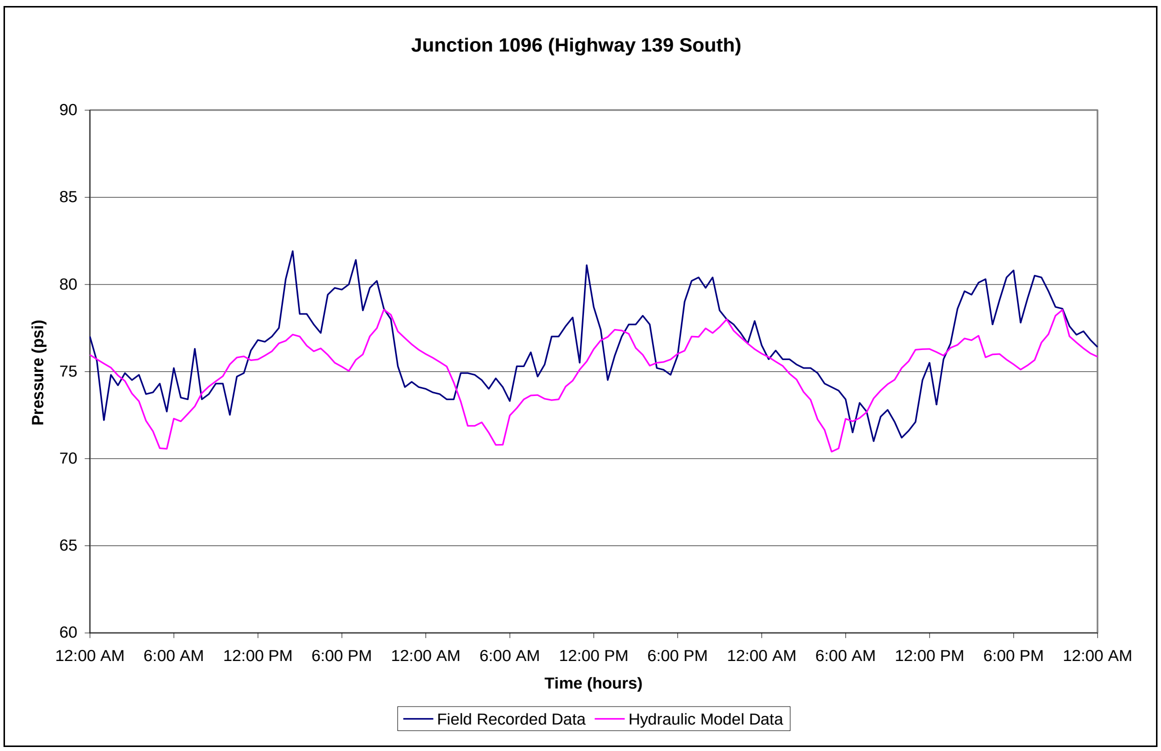

Figure 16

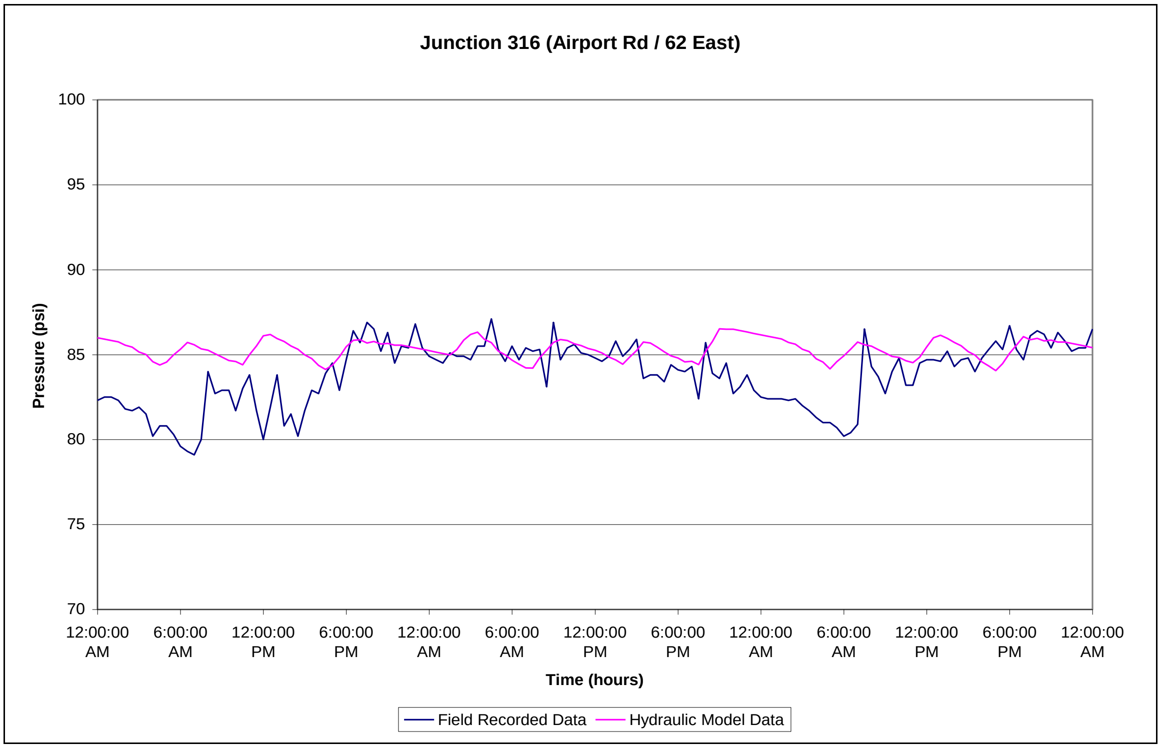

Figure 17

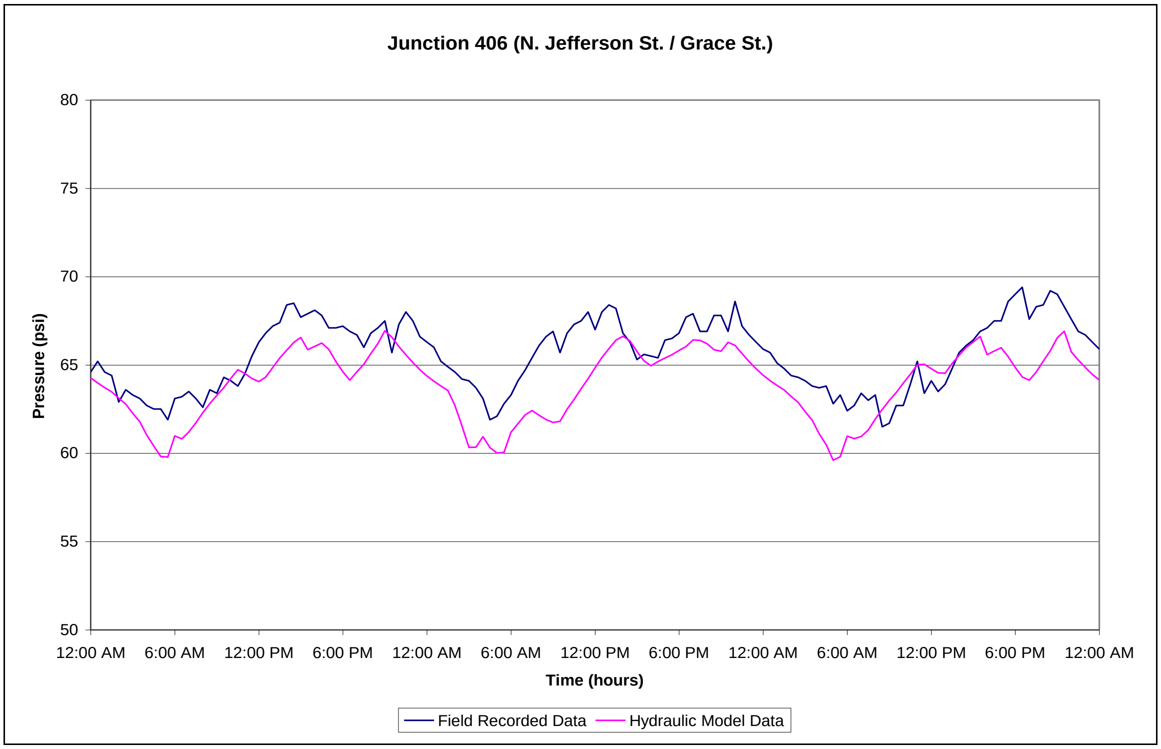

Figure 18

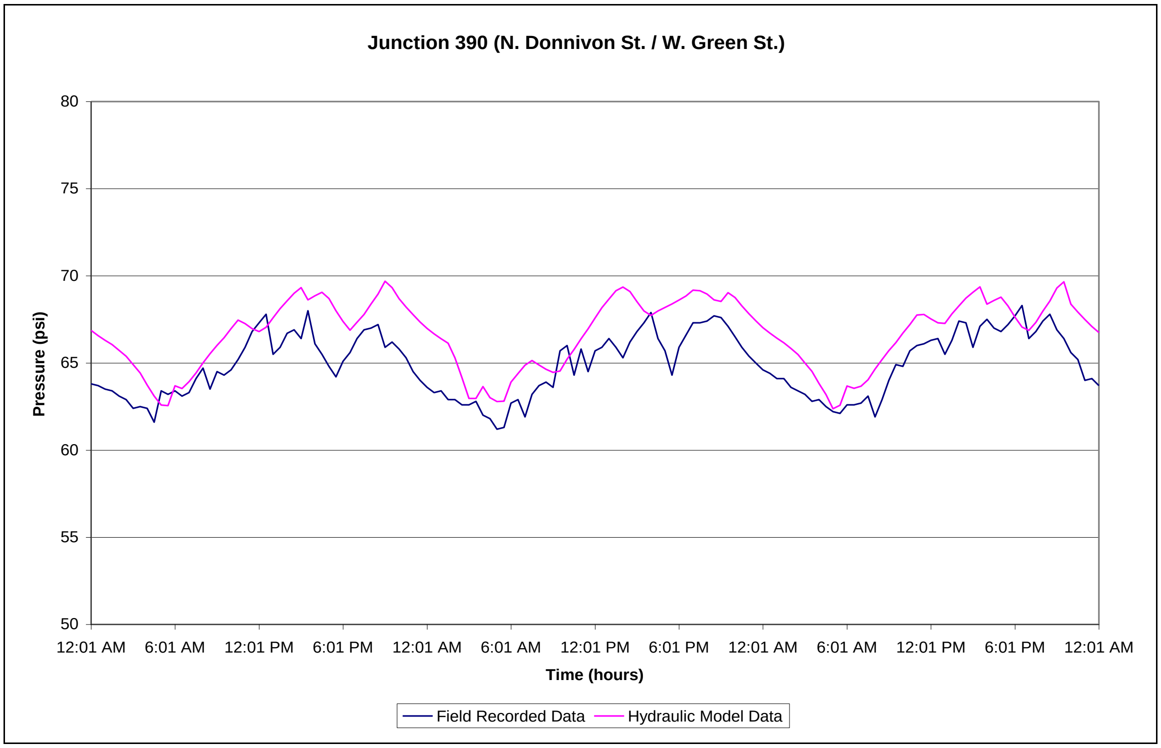

Figure 19

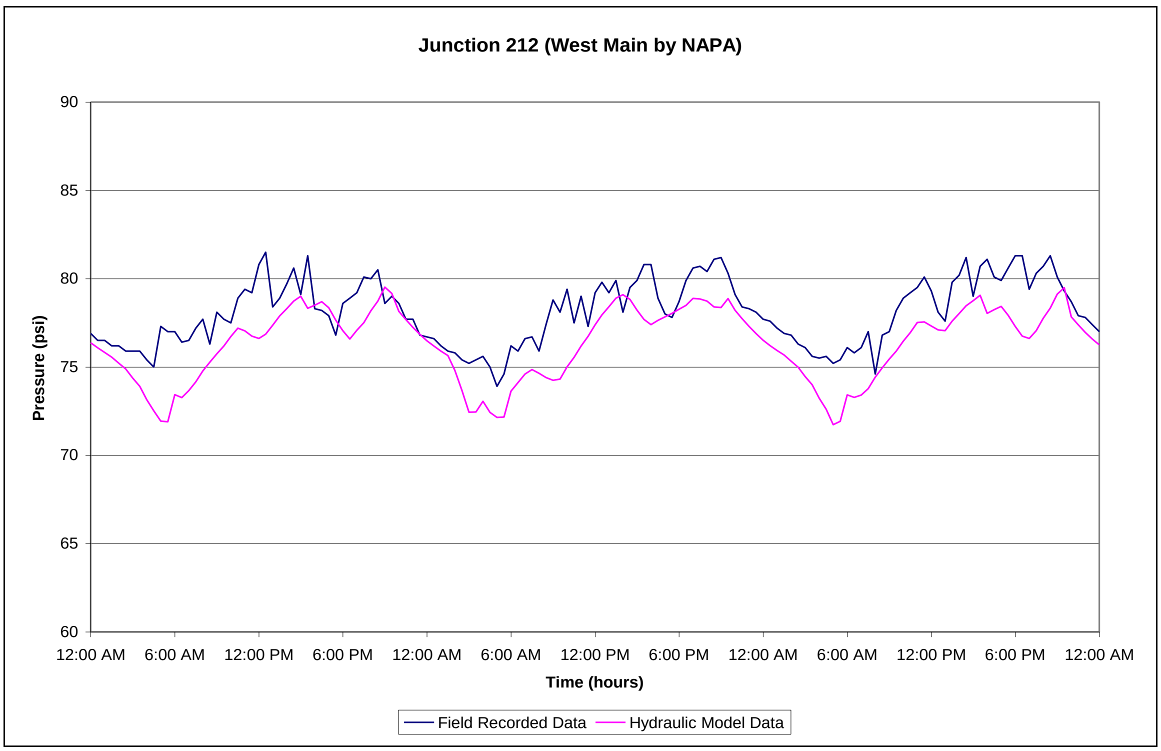

Figure 20

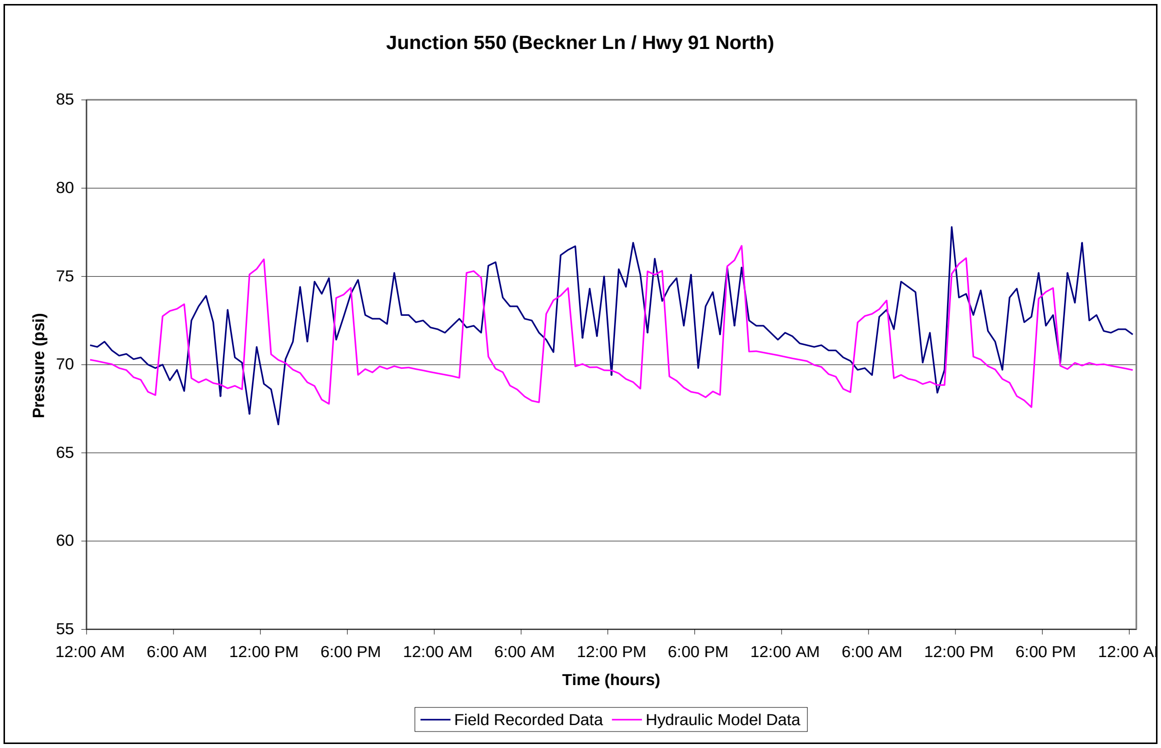

Figure 21

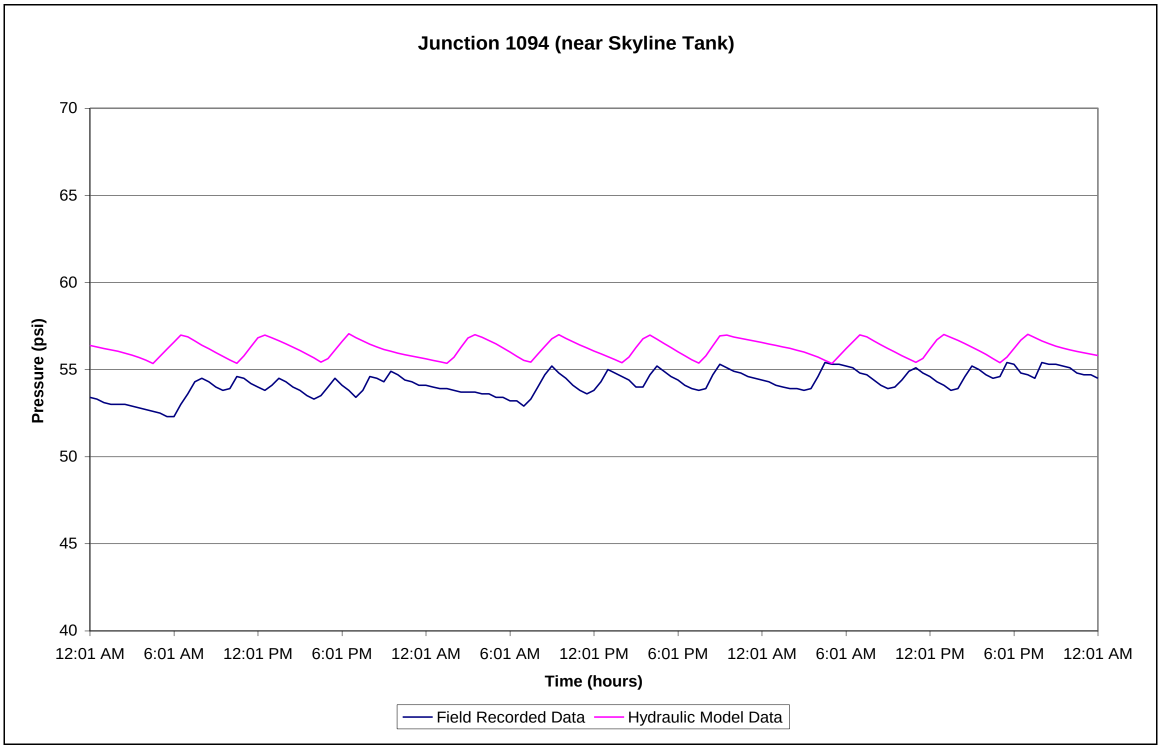

Figure 22

Considering the closeness of modeled results to field-observed results, H&D concluded that the hydraulic model was satisfactorily calibrated. Thus, future modeled scenarios would produce accurate results.

Chapter 4 – Model Results

4.1 General

The existing Princeton water distribution system was analyzed to determine the necessary improvements needed over the next 20 years to enable the system to provide the State allowable minimum domestic pressure, provide fire protection to the entire network, provide adequate flushing at all locations, maximize system storage and tank usage, and accommodate the projected 20-year system demand growth.

Separate scenarios were used to simulate the water system's response to varying demand conditions and system improvements. These simulations helped determine necessary improvements and the effects of each improvement. The scenarios included:

Calibration Scenario (based on actual November 2007 demand and existing November 2007 system characteristics – used to calibrate the model)
Existing Scenario (based on average peak demand from July-September 2007 and existing July-September 2007 system characteristics – used to stress the model and identify deficiencies with existing system)
5-Year Improvement Scenario (based on projected demand – used to improve on existing system deficiencies)
10-Year Improvement Scenario (based on projected demand – used to improve on existing system deficiencies)
15-Year Improvement Scenario (based on projected demand – used to improve on existing system deficiencies)
20-Year Improvement Scenario (based on projected demand – used to improve on existing system deficiencies)

Considering field-observed data was obtained in November 2007, it was necessary to have two scenarios based on existing conditions. These scenarios include (1) Model Calibration, which verified accuracy of model relative to field-observed data, and (2) Peak Demand Conditions, which stressed the model to identify deficiencies under peak demand.

4.2 Existing Princeton Water System

4.2.1 Normal Operation

As discussed in Chapter 2, the existing water distribution system includes one ground level storage tank and two elevated storage tanks. At the water treatment plant, there are two high service pumps to supply treated water to the distribution system and storage tanks. Only one high service pump operates at a time from approximately 4-6:00 am to 10-12:00 pm, for a total of 16 – 20 hours a day. There is also one booster pump station pumping to the elevated Skyline Tank, creating one additional pressure zone.

The hydraulic model is constructed of 521 nodes and 636 pipes. Each node is a junction of two or more pipes, or the endpoint of a dead-end pipe. Each node has an assigned ground elevation and a demand that is representative of the water used by customers in the area of the node, plus a water loss component. Note: Water loss was distributed evenly across all nodes. This method is not precise but could be rather accurate. Normally, nodes are in close proximity in the older parts of town where streets and water lines have many intersections. This is commonly where older lines exist. As the system expanded away from the central part of Town, building, road and water line densities decreased. Water lines in these areas will be newer and they will also have fewer nodes. Older water lines in a more densely configured network and newer water lines in a less dense network may result in an even water loss across each node being an effective distribution of unaccounted water.

4.2.2 System Deficiencies

System deficiencies have been identified and grouped into four categories, based on results from the Existing scenario. These categories include *System Storage*, *Pressure*, *Flushing* and *Fire-Flow*. As mentioned at the beginning of this chapter, these deficiencies were corresponding to peak demand data from July 2007 to September 2007. Each category is described below.

System Storage Deficiency

PW&W identified the elevated Industrial Park Tank as a system deficiency due to its inability to cycle the water level inside the tank. This deficiency was verified in our model, as seen in Figure 23. The water level at this tank fluctuates approximately 4.0 feet. Because nearly stagnant conditions exist in the tank, maintaining adequate chlorine in the tank is difficult. This issue is actually a two-part problem. As the Industrial Park Tank has the same overflow elevation as the Linton Hill Tank (662 feet above MSL), both tanks should behave similarly in ideal conditions. However, the Linton Hill Tank, which fluctuates appropriately, is fed primarily by the 16-inch main from the WTP. The Industrial Park Tank, on the other hand, is fed from the 16-inch main through a series of 2-inch, 3-inch, 4-inch, 6-inch, 8-inch and 10-inch mains that create greater head loss, and thus a lower hydraulic grade line. The second part of the problem is the size of the Industrial Park Tank (1,000,000 gallons) relative to the demand in the corresponding region. The majority of Zone 1 is supplied by the Linton Hill Tank, and the demand in Zone 1 supplied by the Industrial Park Tank is not high enough to warrant a one-million gallon tank. This is not an uncommon occurrence because it is difficult to anticipate the water usage from a fully developed industrial park. Design of such tanks tends to project large demands, and thus large tanks are the result.

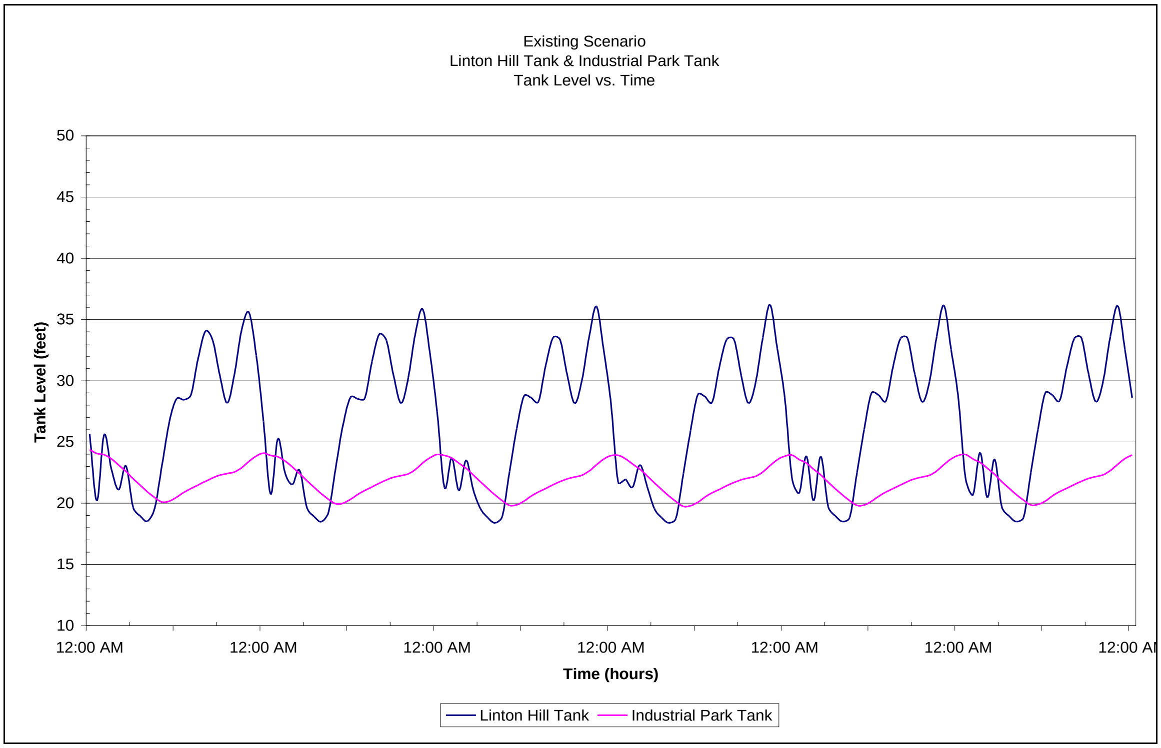

Figure 23

The Linton Hill Tank is equipped with an altitude valve, which would allow the tank to close when full and prevent overflowing. In addition, it would allow the high service pump to bypass the tank and fill the Industrial Park Tank while serving the system. However, in the past PW&W has experienced line bursts throughout the distribution system when the altitude valve has been in operation, due to increases in system pressures. For this reason, PW&W does not utilize the altitude valve at the Linton Hill Tank. Therefore, excessive fluctuations in pressures experienced while using the altitude valve were considered as deficiencies.

Note: While system pressures will increase in Zone 1 when the Linton Hill Tank altitude valve is closed, line bursts could also be a product of inferior water lines, which would be consistent with the age of the system. Therefore, the probability for inferior lines must also be considered as a deficiency during future improvements.

Pressure Deficiency
Any node that possessed a minimum residual pressure below 30 psi at any time over a 144 hour (six day) period under peak demand was identified as deficient. These locations will be discussed in Section 4.2.4.

Flushing Deficiency
Any node in the system that was unable to maintain a minimum residual pressure of 20 psi during a flushing analysis at 2.5 fps was identified as deficient. These locations will be discussed in Section 4.2.5.

Fire-Flow Deficiency
Any node in the system located on a 6-inch (or greater) line that was unable to maintain a minimum residual pressure of 20 psi during a fire-flow of 500 gpm was identified as deficient. These locations will be discussed in Section 4.2.6.

4.2.3 Existing System Storage Results
Refer back to Section 4.2.2 and Figure 23 for existing system storage results.

4.2.4 Existing Pressure Results
Deficient nodes were identified, and are all located at two high-elevation areas in pressure Zone 1. These areas include Hillview Court/Linton Way and Hillview Drive/Skyline Drive. Figure 24 (end of Section 4.2) displays the geographic locations of the deficient nodes while Table 4 illustrates the minimum (and maximum) pressure values of each deficient node, as well as the corresponding elevations and locations.

Existing Scenario - Nodes with Minimum Residual Pressure < 30 psi					
Location	**Junction ID**	**Maximum Pressure (psi)**	**Minimum Pressure (psi)**	**Average Pressure (psi)**	**Elevation (ft)**
Linton Hill Tank	J1088	15.20	7.19	10.95	621.80
Linton Hill Tank	J964	15.42	7.37	11.16	621.30
Skyline Drive	J778	19.68	10.31	15.07	612.10
Elus Drive	J460	21.16	11.77	16.54	608.60
Hillview Drive	J458	24.92	15.54	20.31	600.00
Hillview Drive	J456	26.52	17.14	21.90	596.30
Skyline Drive	J462	27.83	18.46	23.22	593.30
Linton Hill Tank	J800	27.94	19.98	23.67	592.50
Hillview Court	J190	33.64	24.33	28.96	580.80
Hillview Court	J1110	33.64	25.72	29.33	579.50

Table 4

In order to confirm that these nodes are deficient, H&D performed a hydraulic calculation to identify a specific elevation at which the low pressure system could not maintain 30 psi. Based on historical data, a worse-case scenario for the water level in the Linton Hill Tank would be 20 feet (640 feet above MSL), while the lowest pressures in the system occur when the high service pumps are off (static conditions). Pressure can be converted into feet of head (2.31 feet equals 1.0 psi), thus 30 psi is equivalent to 69.3 feet.

Considering the minimum tank elevation of 640 feet, subtracting 69.3 feet from 640 feet results in an elevation of 570.7 feet, which is the maximum elevation the Linton Hill Tank (and Industrial Park Tank) can serve while maintaining 30 psi. In other words, under these static conditions, any node in Zone 1 with an elevation greater than 570.7 feet will be unable to maintain 30 psi of pressure at all times. There are ten such nodes in Zone 1, the same nodes the hydraulic model identified as deficient (refer to Table 4).

4.2.5 Existing Flushing Results

As mentioned earlier, the Kentucky Division of Water mandates that any new development must be capable of flushing a water line at 2.5 fps while maintaining a minimum residual pressure of 20 psi throughout the entire distribution system. By flushing each individual dead-end junction at 2.5 fps, Hethcoat & Davis, Inc. was able to identify fourteen nodes with a pressure below 20 psi during the time of flushing of particular nodes. These results are illustrated in Table 5.

Existing Scenario - Junctions with Pressures < 20 psi at Hour 77 (5:00 am) after Flushing Analyses				
Location of Flushed Junction	**Junction Flushed (flow added based on 2.5 ft/sec)**	**Deficient Junctions (pressure < 20 psi during flushing)**	**Pressure at Deficient Junction (psi)**	**Pipe Size at Deficient Junction (diameter - inches)**
Beckner Lane	J552	J556	9.05	8
Beckner Lane	J552	J1008	13.69	6
Beckner Lane	J552	J1010	2.64	6
Beckner Lane	J552	J1012	16.72	6
Marion Road	J938	J556	13.89	8
Marion Road	J938	J1008	18.87	6
Marion Road	J938	J1010	7.82	6
Highway 293 North	J968	J740	18.15	8
Highway 293 North	J968	J756	18.88	8
Highway 293 North	J968	J1090	16.78	8
Beckner Lane	J1008	J1010	19.33	6
Beckner Lane	J1010	J1010	18.25	6
--	all	J456*	<20	3
--	all	J458*	<20	6

--	all	J460*	<20	6
--	all	J462*	<20	6
--	all	J778*	<20	6
--	all	J964*	<20	12
--	all	J1088*	<20	16

Table 5

Seven of these nodes (identified by an asterick) were previously identified as not being able to maintain 30 psi minimum residual pressure under peak demand conditions (refer to Table 4). Therefore, it is not surprising that the nodes could not maintain 20 psi during flushing analyses.

Note: PW&W has several automatic flushing hydrants at key locations that allow flushing at strategic times when demand is low.

4.2.6 Existing Fire-Flow Results

As mentioned earlier, the Kentucky Division of Water mandates that all hydrants used for fire protection must be capable of delivering 500 gpm for two hours while maintaining a minimum residual pressure of 20 psi throughout the system. However, PW&W requested that 1,000 gpm be available while meeting the same pressure and duration constraints. Therefore, 1,000 gpm of fire-flow was individually applied to 397 nodes, which accounts for all nodes on pipes 6-inches and greater, to determine system fire-flow deficiencies. Upon analysis, it was determined that several locations were unable to meet fire-flow requirements of 1,000 gpm. These deficient areas are shown on Figure 25 (end of Section 4.2).

In total, it was determined that 49 nodes were unable to supply 500 gpm, 104 nodes could supply 500 gpm to 1,000 gpm, and 244 nodes were able to supply 1,000 gpm or greater, all while maintaining 20 psi throughout the system. Table 6 reflects these results.

Amount of Fire-Flow Nodes in Specified Flow Ranges for Existing Scenario							
Scenario	**<500 gpm**	**% Total**	**≥500 gpm, <1,000 gpm**	**% Total**	**≥1,000 gpm**	**% Total**	**Total Nodes**
Existing	49	12.3	104	26.2	244	61.5	397

Table 6

4.3 Priority No. 1 (5-Year Improvements)

Princeton Water & Wastewater is not anticipating a significant increase of demand in the near future. Therefore, the focus of future improvements is to provide the best service as possible to the existing system. All improvements involve correcting/minimizing the current system deficiencies related to system storage, residual pressures, flushing and fire-flows. By further strengthening the distribution system, PW&W will position itself to better serve changes in demand or future growth. Please refer to Figure 26 (end of Section 4.3) for locations of 5-Year Improvements.

4.3.1 Recommended 5-Year Improvements

System Storage Improvements (Group 5A)

(1) Based on discussions with PW&W, the inability of the Industrial Park Tank to adequately cycle is a major concern, and thus improving the tank's ability to cycle was included as an immediate improvement. In order to address this issue, PW&W requested that a booster pump be inserted into the model to help fill the Industrial Park Tank. The pump used was rated at 350 gpm and 35 feet TDH. The intent of the pump is to increase the Tank's maximum water level, which cannot be reached based on system hydraulics alone. Once the water in the tank reaches this desired level, the pump will turn OFF, and the tank level will cycle down towards the level at which it had been operating without the pump. Once the water drops to a specified minimum level, the pump will turn ON, and the tank would once again fill. For modeling purposes, minimum and maximum tank levels of 30 ft and 38 ft were used for pump controls.

Pressure Improvements (Group 5B)

The improvements identified herein (*Group 5B*) are intended to strengthen the network where it has shown a tendency for weakness.

(1) As discussed earlier, a new Super Wal-Mart is under construction with its corresponding 8-inch and 10-inch water lines. As these new lines were not included in the existing scenario, they were placed in the 5-Year Improvements scenario as future projects.

(2) PW&W provided H&D with five locations where they would like to install new 6-inch lines to replace existing 2-inch and 4-inch lines and to loop new and existing lines back into the system. These locations include Robin Road, South Darby Street from West Main Street to Bell Street, Cherry Street from Akers Avenue to Rose Avenue, North Harrison Street from Green Street to Market Street, and Oak Drive from North Jefferson Street to East Sandra Drive.

As noted in Section 4.2.4, the hydraulic model identified two areas of high elevations where minimum residual pressures below 30 psi were observed. Both of these areas are located in the Low Service zone, which is served by the Linton Hill Tank (and Industrial Park Tank). Without increasing the height of the Linton Hill Tank, these areas cannot be adequately served by the Low Service zone. Therefore, to improve residual pressures at these two areas, they must be switched over to the High Service zone (Zone 2), which can serve areas of higher elevations.

(3) The first area requiring improvements is located near the Linton Hill Tank on the north end of Hillview Court. H&D recommends installing a new 6-inch line from the existing 12-inch line on Linton Way on the discharge (north) side of the existing booster pump. The new 6-inch line would parallel the existing 12-inch line running south towards the Linton Hill Tank, then parallel the existing 16-inch line running southeast towards Hillview Court, where it would connect to the existing 6-inch line at the northwest end of Hillview Court. In addition, the existing 6-inch line extending south to Green Street must be valved off from the Low Pressure Zone west of its connection to the existing 4-inch line at North Darby Street. The entire 6" line on Hillview Court is then switched over into the High Service zone, as its elevations dictate.

(4) Focus for residual pressure improvements shifted to the other area of low pressures located along Skyline Drive and Hillview Drive. This area crosses over into the High Service zone (Zone 2), but it is served by Zone 1. As long as this area is served by Zone 1, it will be unable to maintain the 30 psi of pressure at worst-case conditions (refer to Section 4.2.4). However, if it is switched over to Zone 2, the desired 30 psi can be maintained at all times. The system in this area consists of a 6-inch line moving north along Skyline Drive from the 8-inch line on North Jefferson Street to Hillview Drive. At Hillview Drive, the line splits to a 2-inch line to the west ending on Ellis Drive and to a 3-inch line to the east ending just west of North Jefferson Street. To increase pressures in this area, a 6-inch line was added to replace the existing 3-inch line, and extended to the existing 10-inch main on North Jefferson Street. Additionally, the valve that currently separates Zone 1 from Zone 2, which is located north of the 6-inch/8-inch line connection at the intersection of Skyline Drive and North Jefferson Street, was changed from closed to opened. Similarly, the nearest valve south of the same connection was changed from opened to closed. With the addition of this new 6-inch line and the reconfiguration of the two valves, the area along Skyline Drive and Hillview Drive has been switched from the Low Service zone to the High Service zone, as its elevations dictate.

Flushing Improvements (Group 5C)
No improvements were made solely on the basis of flushing as improved flushing is a by-product of increased pressure and fire-flow improvements.

Fire-Flow Improvements (Group 5D)
(1) As noted in Table 6 and Figure 25 (Section 4.2), the fire-flow analysis conducted in the Existing scenario identified several areas where fire-flow could not be achieved. One such area lies on the 6-inch line that extends east from Deerfield Drive across to Cadiz Road, then dead-ends past Meadow Lane on Martins Drive. In order to address this deficiency, a new 8-inch line was inserted into the model along Cadiz Road from the existing 6-inch at Meadow Lane north to an existing 8-inch line on Cadiz Road. With the addition of this line, a hydraulic loop was created which increased fire-flow availability in the area. In addition, the probability for stagnant water at the dead-end of such a long line is removed.

(2) A second area where adequate fire-flow could not be achieved is located along Highway 91 North near Beckner Lane. As adequate residual pressures are maintained at this location under normal operating conditions, H&D concluded that the deficiency was a result of pipe size. The existing 8-inch line on Highway 91 North could not provide enough water to accommodate desired fire-flow. For this reason, H&D recommends that a new 12-inch line be installed along Highway 91 North from Linton Way to Education Drive. Additionally, H&D recommends that a new 8-inch line be installed from the end of the new 12-inch line at Education Drive to an existing 6-inch line running south from Beckner Lane. The additions of these two lines will allow more water to reach the areas with deficient fire-flow availability, as well as remove a dead-end line from the system.

(3) A third area where adequate fire-flow could not be achieved is located at the intersection of Old Connector Road and Highway 62 West, at the dead-end of a 6-inch line. Once again, adequate residual pressures are maintained in this area, so the fire-flow deficiency is a result of pipe size. For this reason, H&D recommends that a new 8-inch line be installed along Highway 62 West from the existing 8-inch line at Ethridge Drive to the existing 6-inch line at Old Connector Road. This improvement will provide more water to the deficient area, as well as remove a dead-end line from the system.

4.3.2 System Storage Results

It should be noted that while the new booster pump may improve the behavior of the Industrial Park Tank, there is some concern as to the behavior of the Linton Hill Tank in response to the booster pump. Figure 27 illustrates the resulting water levels of both tanks after the 5-Year Improvements were incorporated into the model. As the graph depicts, the Linton Hill Tank is not filling as it was prior to installation of the booster pump and other improvements. This most likely results from the booster pump pulling water from the system that would normally help to fill the Linton Hill Tank. In other words, if the same amount of water is entering the system as before, and more water is entering the Industrial Park Tank, then less water will be entering the Linton Hill Tank.

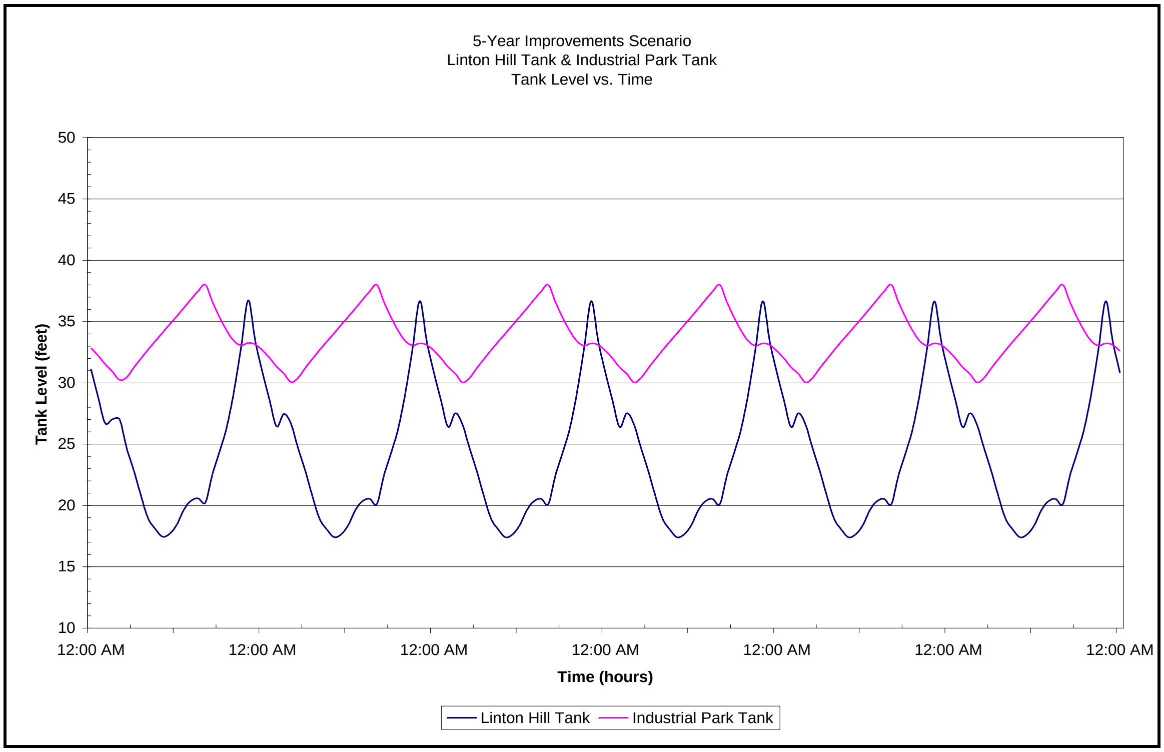

Figure 27

The intent of the booster pump is to force the Industrial Park Tank to cycle, and thus maintain chlorine residual levels. If the decreased level of the Linton Hill Tank with the addition of the booster pump concerns PW&W, then H&D would not recommend using a booster pump. Instead, H&D would recommend installing a mixing system at the Industrial Park Tank. A tank mixing system can improve water quality by creating a continuous vertical circulation within the tank. Without a mixing system, water inside a tank with inefficient turnover will stratify based on its temperature, as the warmest water will remain at the top level and the coldest water will remain at the bottom. The possibility exists in the Industrial Park Tank that the top (warmest) levels of water never exit the tank. The inflow is colder and enters the Tank at the bottom. Likewise, the outlet is at the bottom of the Tank, and only the bottom levels of water exit the Tank. Because this action continually repeats, the water quality in the system is not bad. Even though the Tank only cycles two to four feet each day, the system is drawing the freshest water from the Tank. The concern, however, is when a major demand occurs, such as a fire or a line break. In these cases, the water that had previously been confined to the Tank can quickly be drawn into the system. The very real possibility exists that this water will not have adequate chlorine residual and may be unsafe.

Installation of a tank mixing system would create a continuous rolling effect in the Industrial Park Tank, bringing the bottom levels of water to the top, and the top levels of water to the bottom (see illustration on Page 33). The stratification of water is eliminated, and water quality within the Tank improves.

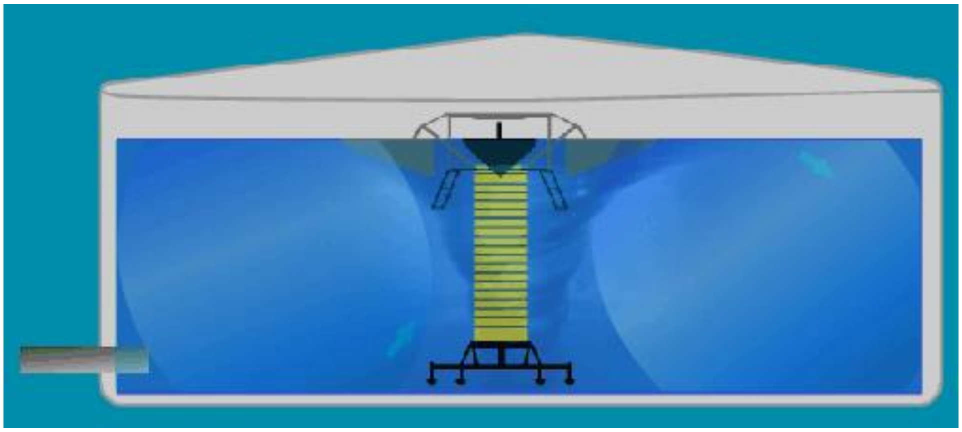

Illustration of Tank Mixing System courtesy of SolarBee, Inc.

It may be necessary for chlorine to be added with the mixer. An adequate chlorine residual should be maintainable with a six to seven gallon per day (gpd) additive of sodium hypochlorite. A Stenner SVPIH7 Variable Speed Pump with a discharge pressure up to 100 psi and a feed rate of 40 gpd could effectively deliver the hypochlorite to the Tank in a four hour period. Several mixing systems are available and have been designed specifically for this application. If a mixing system is desired, PW&W should compare the benefits of various systems before purchase.

Note: Tank graphs in the 5-Year and 10-Year Improvements scenarios depict tank behaviors influenced by a booster pump station. If a tank mixing system is utilized instead of a pump station, tank levels would be similar to levels at existing conditions.

4.3.3 Pressure Results

With the addition of the 5-Year Improvements, only four nodes remain that do not maintain a minimum residual pressure of 30 psi, as Table 7 indicates. **Note: Deficient junctions are shown in bold.**

5-Year Improvements Scenario - Nodes with Minimum Residual Pressure < 30 psi					
Location	Junction ID	Maximum Pressure (psi)	Minimum Pressure (psi)	Average Pressure (psi)	Elevation (ft)
Linton Hill Tank	**J1088**	**15.22**	**5.64**	**9.37**	**621.80**
Linton Hill Tank	**J964**	**15.44**	**5.85**	**9.57**	**621.30**
Skyline Drive	J778	52.45	49.25	50.84	612.10
Elus Drive	J460	53.94	50.72	52.31	608.60
Hillview Drive	J458	57.70	54.50	56.08	600.00
Hillview Drive	J456	59.31	56.10	57.69	596.30

Skyline Drive	J462	60.60	57.40	58.99	593.30
Linton Hill Tank	**J800**	**27.87**	**18.35**	**22.08**	**592.50**
Hillview Court	J190	70.57	64.32	67.21	580.80
Hillview Court	**J1110**	**33.32**	**24.13**	**27.83**	**579.50**

Table 7

All four remaining deficient nodes are located near the Linton Hill Tank (see Figure 28, end of Section 4.3), and hydraulics based on elevations dictate that 30 psi cannot always be achieved in this area when served by the Low Pressure zone.

4.3.4 Flushing Results

After flushing each individual dead-end junction which was identified as deficient in the Existing scenario (refer to Table 5, Section 4.2.5), Hethcoat & Davis, Inc. was able to identify four nodes with a pressure below 20 psi in the 5-Year Improvements scenario. These results are illustrated in Table 8.

5-Year Improvements Scenario - Junctions with Pressures < 20 psi at Hour 77 (5:00 am) after Flushing Analyses				
Location of Flushed Junction	**Junction Flushed (flow added based on 2.5 ft/sec)**	**Deficient Junctions (pressure < 20 psi during flushing)**	**Pressure at Deficient Junction (psi)**	**Pipe Size at Deficient Junction (diameter - inches)**
Highway 293 North	J968	J740	19.29	8
Highway 293 North	J968	J1090	17.92	8
--	all	J964*	<20	12
--	all	J1088*	<20	16

Table 8

Two of these nodes (identified by an asterick) were previously identified as not being able to maintain 30 psi minimum domestic pressure after the 5-Year Improvements (refer to Table 7). Therefore, it is not surprising that the nodes could not maintain 20 psi during flushing analyses. The remaining two deficient nodes experienced a pressure below 20 psi when node 968 was flushed. This flushed node is located at the northern-most end of the 8-inch line on Highway 293 North.

4.3.5 Fire-Flow Results

Additional fire-flow nodes were added with the 5-Year Improvements, and the total number of fire-flow nodes increased from 397 to 411. Upon analysis, it was determined that several locations were unable to meet fire-flow requirements of 500 gpm. These deficient areas are shown on Figure 29 (end of Section 4.3).

In total, of the 411 nodes to which fire-flow was applied, 17 nodes were unable to supply 500 gpm, 109 nodes could supply 500 gpm to 1,000 gpm, and 285 nodes were able to supply 1,000 gpm or greater, all while maintaining 20 psi throughout the system. Table 9 illustrates the improved system fire-flow capabilities.

Amount of Fire-Flow Nodes in Specified Flow Ranges for 5-Year Improvements Scenario							
Scenario	**<500 gpm**	**% Total**	**≥500 gpm, <1,000 gpm**	**% Total**	**≥1,000 gpm**	**% Total**	**Total Nodes**
Existing	49	12.3	104	26.2	244	61.5	397
5 Year	**17**	**4.1**	**109**	**26.5**	**285**	**69.4**	**411**

Table 9

It should be noted that following the 5-Year Improvements, approximately 96 percent of fire-flow nodes (nodes on lines with diameters of 6-inches or greater) appear to be capable of delivering 500 gpm (or greater) while maintaining minimum system pressures of 20 psi. The areas most improved are located (a) on Cadiz Road near Meadow Lane, which is the location of the 8-inch line extension, (b) the area on Hillview Court which was switched over to the High Service zone, (c) the area near Beckner Lane, which reflects the addition of the 12-inch and 8-inch lines along Highway 91 North and Education Drive towards Beckner Lane, and (d) the western-most area on Highway 62 West, which is the location of an 8-inch line extension.

4.3.6 Projected Cost Estimate

Refer to Table 10 (end of Section 4.3) for projected costs of 5-Year Improvements.

4.4 Priority No. 2 (10-Year Improvements)

Please refer to Figure 30 (end of Section 4.4) for locations of 10-Year Improvements.

4.4.1 Recommended 10-Year Improvements

System Storage Improvements (Group 10A)
No improvements were made solely on the basis of system storage. However, improvements associated with fire-flow availability will have a positive impact on system storage.

Pressure Improvements (Group 10B)
No improvements were made solely on the basis of pressures. However, improvements associated with fire-flow availability will have a positive impact on pressures.

Flushing Improvements (Group 10C)
No improvements were made solely on the basis of flushing as improved flushing is a by-product of increased pressure and fire-flow improvements.

Fire-Flow Improvements (Group 10D)
(1) The PW&W water system has a significant amount of 2-inch, 3-inch and 4-inch lines in the interior of the system. Based on State regulations, a fire hydrant can only be located on a line with a diameter of 6-inches or greater. Therefore, in order to increase the available locations for fire hydrants in the interior of the system, H&D is recommending that all 2-inch, 3-inch and 4-inch lines around town which are not dead-end lines be replaced with new 6-inch lines. This line rehabilitation will not only increase fire protection, it will remove old and deficient lines which may be contributing to the line bursts and some of the water loss that the PW&W system has experienced.

In addition to increasing fire protection and reducing water loss, there could be another benefit to replacing the smaller lines. The 2-inch, 3-inch and 4-inch lines, with the exception of those that are dead-ends, transport water to and from larger-sized mains in the system. This transition from a larger line to a smaller line back to a larger line creates a hydraulic “bottleneck” in the system. This “bottleneck” is caused by increased friction losses of the water through the smaller pipes, which results in lower pressures downstream. As mentioned in Section 4.2.2, the inability of the Industrial Park Tank to cycle properly could, in part, be caused by additional friction losses experienced across the system from the 16-inch transmission main to the Tank. By replacing the 2-inch, 3-inch and 4-inch lines with 6-inch lines, these friction losses would be reduced, which could improve the Industrial Park Tank behavior.

4.4.2 System Storage Results

Figure 31 illustrates the resulting water levels of both tanks after the 10-Year Improvements were incorporated into the model. Tank behaviors have not changed from that of the 5-Year Improvements, which is expected as the focus of the 10-Year Improvements was improving fire-flow capabilities. As stated earlier, if the decreased level of the Linton Hill Tank concerns PW&W, then H&D would recommend installing a mixing system at the Industrial Park Tank instead of a booster pump.

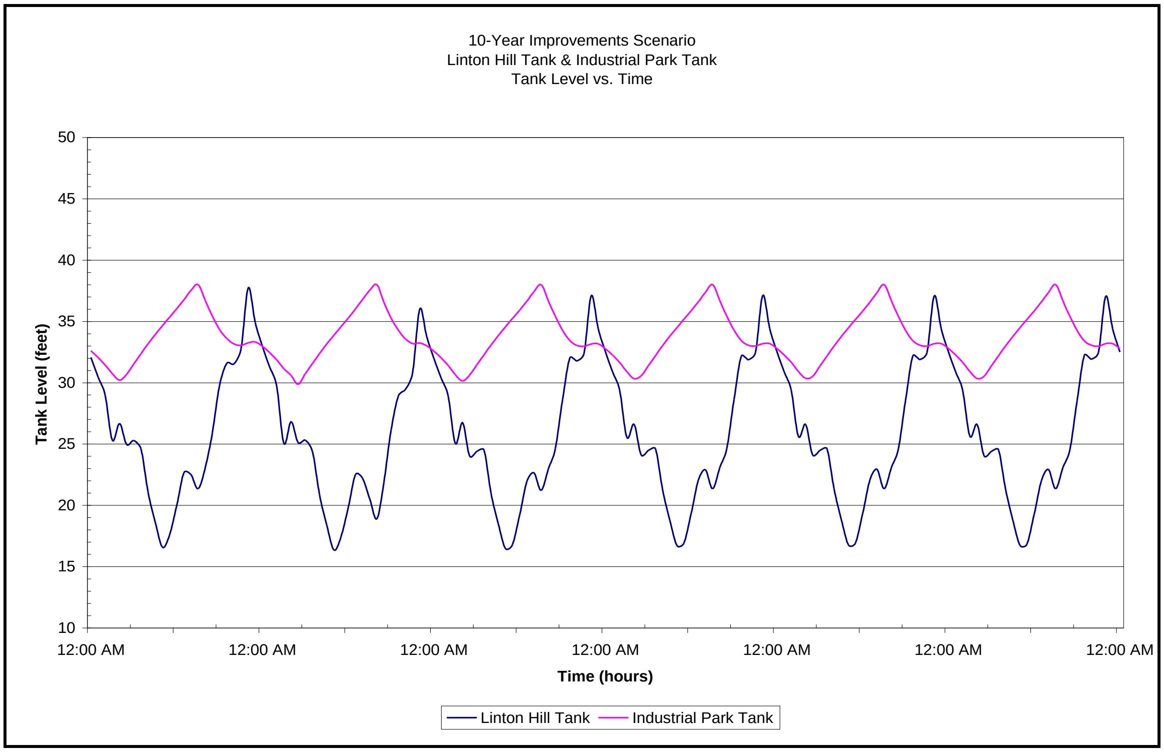

Figure 31

4.4.3 Pressure Results

Table 11 depicts the four nodes which are unable to maintain the minimum residual pressure of 30 psi. **Note: Deficient junctions are shown in bold.**

10-Year Improvements Scenario - Nodes with Minimum Residual Pressure < 30 psi					
Location	Junction ID	Maximum Pressure (psi)	Minimum Pressure (psi)	Average Pressure (psi)	Elevation (ft)
Linton Hill Tank	**J1088**	**15.57**	**5.23**	**10.10**	**621.80**
Linton Hill Tank	**J964**	**15.79**	**5.45**	**10.31**	**621.30**
Skyline Drive	J778	52.51	49.31	50.88	612.10
Elus Drive	J460	53.99	50.78	52.35	608.60
Hillview Drive	J458	57.76	54.55	56.12	600.00
Hillview Drive	J456	59.36	56.16	57.72	596.30

Skyline Drive	J462	60.66	57.46	59.03	593.30
Linton Hill Tank	**J800**	**28.20**	**17.94**	**22.83**	**592.50**
Hillview Court	J190	71.53	64.41	67.30	580.80
Hillview Court	**J1110**	**33.61**	**23.68**	**28.58**	**579.50**

Table 11

As before, all of these nodes are located near the Linton Hill Tank (see Figure 32, end of Section 4.4), and hydraulics based on elevations dictate that 30 psi cannot always be achieved.

4.4.4 Flushing Results

After flushing each individual dead-end junction which was identified as deficient in the Existing scenario (refer to Table 5, Section 4.2.5), Hethcoat & Davis, Inc. was able to identify four nodes with a pressure below 20 psi in the 10-Year Improvements Scenario. These results are illustrated in Table 12.

10-Year Improvements Scenario - Junctions with Pressures < 20 psi at Hour 77 (5:00 am) after Flushing Analyses				
Location of Flushed Junction	**Junction Flushed (flow added based on 2.5 ft/sec)**	**Deficient Junctions (pressure < 20 psi during flushing)**	**Pressure at Deficient Junction (psi)**	**Pipe Size at Deficient Junction (diameter - inches)**
Highway 293 North	J968	J740	19.44	8
Highway 293 North	J968	J1090	18.07	8
--	all	J964*	<20	12
--	all	J1088*	<20	16

Table 12

Two of these nodes (identified by an asterick) were previously identified as not being able to maintain 30 psi minimum domestic pressure after the 10-Year Improvements (refer to Table 11). Therefore, it is not surprising that the nodes could not maintain 20 psi during flushing analyses. The remaining two deficient nodes experienced a pressure below 20 psi when node 968 was flushed. This flushed node is located at the northern-most end of the 8-inch line on Highway 293 North.

4.4.5 Fire-Flow Results

Additional fire-flow nodes were added with the 10-Year Improvements, and the total number of fire-flow nodes available increased from 411 to 454. Upon analysis, it was determined that several locations were unable to meet fire-flow requirements of 500 gpm. These deficient areas are shown on Figure 33 (end of Section 4.4).

In total, of the 454 nodes to which fire-flow was applied, 15 nodes were unable to supply 500 gpm, 102 nodes could supply 500 gpm to 1,000 gpm, and 337 nodes were able to supply 1,000 gpm or greater, all while maintaining 20 psi throughout the system. Table 13 illustrates the improved system fire-flow capabilities.

Amount of Fire-Flow Nodes in Specified Flow Ranges for 10-Year Improvements Scenario							
Scenario	**<500 gpm**	**% Total**	**≥500 gpm, <1,000 gpm**	**% Total**	**≥1,000 gpm**	**% Total**	**Total Nodes**
Existing	49	12.3	104	26.2	244	61.5	397
5 Year	17	4.1	109	26.5	285	69.4	411
10 Year	**15**	**3.3**	**102**	**22.5**	**337**	**74.2**	**454**

Table 13

It should be noted that following the 10-Year Improvements, approximately 97 percent of fire-flow nodes appear to be capable of delivering 500 gpm (or greater) while maintaining minimum system pressures of 20 psi. The area most improved is located downtown at the locations of line replacements.

4.4.6 Projected Cost Estimate

Refer to Table 14 (end of Section 4.4) for projected costs of 10-Year Improvements.

4.5 Priority No. 3 (15-Year Improvements)

Please refer to Figure 34 (end of Section 4.5) for locations of 15-Year Improvements.

4.5.1 Recommended Improvements

System Storage Improvements (Group 15A)

(1) The friction loss experienced from the 16-inch main to the Industrial Park Tank is much greater than that experienced by the Linton Hill Tank, thus the tanks do not behave similarly. For this reason, H&D recommends that a new 16-inch transmission line be installed from the existing 16-inch main to the existing 10-inch main on Highway 62 West. The route for the new line would run west from the existing 16-inch line along Maple Avenue, Varmint Trace Road and Highway 62 West, where it would connect to the existing 10-inch line at Park Avenue. With the addition of this line, system head loss experienced across the system would be greatly reduced, and the hydraulic grade lines of the Industrial Park Tank would become increasingly similar to that of the Linton Hill Tank.

(2) The Linton Hill Tank is equipped with an altitude valve which is not in operation, as PW&W has experienced line bursts in the system while it has been used in the past. However, with the addition of the 16-inch line, pressure fluctuations previously encountered by PW&W should not be as significant, as head loss across the system has been reduced. In addition, with the 10-Year Improvements, the majority of 2-inch, 3-inch and 4-inch lines in the system have been upgraded to 6-inch, and it is believed that any deficient lines which may have contributed to these line bursts will have been eliminated with the upgrades. In conjunction with the new 16-inch line, and with the possibility of line bursts greatly reduced, H&D recommends once again utilizing the altitude valve at the Linton Hill Tank.

(3) Additionally, the high service pump controls should be manipulated to react based on the Industrial Park Tank levels. By reconfiguring the high service pump controls and utilizing the altitude valve, the high service pumps should initially fill the Linton Hill Tank. Once full, the altitude valve would close, and the high service pump would continue to fill the Industrial Park Tank until it is full, at which point the pump would turn OFF. The pump would stay OFF until the Industrial Park Tank dropped to a specified level. For modeling purposes, minimum and maximum tank levels of 35 ft and 40 ft were used for pump controls. **Note: Prior to these improvements, the booster pump at the Industrial Park Tank would have to be removed from the system.**

Pressure Improvements (Group 15B)

No improvements were made solely on the basis of pressures. However, improvements associated with system storage will have a positive impact on pressures.

Flushing Improvements (Group 15C)

No improvements were made solely on the basis of flushing as improved flushing is a by-product of increased pressure and fire-flow improvements.

Fire-Flow Improvements (Group 15D)

No improvements were made solely on the basis of fire-flow availability. However, improvements associated with system storage will have a positive impact on fire-flow.

4.5.2 System Storage Results

With the addition of the 15-Year Improvements, the Industrial Park Tank levels would begin to behave more closely to the Linton Hill Tank levels. Figure 35 illustrates the corresponding level behaviors of both tanks.

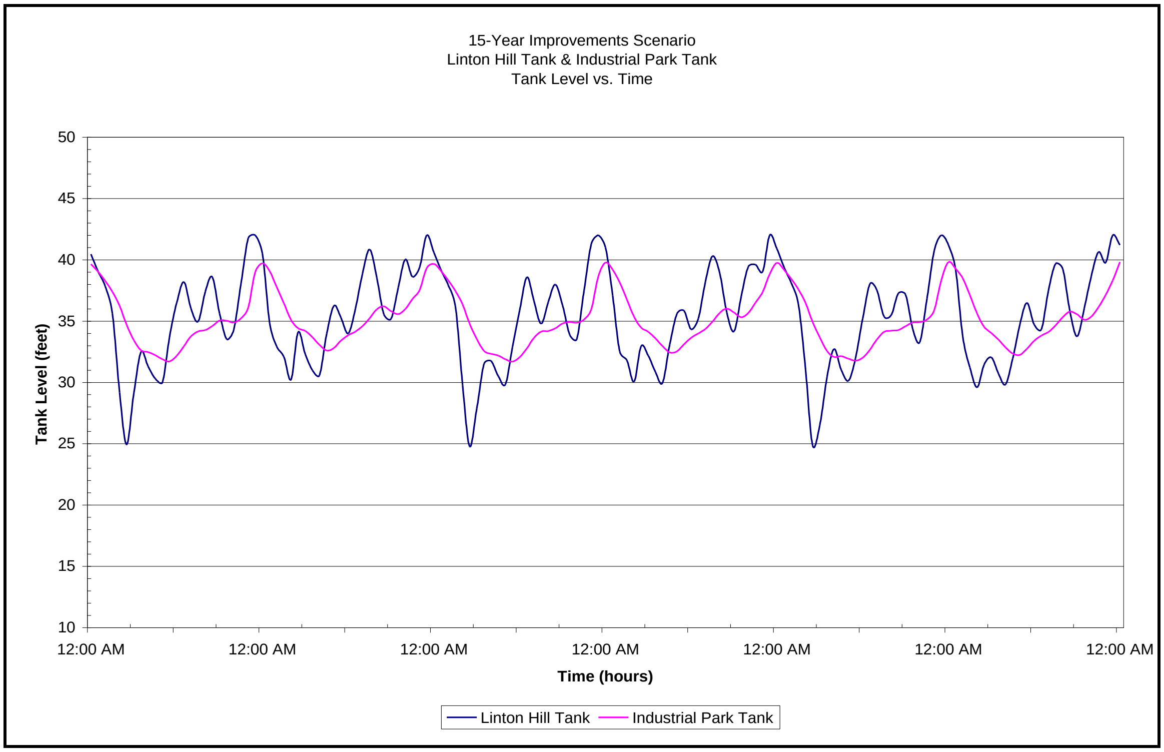

Figure 35

4.5.3 Pressure Results

Table 15 depicts the four nodes which are unable to maintain the minimum residual pressure of 30 psi. **Note: Deficient junctions are shown in bold.**

15-Year Improvements Scenario - Nodes with Minimum Residual Pressure < 30 psi					
Location	Junction ID	Maximum Pressure (psi)	Minimum Pressure (psi)	Average Pressure (psi)	Elevation (ft)
Linton Hill Tank	**J1088**	**33.81**	**9.48**	**15.08**	**621.80**
Linton Hill Tank	**J964**	**34.03**	**9.66**	**15.28**	**621.30**

Skyline Drive	J778	52.57	49.43	50.88	612.10
Elus Drive	J460	54.05	50.90	52.35	608.60
Hillview Drive	J458	57.82	54.67	56.12	600.00
Hillview Drive	J456	59.42	56.28	57.72	596.30
Skyline Drive	J462	60.72	57.58	59.02	593.30
Linton Hill Tank	**J800**	**46.51**	**22.11**	**27.82**	**592.50**
Hillview Court	J190	72.51	64.37	67.58	580.80
Hillview Court	**J1110**	**52.15**	**27.48**	**33.62**	**579.50**

Table 15

As before, all of these nodes are located near the Linton Hill Tank (see Figure 36, end of Section 4.5), and hydraulics dictate that 30 psi cannot always be achieved.

Note: Upon examining results in the central areas of town (locations of past line bursts), the maximum pressure experienced after the 15-Year Improvements is approximately 113 psi. In addition, the maximum pressure fluctuation is approximately 30 psi. With the 6-inch line upgrades (10-Year Improvements) and the addition of the 16-inch line, the probability of future line bursts in this area is greatly reduced. If PW&W does experience line bursts in the future, it will be a result of deficient lines (old lines not replaced in 10-Year Improvements), and not pressure fluctuations. H&D would recommend replacing these deficient lines as needed in the future.

4.5.4 Flushing Results

After flushing each individual dead-end junction which was identified as deficient in the Existing scenario (refer to Table 5, Section 4.2.5), Hethcoat & Davis, Inc. was able to identify five nodes with a pressure below 20 psi in the 15-Year Improvements Scenario. These results are illustrated in Table 16.

15-Year Improvements Scenario - Junctions with Pressures < 20 psi at Hour 77 (5:00 am) after Flushing Analyses				
Location of Flushed Junction	**Junction Flushed (flow added based on 2.5 ft/sec)**	**Deficient Junctions (pressure < 20 psi during flushing)**	**Pressure at Deficient Junction (psi)**	**Pipe Size at Deficient Junction (diameter - inches)**
Highway 293 North	J968	J740	19.16	8
Highway 293 North	J968	J756	19.90	8
Highway 293 North	J968	J1090	17.79	8

--	all	J964*	<20	12
--	all	J1088*	<20	16

Table 16

Two of these nodes (identified by an asterick) were previously identified as not being able to maintain 30 psi minimum domestic pressure after the 15-Year Improvements (refer to Table 15). Therefore, it is not surprising that the nodes could not maintain 20 psi during flushing analyses. The remaining three deficient nodes experienced a pressure below 20 psi when Node 968 was flushed. This flushed node is located at the northern-most end of the 8-inch line on Highway 293 North. It should be noted that Node 756 did not appear as a deficient node in either the 5-Year Improvements or 10-Year Improvements flushing results. However, upon examining the results of these previous scenarios, Node 756 had a pressure only slightly above 20 psi. Therefore, the drop in pressure experienced in the 15-Year Improvements is not a concern.

4.5.5 Fire Flow Results

Additional fire-flow nodes were added with the 15-Year Improvements, and the total number of fire-flow nodes increased from 454 to 456. Upon analysis, it was determined that several locations were unable to meet fire-flow requirements of 500 gpm. These deficient areas are shown on Figure 37 (end of Section 4.5).

In total, of the 456 nodes to which fire-flow was applied, 13 nodes were unable to supply 500 gpm, 103 nodes could supply 500 gpm to 1,000 gpm, and 340 nodes were able to supply the 1,000 gpm or greater, all while maintaining 20 psi throughout the system. Table 17 illustrates the improved system fire-flow capabilities.

Amount of Fire-Flow Nodes in Specified Flow Ranges for 15-Year Improvements Scenario							
Scenario	**<500 gpm**	**% Total**	**≥500 gpm, <1,000 gpm**	**% Total**	**≥1,000 gpm**	**% Total**	**Total Nodes**
Existing	49	12.3	104	26.2	244	61.5	397
5 Year	17	4.1	109	26.5	285	69.4	411
10 Year	15	3.3	102	22.5	337	74.2	454
15 Year	**13**	**2.9**	**103**	**22.6**	**340**	**74.5**	**456**

Table 17

It should be noted that following the 15-Year Improvements, approximately 97 percent of fire-flow nodes appear to be capable of delivering 500 gpm (or greater) while maintaining minimum pressures of 20 psi. Considering that these improvements were focused on the Industrial Park Tank behavior, the minimal increase in fire-flow availability is not surprising.

4.5.6 Projected Cost Estimate

Refer to Table 18 (end of Section 4.5) for projected costs of 15-Year Improvements.

4.6 Priority No. 4 (20-Year Improvements)

Please refer to Figure 38 (end of Section 4.6) for locations of 20-Year Improvements.

4.6.1 Recommended Improvements

System Storage Improvements (Group 20A)
No improvements were made solely on the basis of system storage. However, improvements associated with fire-flow availability and pressure will have a positive impact on system storage.

Pressure Improvements (Group 20B)
(1) Dead-end lines create the possibility for stagnated water and they limit water availability to only one direction. Hydraulically speaking, it is always better to "loop" the line back into the system so that the water can be fed from both directions to those served on the line. For this reason, H&D is recommending that the existing 4-inch dead-end line from Harvey Lane to Varmint Trace Road be replaced with a 6-inch line. This new 6-inch line would tie into the existing 10-inch and 16-inch lines running along the railroad. This rehabilitation would remove a dead-end line from the system, reduce friction losses, and thus increase system pressures and fire-flow availabilities.

(2) There appears to be several lines which cross, but do not connect to, the existing 10-inch transmission line running east-to-west (along railroad). As this line is a major route to transmit water across the system (east to west), the more connections that can feed water into or receive water from the 10-inch line will result in reduced friction losses, and thus increased system pressures and fire-flow availabilities. Therefore, H&D recommends that all lines which cross the existing 10-inch water main on the south side of town be connected to the main. These connections include the 6-inch line on Varmint Trace Road from Eagon Street to Lake Street, the 8-inch line east of South Darby Street and north of Varmint Trace Road, the 16-inch main on South Seminary Street at Cash Drive, and the 6-inch line on South Jefferson Street at Cash Drive.

(3) Upon visual inspection of the water system map, it was noted that the existing 6-inch line on Kentucky Avenue beginning at West Main Street runs southwest along Kentucky and parallels an existing 8-inch line on Highway 62 West. This 6-inch line turns southeast along Baldwin Avenue and connects to a 6-inch line extending from Good Street. It appears that this 6-inch line, which is approximately 3,700 linear feet in length, would be a good route for transmitting water across the system if it were connected to the existing 8-inch line running along Highway 62 West. Therefore, H&D is recommending that a new 8-inch line be installed from the existing 6-inch line at the intersection of Baldwin Avenue and Kentucky Avenue to the existing 8-inch line on Highway 62 West.

Flushing Improvements (Group 20C)
No improvements were made solely on the basis of flushing as improved flushing is a by-product of increased pressure and fire-flow improvements.

Fire-Flow Improvements (Group 20D)
After the additions of the 15-Year Improvements, there remain several areas throughout the system that cannot achieve adequate fire-flow while maintaining minimum pressure (refer to Figure 37, end of Section 4.5). In order to supply adequate fire-flows, larger pipes need to extend into these areas, or additional pipes need to be added. For this reason, H&D recommends the following improvements.

(1) A new 10-inch line on Highway 293 North from the end of the existing 10-inch main running north to the end of the existing 8-inch dead-end line south of Nichols Road.

(2) A new 6-inch line running west from the existing 8-inch line at West Main Street and Marion Road, crossing the Western Kentucky Parkway, paralleling an existing 6-inch line along Old Fredonia Road, and connecting to the existing 6-inch dead-end line.

(3) A new 6-inch line paralleling the existing 6-inch line on Jeff Watson Road from Highway 91 North to Jackson Road, and connecting to the existing 6-inch line on Jackson Road.

(4) A new 6-inch line along Vivian Drive from the existing 8-inch line on Dawson Road to the existing 6-inch line at Willard Drive.

(5) A new 6-inch line along an existing fence line from the existing 6-inch line at the end of Cooper Circle to the existing 6-inch line on North Highland Avenue.

(6) A new 8-inch line on University Drive from Sandlick Road to Highway 91 South.

4.6.2 System Storage Results
As Figure 39 illustrates, the addition of the 20-Year Improvements has no significant affect on the behaviors of the Industrial Park Tank and the Linton Hill Tank. Since the majority of these improvements focused on fire-flow availability near the perimeter of the system, it is not surprising to see the tank behaviors unchanged from the 15-Year Improvements.

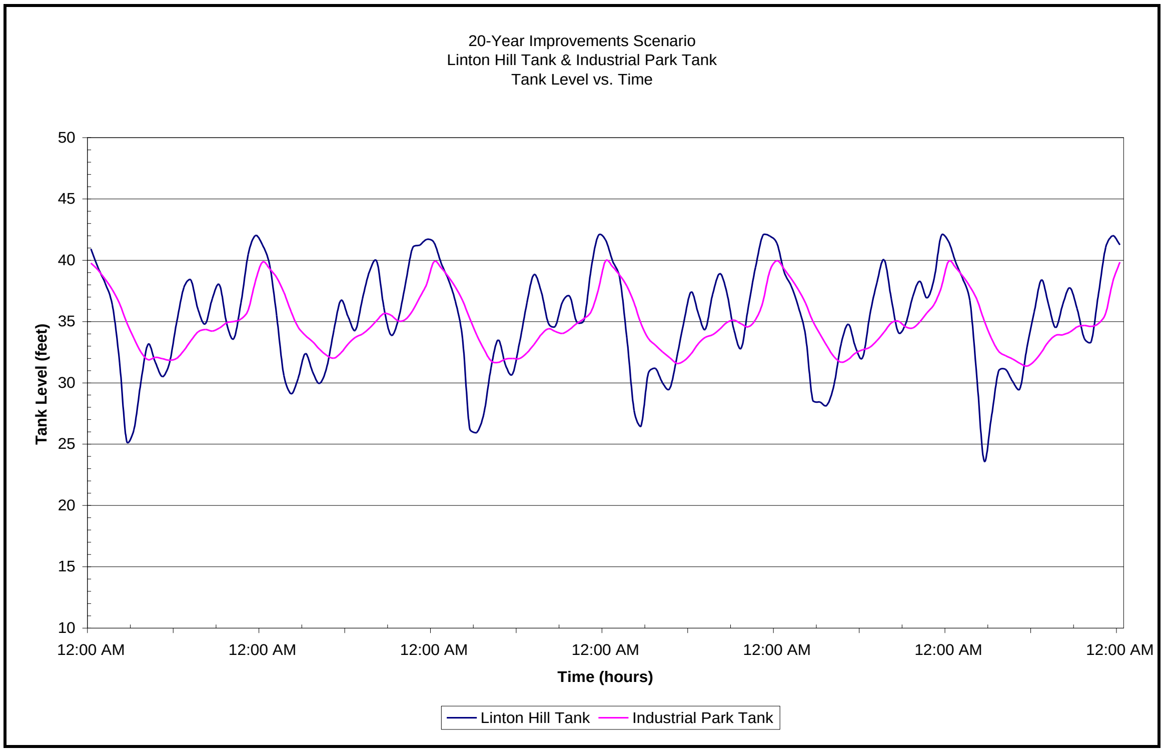

Figure 39

4.6.3 Pressure Results

Table 19 depicts the four nodes which are unable to maintain the minimum residual pressure of 30 psi. **Note: Deficient junctions are shown in bold.**

20-Year Improvements Scenario - Nodes with Minimum Residual Pressure < 30 psi					
Location	**Junction ID**	**Maximum Pressure (psi)**	**Minimum Pressure (psi)**	**Average Pressure (psi)**	**Elevation (ft)**
Linton Hill Tank	**J1088**	**33.68**	**9.45**	**15.08**	**621.80**
Linton Hill Tank	**J964**	**33.89**	**9.64**	**15.28**	**621.30**
Skyline Drive	J778	52.55	49.45	50.88	612.10
Elus Drive	J460	54.03	50.92	52.35	608.60
Hillview Drive	J458	57.80	54.70	56.12	600.00
Hillview Drive	J456	59.40	56.30	57.72	596.30
Skyline Drive	J462	60.70	57.60	59.03	593.30
Linton Hill Tank	**J800**	**46.37**	**22.08**	**27.81**	**592.50**
Hillview Court	J190	72.69	64.53	67.56	580.80

Hillview Court	**J1110**	**52.01**	**27.45**	**33.60**	**579.50**

Table 19

As before, all of these nodes are located near the Linton Hill Tank (see Figure 40, end of Section 4.6), and hydraulics based on elevations dictate that 30 psi cannot always be achieved.

4.6.4 Flushing Results

After flushing each individual dead-end junction which was identified as deficient in the Existing scenario (refer to Table 5, Section 4.2.5), Hethcoat & Davis, Inc. was able to identify only two nodes with a pressure below 20 psi in the 20-Year Improvements Scenario. These results are illustrated in Table 20.

20-Year Improvements Scenario - Junctions with Pressures < 20 psi at Hour 77 (5:00 am) after Flushing Analyses				
Location of Flushed Junction	**Junction Flushed (flow added based on 2.5 ft/sec)**	**Deficient Junctions (pressure < 20 psi during flushing)**	**Pressure at Deficient Junction (psi)**	**Pipe Size at Deficient Junction (diameter - inches)**
--	all	J964*		12
--	all	J1088*		12, 16

Table 20

These two nodes (identified by an asterick) were previously identified as not being able to maintain 30 psi minimum domestic pressure after the 20-Year Improvements (refer to Table 19). Therefore, it is not surprising that the nodes could not maintain 20 psi during flushing analyses

4.6.5 Fire-Flow Results

Additional fire-flow nodes were added with the 20-Year Improvements, and the total number of fire-flow nodes increased from 456 to 457. Upon analysis, it was determined that all locations were able to meet fire-flow requirements of 500 gpm (see Figure 41, end of Section 4.6).

In total, of the 457 nodes to which fire-flow was applied, zero nodes were unable to supply 500 gpm, 110 nodes could supply 500 gpm to 1,000 gpm, and 347 nodes were supply to flow 1,000 gpm or greater, all while maintaining 20 psi throughout the system. Table 21 illustrates the improved system fire-flow capabilities.

Amount of Fire-Flow Nodes in Specified Flow Ranges for 20-Year Improvements Scenario							
Scenario	**<500 gpm**	**% Total**	**≥500 gpm, <1,000 gpm**	**% Total**	**≥1,000 gpm**	**% Total**	**Total Nodes**
Existing	49	12.3	104	26.2	244	61.5	397

5 Year	17	4.1	109	26.5	285	69.4	411
10 Year	15	3.3	102	22.5	337	74.2	454
15 Year	13	2.9	103	22.6	340	74.5	456
20 Year	**0**	**0.0**	**110**	**24.1**	**347**	**75.9**	**457**

Table 21

It should be noted that following the 20-Year Improvements, 100.0 percent of fire-flow nodes appear to be capable of delivering 500 gpm (or greater) while maintaining minimum system pressures of 20 psi.

4.6.6 Projected Cost Estimate

Refer to Table 22 (end of Section 4.6) for projected costs of 20-Year Improvements.

Chapter 5 – Conclusions and Recommendations

5.1 General

The intent of all proposed improvements was to correct (or minimize) the Princeton Water and Wastewater water system deficiencies related to system storage, residual pressures, flushing and fire-flows. These deficiencies were identified in the Existing scenario.

5.2 Conclusions

5.2.1 Storage

With the addition of a 16-inch transmission line, the utilization of the altitude valve at the Linton Hill Tank and the reconfiguration of the high service pump controls (see Section 4.5), the total system storage has been utilized. This improvement can primarily be seen in the water level of the Industrial Park Tank. Whereas the existing conditions allow the Tank to fill to a level of only 24 feet, the total system improvements allow the Tank to fill to its maximum level of 40 feet. Therefore, approximately 400,000 additional gallons of available storage will be utilized at the Industrial Park Tank with the proposed improvements, as Figure 42 illustrates. Note: As demand changes over time in the area served by the Industrial Park Tank, High Service Pump controls may need to be altered to optimize Tank fill/discharge flow rates.

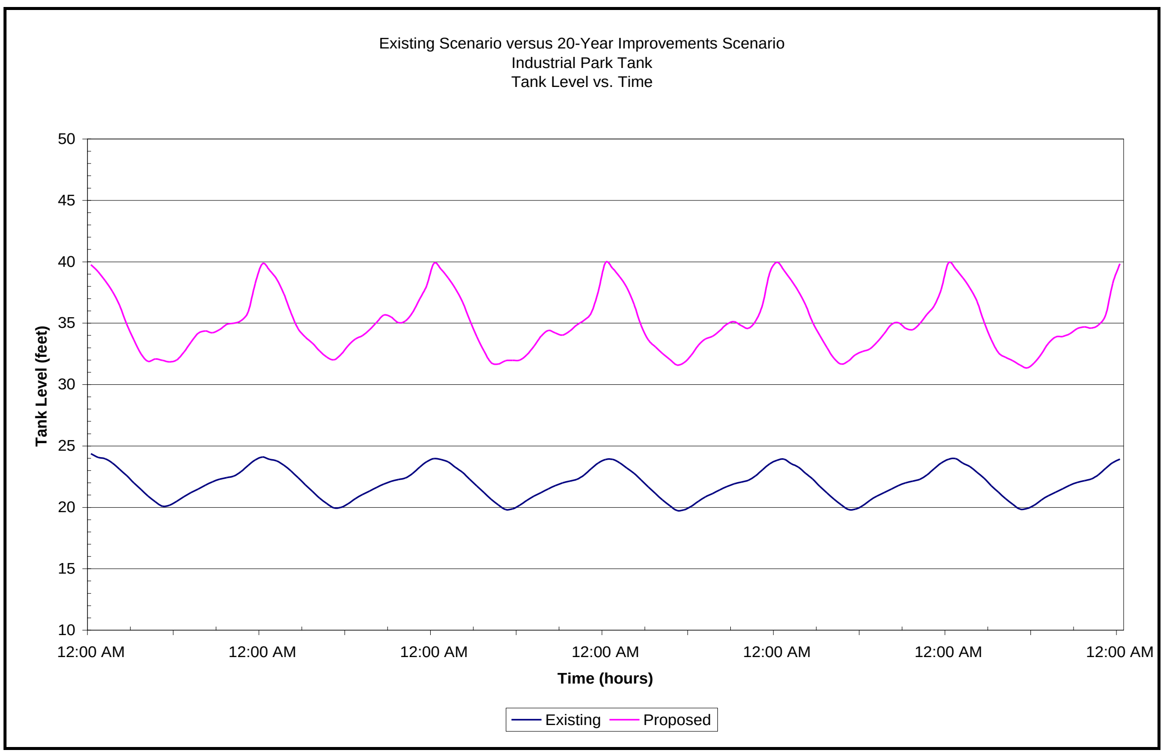

Figure 42

5.2.2 Pressure

Under existing conditions, the water system contains ten nodes which are unable to maintain a minimum residual pressure of 30 psi at all times. With the addition of these proposed improvements, this deficiency is reduced by 60 percent, as only four nodes will be unable to maintain the required residual pressure. Considering that these nodes are all located near the Linton Hill Tank, no development in this area should be served by the Low Pressure zone. It is for this reason that the remaining four deficient nodes can be ignored. Note: In the 5-Year Improvements scenario, a 6-inch line was installed and connected to the High Service zone, which will accommodate future service in this area.

5.2.3 Flushing

Under existing conditions, the water system contains fourteen nodes which are unable to maintain 20 psi during flushing. With the addition of these proposed improvements, the number of deficient nodes is reduced to only two nodes. As these nodes located near the Linton Hill Tank are hydraulically unable to maintain residual pressures of 30 psi, the inability to maintain 20 psi during flushing is not a concern and can be ignored (refer to Section 5.2.2).

5.2.4 Fire-Flow

Based on the existing conditions, it was determined that 49 nodes were unable to supply 500 gpm, 104 nodes could supply 500 gpm to 1,000 gpm, and 244 nodes were able to flow 1,000 gpm or greater, all while maintaining 20 psi throughout the system.

Based on total system improvements, of the 457 nodes on lines 6-inches or greater to which fire flow was applied, zero nodes were unable to supply 500 gpm, 110 nodes could supply 500 gpm to 1,000 gpm, and 347 nodes were able to flow 1,000 gpm or greater, all while maintaining 20 psi throughout the system. Table 23 illustrates the improved system fire-flow capabilities.

No. of Fire Flow Nodes in Specified Flow Ranges for Existing Scenario versus 20-Year Improvements Scenario

Scenario	<500 gpm	% Total	≥500 gpm, <1,000 gpm	% Total	≥1,000 gpm	% Total	Total Nodes
Existing	49	12.3	104	26.2	244	61.5	397
20 Year	0	0.0	110	24.1	347	75.9	457
% +/-		-12.3		-2.1		+14.4	

Table 23

From the existing conditions to the 20-Year Improvements, the amount of nodes capable of achieving 1,000 gpm increased by over fourteen percent, and the amount of deficient nodes decreased to zero. With the additions of all recommended improvements, PW&W can feel confident that adequate fire-flow of at least 500 gpm can be achieved throughout the system while maintaining residual pressures above or equal to 20 psi.

5.4 Summary

With the additions of all proposed improvements, the Princeton Water and Wastewater water distribution system will be capable of providing safe drinking water, adequate pressure and fire protection to all customers in the system. Additionally, system storage will be maximized, and water loss should be reduced. PW&W should feel confident that they will be meeting or exceeding all Kentucky Division of Water distribution system requirements while providing the best service possible to the existing system. Furthermore, the system will be well equipped to handle future growth and demand in the surrounding areas.

Princeton Water and Wastewater
Water System Master Plan - Recommended Improvements Summary

5 Year Improvement Recommendations	Description
5A(1)	Booster Pump at Industrial Park Tank (or Mixer/Chlorination System)
5B(1)	8-inch and 10-inch Lines at Super Wal-Mart
5B(2)	6-inch Lines at Robin Road, South Darby Street, Cherry Street, North Harrison Street, Oak Drive (Approx 6,800 LF w/68 Service Reconnections)
5B(3)	Approximately 900 LF of 6-inch Line at Hillview Court (Pressure Zone Change)
5B(4)	Approximately 900 LF of 6-inch Line at Hillview Drive w/8 Service Reconnections (Pressure Zone Change)
5D(1)	Approximately 3,200 LF 8-inch Line at Cadiz Road
5D(2)	Approximately 1,400 LF 8-inch and 4,600 LF 12-inch Lines at Highway 91N
5D(3)	Approximately 1,400 LF 8-inch Line at Highway 62 West
Estimated Project Cost	$1,445,000

10 Year Improvement Recommendations	Description
10D(1)	Approximately 60,000 LF of Replacement of all 2-inch, 3-inch and 4-inch Lines Which Are Not Dead-end Lines With New 6-inch Lines w/600 Service Reconnections Estimated
Estimated Project Cost	$3,524,000

15 Year Improvement Recommendations	Description
15A(1)	Approximately 8,200 LF of 16-inch Transmission Line
15A(2)	Utilization of Altitude Valve at Linton Hill Tank
15A(3)	High Service Pump Controls Changed to Read Levels of Industrial Park Tank
Estimated Project Cost	$1,292,000

20 Year Improvement Recommendations	Description
20B(1)	Approximately 1,400 LF of 6-inch Line at Harvey Lane
20B(2)	Connections to Existing 10-inch Lines
20B(3)	8-inch line at Baldwin Avenue
20D(1)	Approximately 11,700 LF of 10-inch line at Highway 293 North
20D(2)	Approximately 7,400 LF of 6-inch Line at Old Fredonia Road
20D(3)	Approximately 2,250 LF of 6-inch Line at Jeff Watson Road
20D(4)	Approximately 1,950 LF of 6-inch Line at Vivian Road
20D(5)	Approximately 1,200 LF of 6-inch Line at Cooper Circle
20D(6)	Approximately 2,260 LF of 8-inch Line at University Drive
Estimated Project Cost	$1,844,000

Type of Improvement -- "A" Storage Improvement; "B" Pressure Improvement; "C" Flushing Improvement; "D" Fire Flow Improvement