PERFORMANCE EVALUATION OF A DRINKING WATER DISTRIBUTION SYSTEM USING HYDRAULIC SIMULATION SOFTWARE FOR THE CITY OF OILTON, OKLAHOMA

By

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2004

Submitted to the Faculty of the
Graduate College of the
Oklahoma State University
in partial fulfillment of
the requirements for
the Degree of
MASTER OF SCIENCE
December, 2009

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ACKNOWLEDGMENTS

I would like to take this opportunity to thank Dr Dee Ann Sanders, my thesis advisor, for her unending support and guidance throughout the project and my thesis. It was a wonderful experience working with you. I would like to thank Dr Arthur Stoecker for his efforts in managing the entire project and sharing his resources related to the project and Dr. Brain Whitacre for his coordinated efforts. I extend my thanks to the Oklahoma Water Resources Research Institute, for funding this project. My thesis is based on a small part of a larger project and therefore I would like to thank my project members Michael Lea, Sara Senyondo, Ernest Atta-Asiamah, Deepayan Debnath and Jong San Choi.

I would like to acknowledge and thank Mr. Green, the Mayor and Mr. Bruce Coldiron, the Public Works Director of City of Oilton for their co-operation and providing me with the necessary information for this project. I would also like to thank Mr. John McElhenney from INCOG for providing water system data, especially the GIS shape files for the City of Oilton.

I would like to acknowledge, the faculty and the staff members of the Civil and Environmental Engineering, for giving me an opportunity to pursue my graduate studies and provide me guidance which will be beneficial in shaping my future in the field of

Environmental Engineer.

I thank my parents, Rupa and Mangesh Bhadbhade for giving me the opportunity to get this international experience. Your efforts in nurturing my dreams cannot be put into words and you both are my biggest strength. At the end, I would like to thank my friend Francis Ekene Akanisi for his support and co-operation.

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CHAPTER I

INTRODUCTION

Background of Project

The 2007 Environmental Protection Agency (EPA) report of Drinking Water Infrastructure Needs Survey and Assessment stated that the United States would need an investment about 335 billion dollars to upgrade its water infrastructure in the coming 20 years. The report said that out of this entire revenue, 60% would be required for just upgrading the distribution systems. The state-by-state classification of the report said that Oklahoma would need about 2.6 billion dollars, out of which 1.4 billion dollars would be required to upgrade the systems serving populations fewer than 3300 people (EPA, 2009). The Oklahoma Water Resources Board (OWRB) set a new water plan to project water demands and the required inventory to meet these demands up to the year 2060. The preliminary goals of this project were as follows:

- Identify those regions having problems related to water supply
- Collect data, maps and other vital information regarding their water infrastructure.
- Evaluate the performance of their systems on the basis of their existing demands.
- Identify the necessary changes in the system to meet future water demands (OWRB, 2006)

OWRB identified 1717 active public water systems, out of which 1240 systems were community water systems, either municipal or rural water districts. Partners in this planning process were the Oklahoma Water Resources Research Institute, the Oklahoma Association of Regional Councils (COG's), Oklahoma Department of Environmental Quality (ODEQ) and federal partners. Based on the water plan for Oklahoma, a project goal was set to develop a cost efficient methodology, which would assist rural water districts in Oklahoma to manage and upgrade their drinking water distribution systems. This project was funded by the Oklahoma Water Resources Research Institute (OWRRI).

Scope of the study

The primary goal of the study is to assess the performance of the existing distribution system of the City of Oilton, Oklahoma, which is aimed to help the City of Oilton understand its distribution system needs and assist them in long-term planning of water assets. Thus, the scope of this study is to evaluate the performance of the existing drinking water distribution system using hydraulic simulation software and recommend changes, if any, in the existing system. The detailed design of the proposed changes to the current distribution system and cost analysis is not within the scope of the project.

Selection of hydraulic simulation software

The hydraulic simulation software used for this study is WaterCAD V8i distributed by Bentley Systems. Other research group members who have done similar investigations for other towns in Oklahoma have used EPAnet, which is free software. The aim was this project was to provide an economic tool which would be affordable to

rural water districts. However, after completion of a previous study carried out by my fellow research group member, it was evident that these hydraulic simulation softwares are too sophisticated to be handled and updated by the rural water districts' staff.
Thus this project has a demonstration approach. WaterCAD V8i was selected due to ease of model building and operation and its greater programming capabilities as compared to EPAnet. This study would be useful to those technical consulting firms which carry out work for smaller communities.

Selection of Site

As mentioned previously the goal of this project was to assist smaller communities in planning their infrastructural needs in a sustainable and cost-efficient manner. The City of Oilton was selected because it is a non-metropolitan community which has a population less than 10,000 people. The current population of the town is 1200 people. Oilton is one of the rural water districts in Creek County. After a discussion with the project group, the City of Oilton was found to be suitable for carrying out the investigations and was thus selected.

Site description:

The City of Oilton is located in Creek County and is approximately 54.6 miles to the west of Tulsa and about 42 miles to the east of Stillwater. Located close to the Cimarron River, the city of Oilton houses a small community having a population of about 1200 people. The approximate area of the city is 0.65 square miles, which is about 416 acres. The City of Oilton receives its water supply through groundwater. The system has two wells that are located about 5 miles to the south of the city. The storage facilities used by the town are two standpipe tanks. One tank is located outside the city and the other tank is located in the city. The exact age of the pipelines is not known, but a map of the lines classified by remaining utility life is shown later in Chapter 3. The main pipeline that brings water to the city is an eight inch asbestos cement pipeline. There are two main pipes, one which is an eight inch PVC pipeline and one is an 8 inch asbestos cement pipeline. All other mains and sub-mains are in the range of 1 to 6 inches. Figure 1 shows the map of the town. Figure 2 shows the picture of the standpipe outside the town and Figure 3 shows the standpipe in the town. Figure 4 shows the elevation profile of the water well and the standpipes and the related information is also summarized in Table 1. The city does not have any water treatment plant. Chlorine in form of compressed chlorine gas is used for disinfection. The north well has considerable amount of iron. The water has a reddish color and metallic taste (Coldiron, 2009). According to the Safe Drinking Water Information System (SDWDIS) Violation Report, the city has failed a number of times to collect samples for coliform testing in timely manner. However, there were no health-related violations reported (EPA, 2009). The town does not have any major industries or commercial institutions. Oilton has an ISO fire class 6 (Green, 2009).

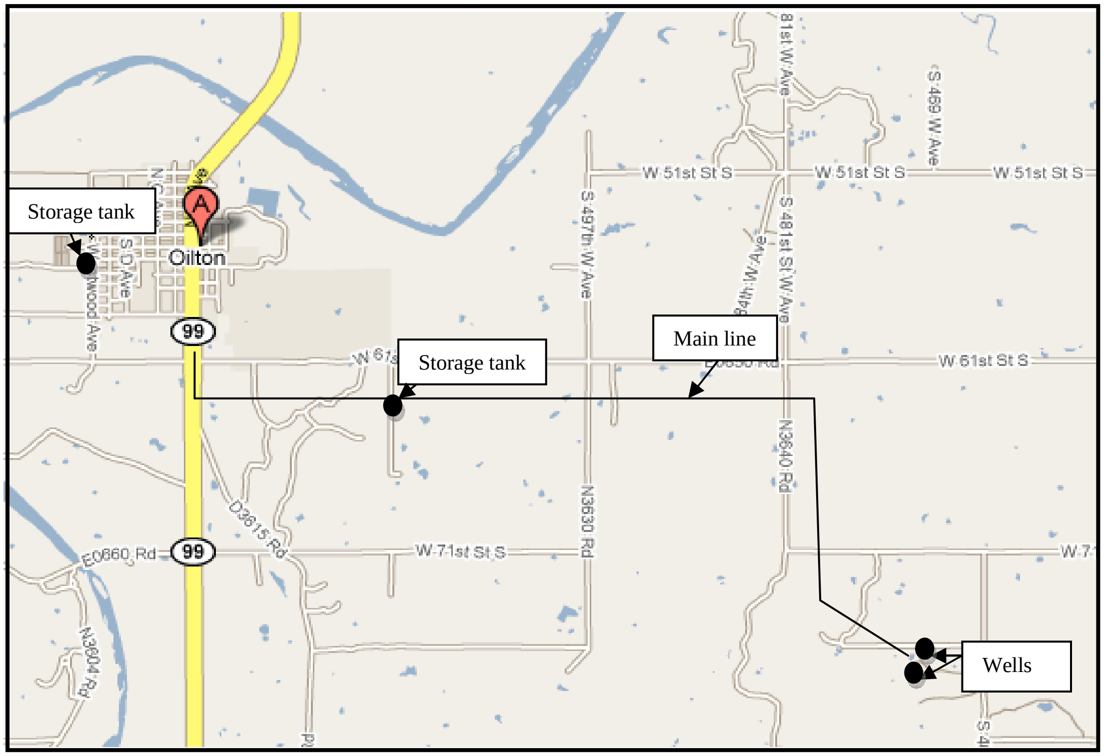

Figure 1: Map of the City of Oilton (Source: Google Maps, 2009)

Table 1: Elevation and distance data of water supply facilities

Parameter	**North Well**	**Standpipe outside the Town**	**Standpipe in the Town**
Ground elevation above mean sea level	877.11 ft	898 ft	858.73 ft
Highest Elevation of water	350 ft below ground elevation = 527.11 ft	60 ft above the ground elevation = 958 ft	126 ft above the ground elevation = 984.73 ft
Distance from the well	Not applicable	3 miles from North Well	2.2 Miles from the North well

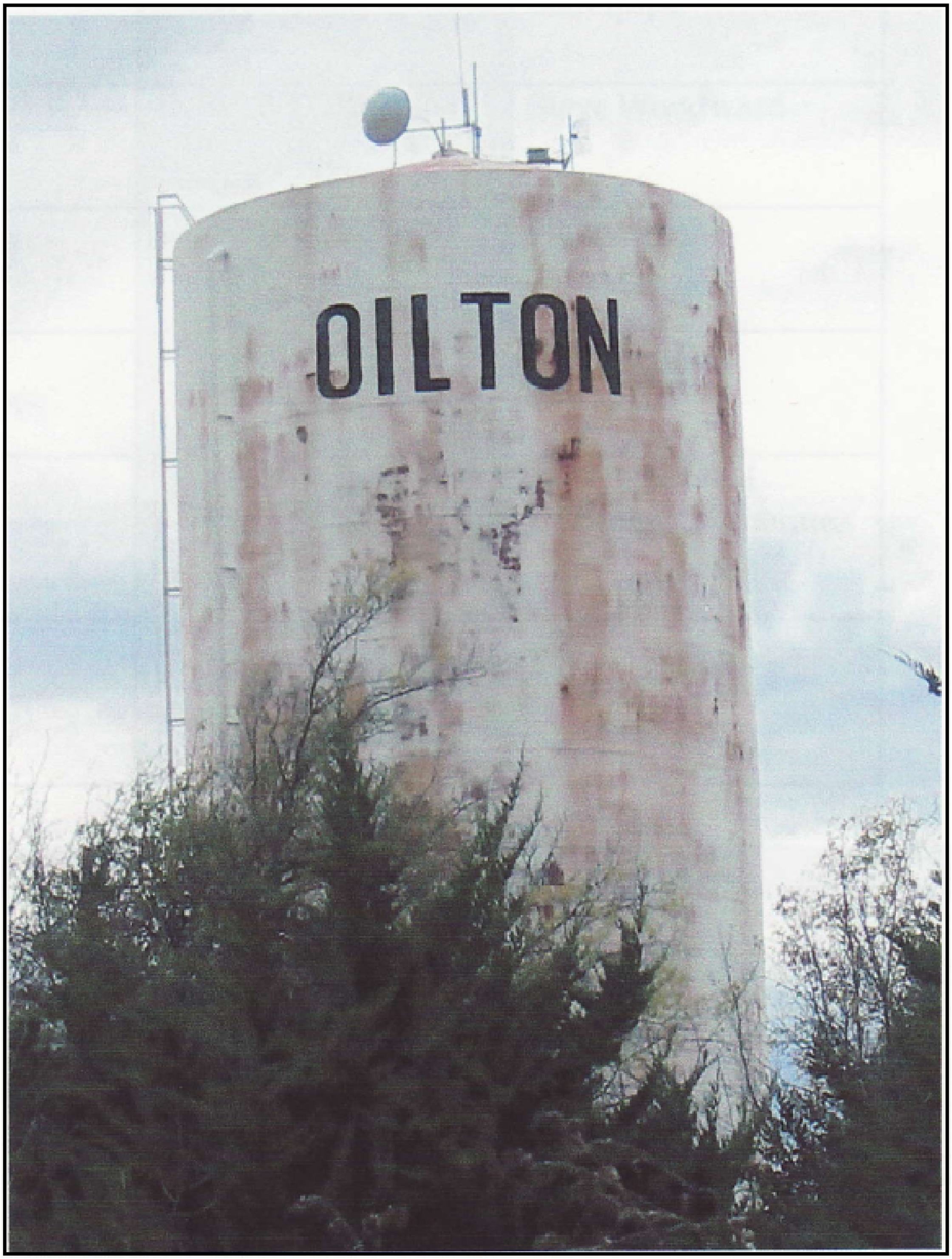

Figure 2: Picture of the 350,000 gallon Standpipe outside the town.

Figure 3: Picture of the 600,000 gallon standpipe in the town.

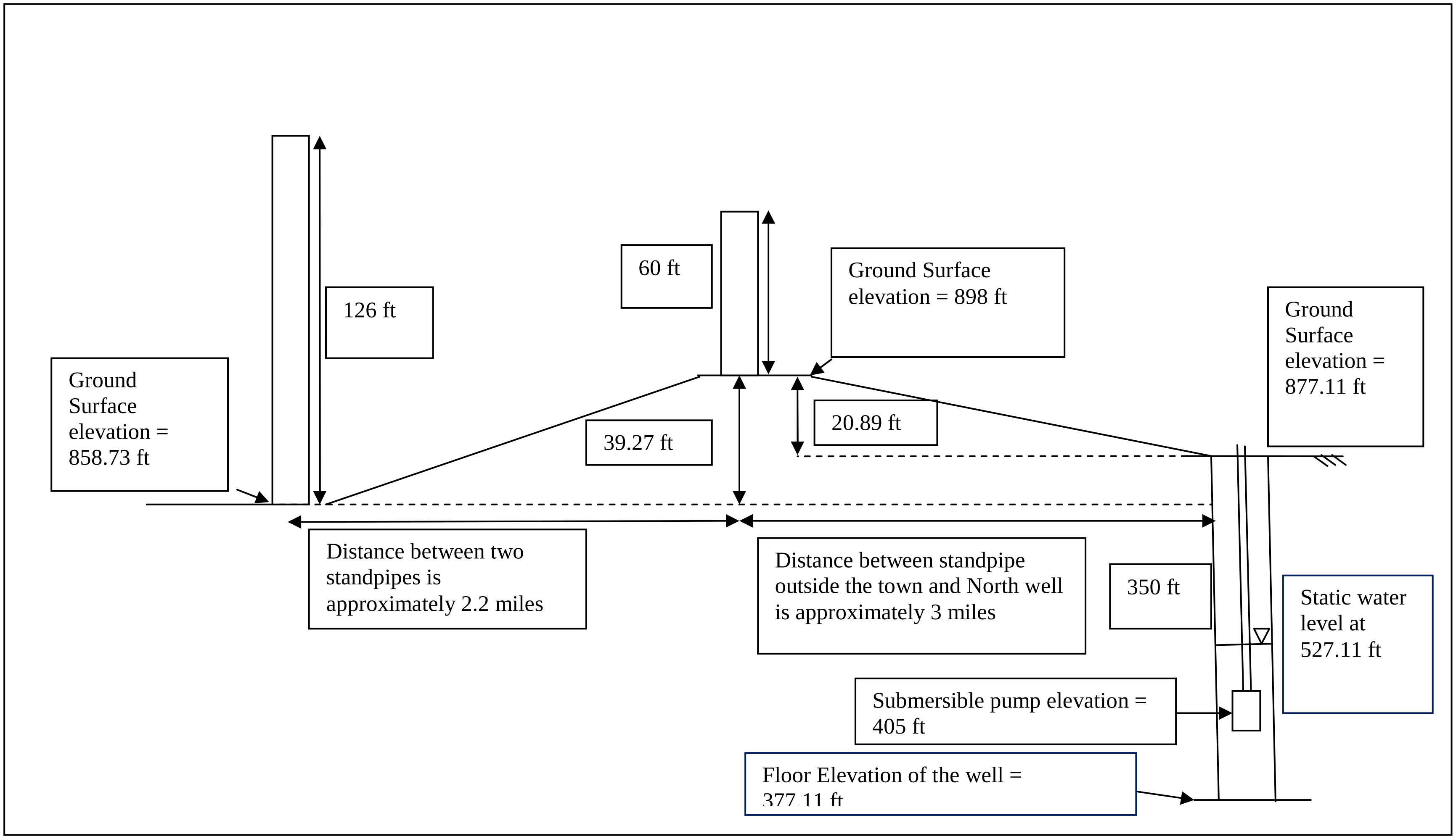

Figure 4: Elevation Profile of the water well and the standpipes

CHAPTER II

REVIEW OF LITERATURE

Water problems in Small water supply systems:

The National Drinking Water Regulations as amended through January 14, 2002, (2000 Code of Federal Regulations, Title 40,Vol 19, Part 141) states that a community water system has at least 15 service connections used by the year round residents, or regularly serves at least 25 year-round residents. These water systems generally serve cities and towns. They may also serve special residential communities, such as mobile home parks and universities, which have their own water supply (Salvato 2003). “The community water system is considered to be small if it serves less than 10,000 people” (National Research Council (U.S.) Committee on Small Water Supply Systems, 1997). A 1995 EPA survey stated that in 1963 there were 16,700 small community water systems and by 1993 the number of such communities had risen to 54,200, which was more than 3 times those that existed in 1963. This survey also estimated that about 1000 such communities are formed every year (National Research Council (U.S.). Committee on Small Water Supply Systems, 1997). After the Safe Drinking Water Act was passed in 1974, the US Congress thought that these small communities would unite to form large regional systems. However they continued to increase, forcing EPA to implement plans and undertake initiatives to provide technical and financial assistance to the communities serving fewer than 3300 people (Stoecker, 2007). The biggest problem with the small water communities is inadequacy of funds to maintain and upgrade their water infrastructure. Due to this most of the small water communities have difficulty funding construction of new facilities and maintenance of staff. Many times these communities lack the technical resources that can guide them to optimize and sustainably operate their water distribution systems. One other major problem area is that these small communities do not have sufficient data and records for the proper maintenance of their distribution systems. Lack of technical expertise causes failures and many of these communities can only fix problems and they continue to expand the existing systems without considering the reliability of the existing design. These problems lead to non-compliance with the state drinking water quality regulations or the Safe Drinking Water Act (SDWA). Small communities are prone to violate these regulations as much as three times more often than the large cities (National Research Council (U.S.). Committee on Small Water Supply Systems,1997). Thus, there is a need to evaluate the performance of distribution system of these towns to address the current and future issues related to their water distribution systems.

Need for hydraulic modeling

Most small communities do not have very complex networks as compared to cities; however, they have poor data and records regarding their systems. In such cases, when one has to evaluate the hydraulics and the water quality of the distribution systems, it is advantageous to use computer models. Computer models making use of hydraulic simulation software are capable of mimicking the behavior of a real time system and have the capability of predicting the performance of the same system for future ‘what if’ scenarios (Haestad Methods, 2003). Some rural water districts that have the capability of maintaining and updating real time models, have used hydraulic simulation models in conjunction with geographic information systems, allowing them to perform criticality studies with greater precision (Zhang, ESRI Users Conference 2009). This can be cost effective as it will provide decision support in operation and maintenance of their systems.

Design criteria for Performance Evaluation

The performance of the system is measured based on its ability of the system to deliver good quality water at all the times under suitable set of operating conditions (Coelho, 1997). This performance depends on a number of criteria. Planning of these systems is very important and the factors that need to be considered are as follows:

- Design life of the system
- Appropriate advantages of topographic features to reduce energy costs
- Projected population growth
- Projected industrial and commercial growth
- Water consumption data: average daily consumption, per capita consumption and peak flow factors
- Minimum and maximum acceptable pressures.
- Storage facilities (Swamee, 2008)

Engineering of a good water supply system is very complex. Based on the above criteria, design period can be based on projected growth. Alternatively, for static populations like rural water communities the design period can be based on the life period of the pipes (Swamee, 2008). The Oklahoma Department of Environmental Quality has issued guidelines for public drinking water systems. As per these guidelines, if a community is being supplied with groundwater, it should have at least two water wells, or have a standby source which can provide adequate water supply. Also, "if the town is supplied water only through the groundwater resource, then the capacity of the well must be equal to or greater than the design maximum day demand and the design average day demand, with the largest producing well out of service" (ODEQ, 2009). All pumping stations will have two pumps. In case of failure of one pump, the other pump should have a capacity of providing water during the peak periods in the day maintaining optimum pressures. All storage tanks should be able to provide enough storage facility to meet the regular average daily demands satisfying peak hourly periods but most importantly fire flow demands at a key location peak hours (Salvato, 1992). Generally, the peak hourly flow factors are 3 to 6 times the average daily flows (Haestad Methods, 2003). Also the maximum design variation in the storage levels should not vary more than 30 ft to maintain the required pressures. In case the distribution system does not provide fire protection, then it should have storage capacity of 24 hours and must be able to maintain a pressure of at least 25 psi throughout the distribution system (ODEQ, 2009). As per the Insurance Services Office (ISO), towns having fire class greater than 8 should be able to provide a flow of 250 gpm at peak daily demand at a pressure of at least 20 psi (ISO Mitigation Online, 2009). "Dead ends should be minimized by looping them to the main network system" (ODEQ, 2009). A hydrant or a flushing device should be preferably installed at dead ends so as to not have issues of water contamination due to stagnation (ODEQ, 2009). Main water lines should be at least 6 inches in diameter, and the least diameter of the pipe in the system should be 2 inches. The design velocities in the pipes can range from 3.3 to 6.6 ft/s (Salvato, 1992). "Also if the groundwater is subjected to low contamination, then only chlorination shall be used for disinfection, if the coliform count is not more than 50 per 100 ml on an average in one month and if the turbidity of the water is not greater than 5 NTU." (ODEQ, 2009)

***Basic Principles of Hydraulic Modeling*:**

In hydraulic simulation modeling a distribution network is considered to be one in which all elements are connected to each other, every element is influenced by its neighbors, and each element is consistent with the condition of all other elements. These conditions are mainly controlled by two laws: Law of Conservation of Mass and Law of Conservation of Energy. "Thus the total mass of water entering the system should be equal to the total mass of water leaving the system, and the sum of the flows at any given node should be equal to zero. The principle of conservation of energy is mainly dictated by the Bernoulli's equation, which states that the difference in the energy between any two points should be the same regardless of the path taken" (Haestad Methods, 2003).

A typical network in hydraulic model consists of the following components:

- Nodes linking the pipes
- Pipes
- Storage tanks

- Reservoirs
- Pumps
- Additional appurtenances like valves (Haestad Methods, 2003 ; Rossman, 2000)

The junctions or nodes represent points having particular base demands. Tanks are those points in model, which can have a specific storage capacity that varies with time. Reservoirs in a hydraulic model are assumed to be an infinite source of water (Haestad Methods, 2003; Rossman 2000). Pumps are energy devices which provide pressure and head to the water. The graph of head vs. flow for a particular pump is called the '*pump curve*'. Figure 5 shows a typical pump curve. Generally there are three parameters that define the pump operation; the shut off head, the design point, and the maximum point. The system curve is an important curve necessary to decide the best operating point of pump. The pump should be able to overcome the elevations differences, which is dependent on the topography of the system. The head added on the pump to overcome these differences is called the static head. Friction and minor losses also affect the discharge through the pump. "When these losses are added to the static head for different discharge rates, the plot obtained is called system head curve" (Haestad Methods, 2003). The operating point is considered to that where the system curve intersects the pump curve as shown in Figure 6. The other important curve related to the pumps is the pump efficiency curve as shown in Figure 7. The point at which the peak efficiency occurs is the best operating point.

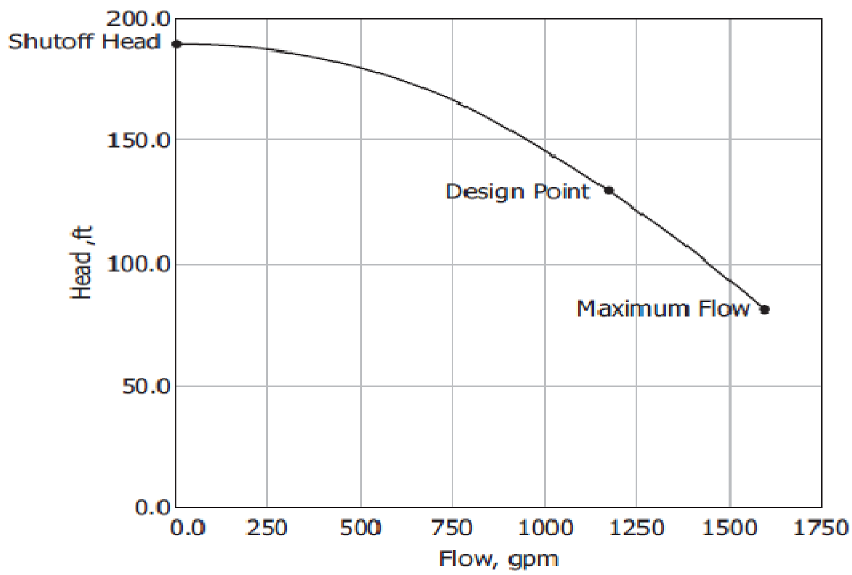

Figure 5: Pump Curve (Source: Haestad Methods 2003)

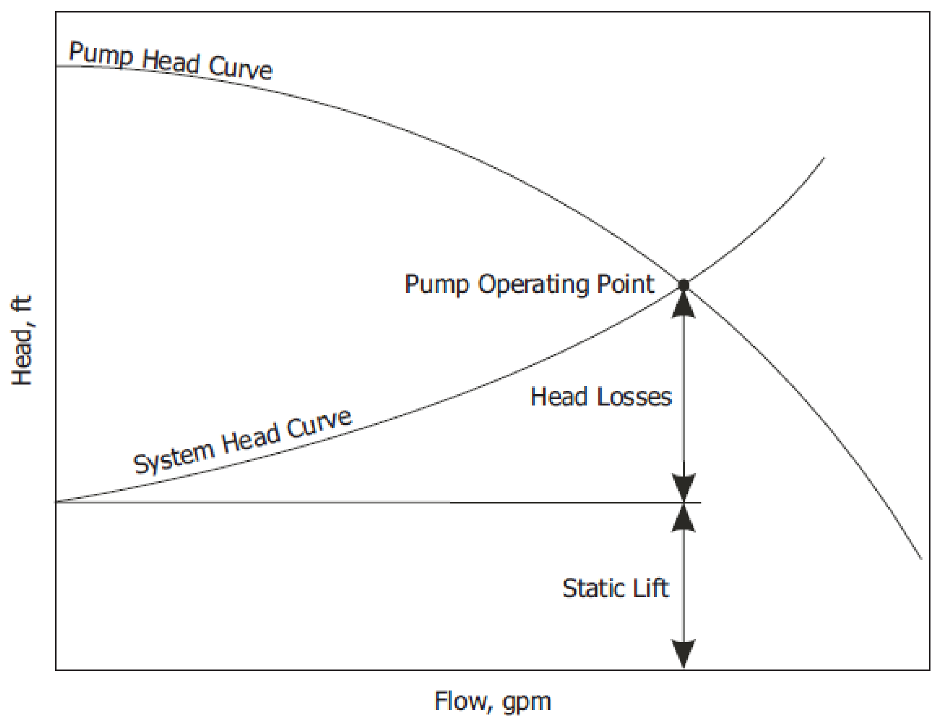

Figure 6: System Curve Vs the Pump curve (Source: Haestad Methods 2003)

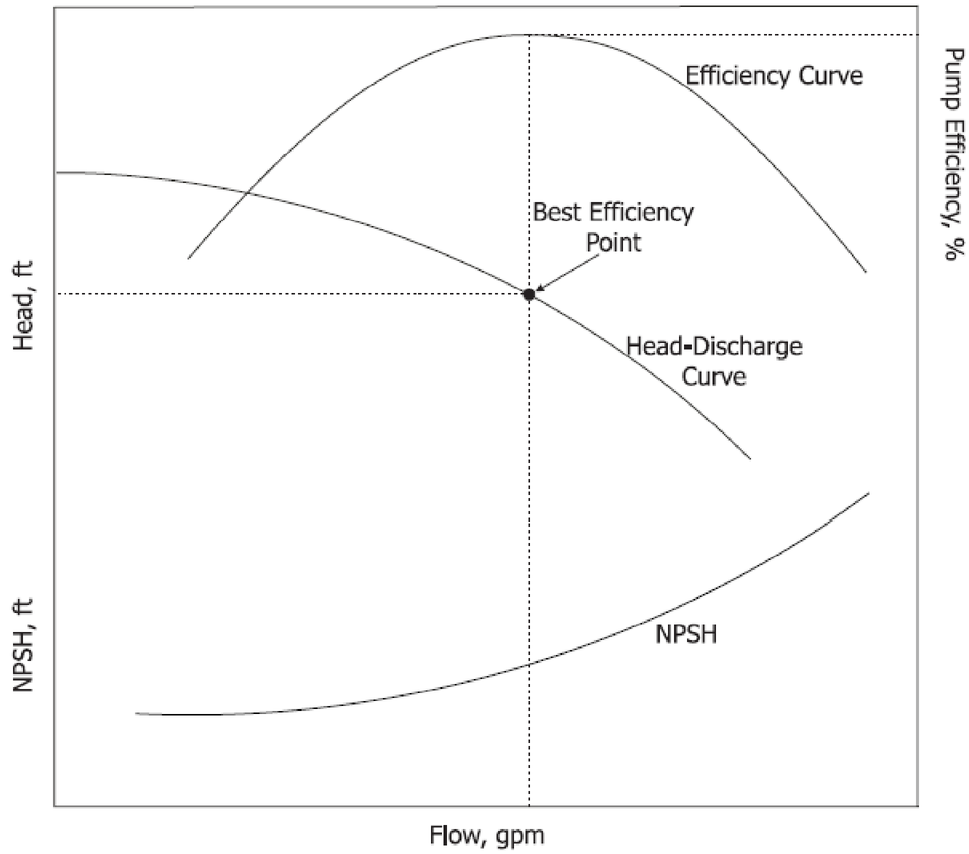

Figure 7: Pump efficiency curve (Source: Haestad Methods 2003)

In addition to evaluating the hydraulics of the system, the hydraulic simulation models can also evaluate the water quality. "The hydraulic models mainly consider two principles of transport mixing and decay while computing the water quality in the system. Network hydraulic solutions are utilized to compute water quality." (Haestad Methods, 2003). WaterCAD uses the equations developed by Grayman, Rossman and Geldreich (2000) for determining the transport of constituents through the pipe, mixing at the nodes and the tanks and decay of constituents.

Modeling a system using WaterCAD

WaterCAD is hydraulic simulation software, distributed by Bentley Systems. Once the spatial model is built, the parameters that need to be defined for each model components include:

- Nodes: Elevations and the base demands
- Pipes: Pipe diameters, lengths and the friction coefficient factors. By default WaterCAD considers the pipe material as ductile iron having a Hazen William friction coefficient factor of 130
- Tanks: Base Elevation, the minimum and maximum levels, diameter of the tank
- Pumps: The most important parameter defining the pump operation is the pump curve. Other input needed is the elevation of the pump
- Reservoir: Elevation

After all the parameters required to run the simulation are entered into the model, the successful simulation run provides solution for the following:

- Pressure at every single element in the system
- Flows at every point of time in the system
- Velocities in the pipes
- Levels in the tanks
- Pump cycles
- Water age and constituent concentration.

Additionally it has the capability of performing the analysis of the system for the steady state scenarios and for an extended period of any length. The other capabilities of the software are as follows:

- Evaluate the hydraulics for different demands at a single node with varying time patterns
- Solve for different frictional head losses using Hazen-William, Darcy-Weisbach or the Chezy-Manning equations
- "Can determine immediate inefficiencies in the system" (Haestad Methods 2003)
- Determine fire flow capacities for hydrants
- Model tanks, including those which are not circular
- Model various valve operations
- Provides control based operations
- Perform energy cost calculations
- Model fire sprinklers, irrigation systems, leakages and pressure dependent demands at any particular node (Haestad Methods 2003)

Figure 8 shows the user interface for WaterCAD.

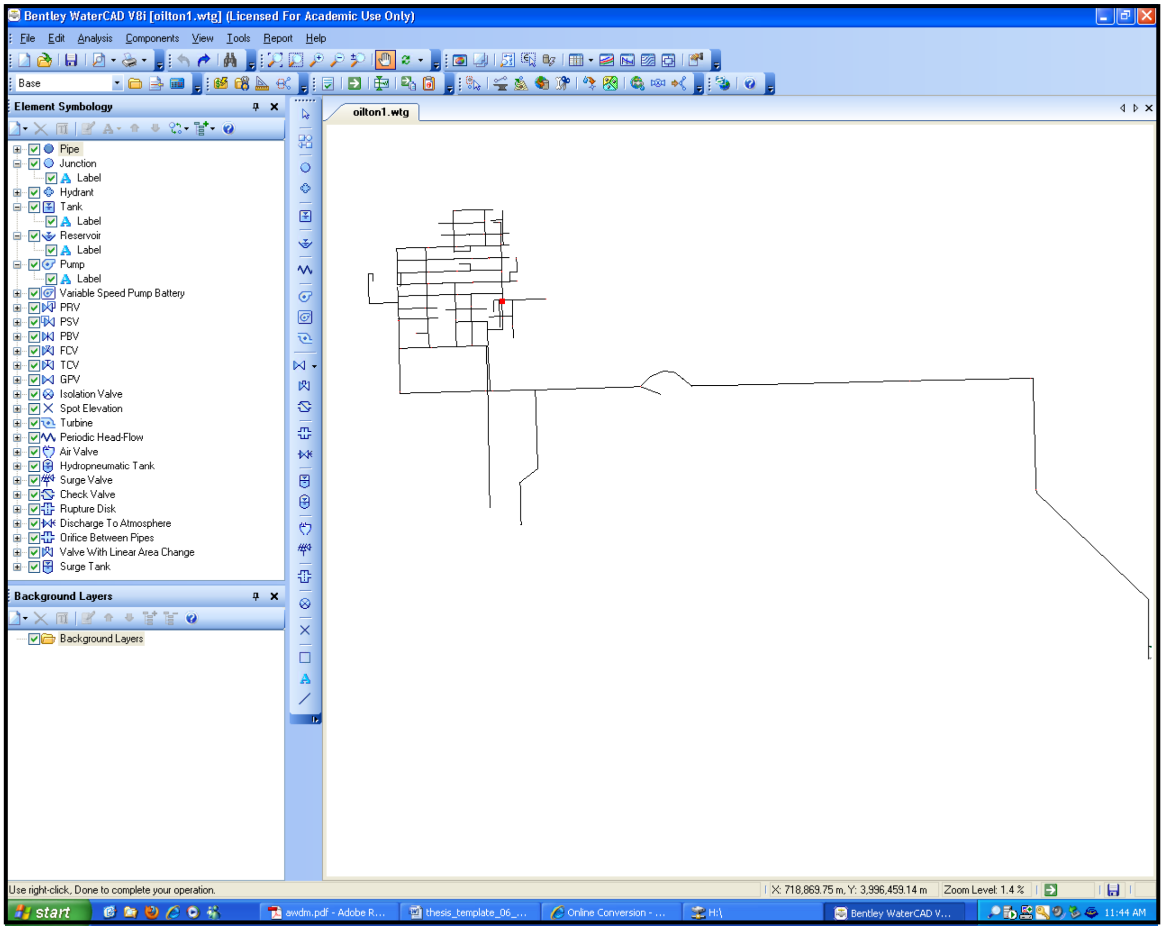

Figure 8: Screenshot of the user interface of WaterCAD.

CHAPTER III

METHODOLOGY

Introduction:

This chapter discusses in detail the steps taken to construct the model of the existing drinking water distribution system of the city of Oilton. The steps were:

- Preliminary Data Collection
- Building the model in WaterCAD
- Assigning water demands to each node
- Hydraulic Modeling using WaterCAD

Each of the following sections will discuss the above steps in detail.

***Data Collection*:**

The most important step in any research study is data collection. In building the model of the distribution network, the data were first gathered regarding all the distribution system parameters. A field visit to the City of Oilton was conducted on the July 7, 2009. The information obtained from Mr. Green, the Mayor of the town is summarized in Table 2

Table2: Preliminary Information of the distribution system for City of Oilton

Distribution system Parameter	Information obtained
Water Reservoir type	Groundwater: 2 wells, north well and south well. Both are located about 5 miles to the south-east of the main town. Each well is about 500 ft deep. No information was provided regarding the static water level of the wells.
Pumps stations	No pumping station. They have single submersible pumps on each well. No stand-by pumps. Rating of Pump of north well = 30 HP. Rating of the pump on south well = 25HP
Storage tank (Standpipes)	Tank 1: Located outside the town. Storage capacity: 350,000 gallons Dimensions: Diameter= 32 ft and Height= 60ft. Tank 2: Located within the town near the cemetery. Storage Capacity = 600,000 gallons Diameter = 30 ft, Height = 126 ft

Distribution system parameters	Information Obtained
Pipelines	Detailed hand drawn maps showing the distribution network classified by line sizes and age were provided by the city (See Figures 9 and 10)
Hydrants	The location of the fire hydrants is shown in the maps (See Figures 9 and 10)
Water consumption and water quality data	Average daily use is 118,000 GPD. The city uses chlorine gas for disinfection. They have injection pumps at each well. No water quality problems reported.

Additional information obtained after discussion with Mr. Green was that the City does not expect any industrial or commercial growth in near future. The city has a school located on Peterson Street two blocks from Highway 99. The Public Works Director, Mr. Bruce Coldiron, said that so far they have not received any complaints regarding poor pressures. The north well has considerable amount of iron. Mr. Coldiron said that occasionally the water has metallic taste and reddish color. The south well also has a lot of iron problems and hence is shut down for repairs. There is one house located at the south of the town experiences low pressure. This house is located at a relatively high elevation compared to the main town. The most important information obtained from Mr.

Green was that Indian Nation Council of Government (INCOG) did the work of mapping their water assets. Thus, the detailed shapefiles of the waterlines, towers and the groundwater wells were obtained from INCOG.

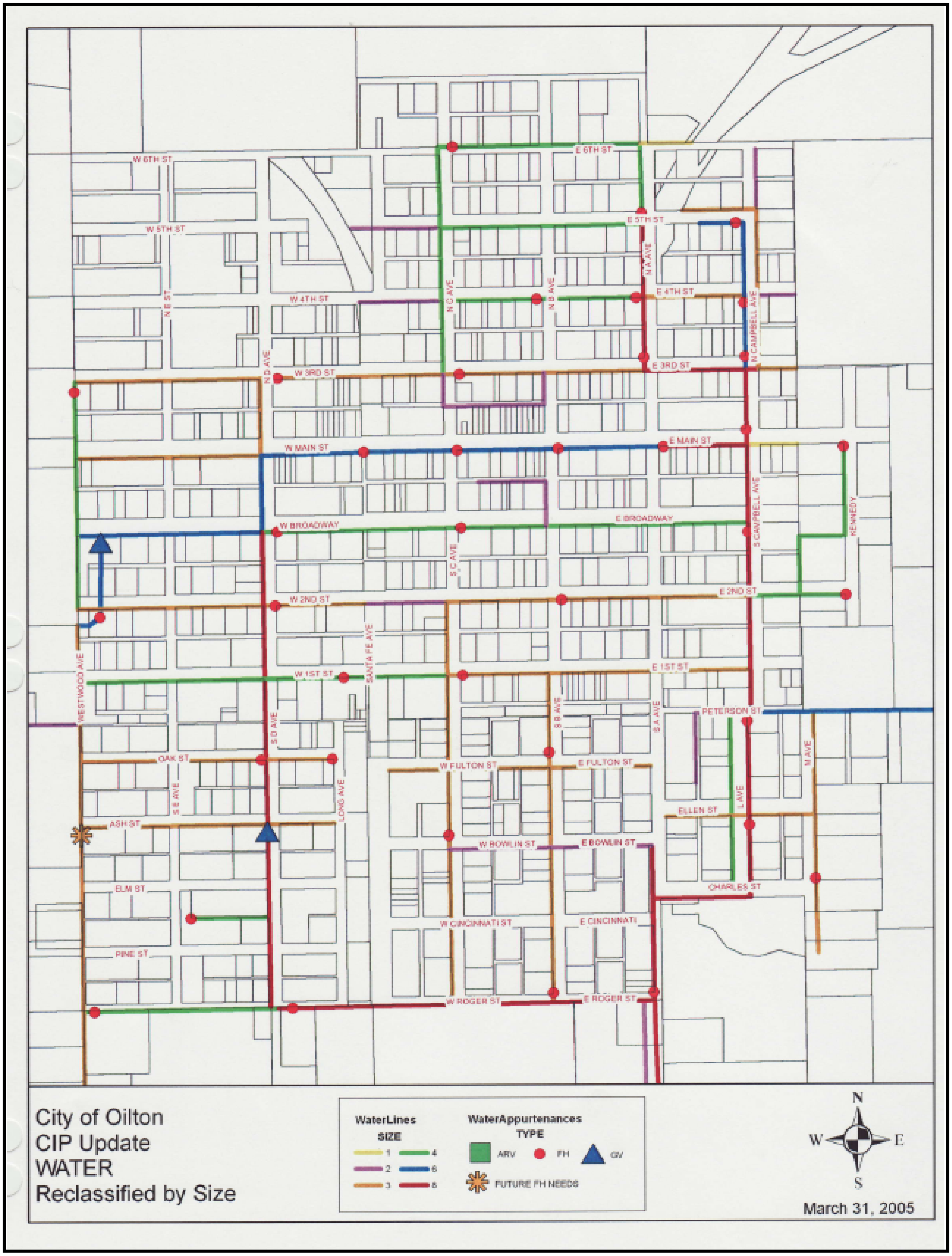

Figure 9: Map showing the distribution lines classified by size (Oilton, 2009)

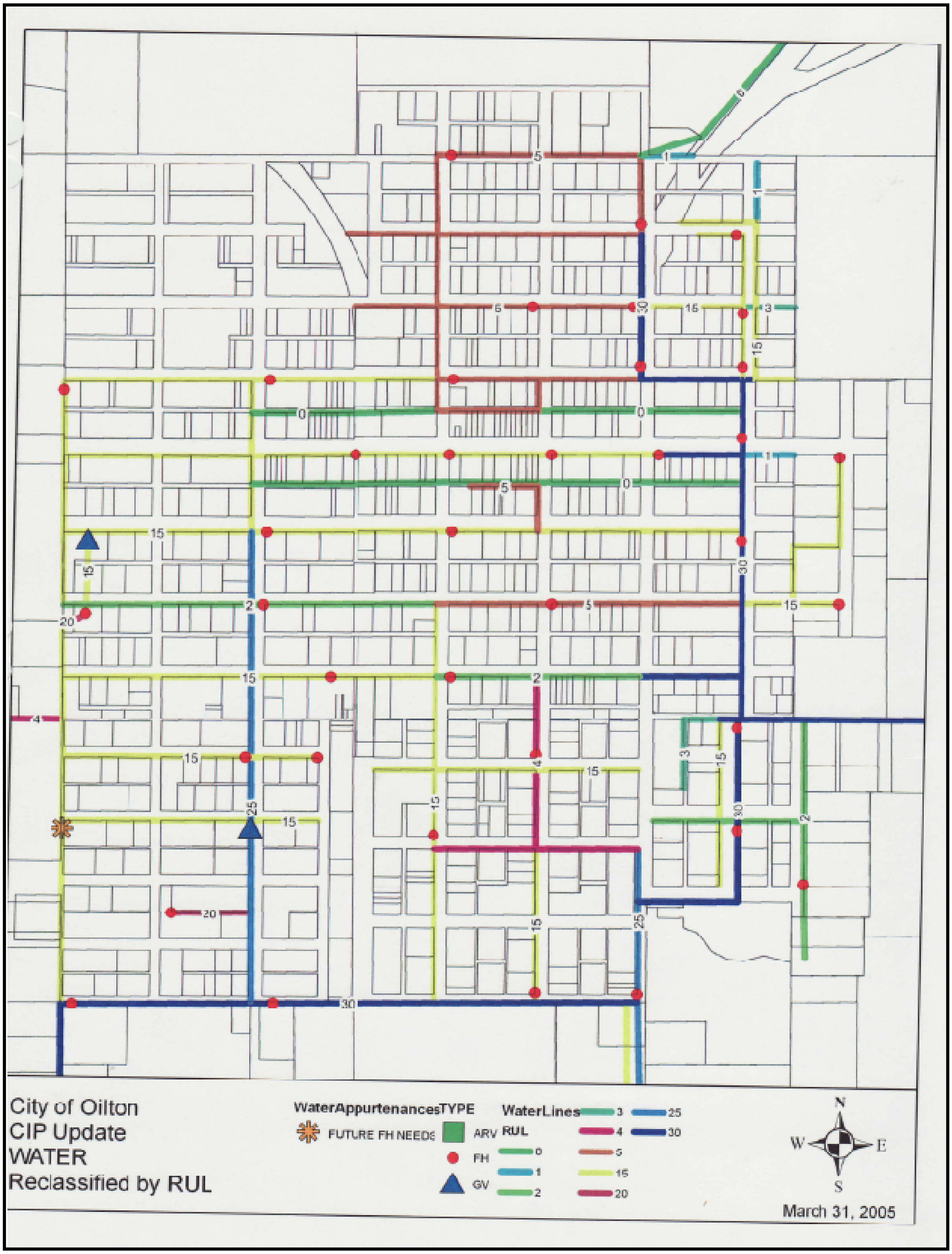

Figure 10: Map showing the classification of the lines based on remaining life (Oilton, 2009).

Apart from the preliminary information, additional inputs were required for the simulation of the model. The most important was the elevation dataset. Without the elevations, it is not possible to run the hydraulic simulation. The elevation dataset was obtained from the United States Department of Agriculture (USDA) website called the "Geospatial Data Gateway" (USDA, 2009). The user can zoom into the area of interest using the toolbars provided and then further select it. The site offers a choice of a number of downloadable datasets, the most important being landuse landcover maps, ortho-imagery and national elevation datasets. For the elevation data set, files with horizontal resolution of 10 meter and 30 meter are available. For this project, the 30 meter dataset was selected. The second important dataset necessary was the information regarding houses in each census block. This information is required to assign base water demands to each node. The census block data was obtained from the US Census Bureau website called the "2008 TIGER/Line Shapefiles". The user can select the respective state and county. Again, a variety of census data are available for download. The file selected was the Census 2000 Block. Again, the USDA Geospatial data Gateway website was used to download the ortho-images of Oilton for identification of the houses in each census block.

***Creating the Model in WaterCAD*:**

This section describes steps involved in building the hydraulic model in WaterCAD V8i. This software offers different ways of modeling the network. The user can physically draw the network if the drawings and the dimensions are available or the user can import files from AutoCAD and EPAnet. One very good feature that WaterCAD offers is the Model Builder. Using this tool one can directly import all the shapefiles at once. In the Model Builder, one can select the '*data source type'* as *shapefiles* and then click on the browse button. The user then has to browse to the specific location where the shapefiles and stored and then select all of them. One very important aspect that the user has to consider during modeling is that all the geospatial data files used during modeling should have the same geographic projection. The shapefiles of the water lines, appurtenances, reservoirs and the storage facilities were projected with respect to the co-ordinate system of ortho-images of Oilton. This co-ordinate system was *NAD1983_UTM_Zone_14N*. Once the shapefiles are selected the user can preview the attribute tables of each shape file. Next the user needs to specify the co-ordinate unit of the data source. The co-ordinate unit selected was'*meters*'. The check boxes "*Create nodes if none found at the end of the pipeline*" and "*Establish connectivity using spatial data*" need to be selected and the tolerance is entered as 1m. This option connects pipe nodes which are in a range of one meter. The Model Builder gives you an option on whether the data should be imported as a current scenario or new scenario. For this project the option "*Current Scenario*" was selected. In the next window, the key fields used for object mapping need to be identified. These are fields that should be identified based on the unique ID they possess. Therefore, the fields selected for each shape file are:

Appurtenances (hydrants): Apprt_ID

Water Lines: Segment_ID

Water Supply: Supply_ID

The Model Builder then executes the build operations evaluating the user defined conditions. Once the model has been built, the user has to edit the network. The Model

Builder creates the network but does not physically connect the hydrants to the water lines. This is due to the way in which the geospatial data are created. For this reason the hydrants had to be manually connected to the nodes. Also, once the model is built, all the supply facilities are converted into water tanks, including the reservoirs. The tanks representing the reservoirs need to be physically changed to reservoirs. This is again due to the fact a single shapefile was created to represent all the water supply facilities.

Assigning elevations

For this purpose, the TREX wizard is used. The TREX wizard extracts elevation information from the elevation dataset file by interpolation. The data source type selected was *shapefile*. A shapefile in GIS is defined as that which is either a polygon, point or a line file type. The elevation dataset for Oilton that was downloaded was a raster file. A raster file in the simplest way can be defined as a digitized file of a photographic image. The raster file cannot be directly used to assign elevations to the nodes. The raster file has to be converted into a point shapefile before it can be suitably applied. In ArcMAP, one can convert a raster file to point shape file using the conversion tools. This takes a long time because of very large amount of continuous data that has been digitized into millions of pixels. The shape file also has a size of about 50 MB. After the conversion, in the TREX Wizard, this converted shapefile was selected. Since the original units of the data elevation set are in meters, the units of the model were also set to the SI system and then converted to US standard units. The user needs to specify the Z-coordinate as *GRID_CODE*. Once this is done, the wizard assigns elevations to each node, including the hydrants, the water tanks and the reservoirs. A table showing all the extracted elevations appears once the application is complete.

***Assigning base water demands to each node*:**

To assign base demand to each supply node, it is necessary to know the houses around each supply node. It is a multi-step procedure, which is as follows:

- **Identification of houses around each supply node**: In ArcMAP, the ortho-image for the City of Oilton was opened and the shapefile of the distribution network was overlayed on it. The 2000 Census Block shapefile was added. The number of houses in each census block were physically counted and assigned to the nearest supply node. An Excel sheet was created for demand allocation. The first column contained all the 124 demand nodes. The second column showed the number of houses assigned to that node. Currently, the population of the town is about 1200 people. The total number of houses identified was 418, giving an average count of 2.87 people per house. In the third column the number of houses was converted to the number of people by multiplying by the above conversion factor.
- **Conversion of the number of houses into the amount of water**: The amount of water consumed daily by the town is 118,000 GPD or 82 gpm. Then the fraction of the demand required for those houses around a particular supply node is determined by the following equation.

  Base Demand for a supply node = (Population served by that node)/(Total Population)* 82 gpm.

- In the same excel sheet all the nodes were thus assigned, base water demand using the above equation. Appendix A shows the calculations for assigning base flow demands to each node.

Hydraulic modeling in WaterCAD

This section describes how all the model parameters, scenarios and alternatives necessary to run the model were set:

Setting the elevation for the groundwater well

From the information received from Oilton, each of their wells was about 500 ft deep. The static water level of the wells was unknown. However, The Public Works Director Mr. Bruce Coldiron in his interview on July 7th 2009 said that he presumed that the water in the well was found 350 ft below the surface level. Therefore, for both the wells the static water elevation was assumed to be 350ft below the ground surface. The ground surface elevation at the location of the North well is 877.11 ft (See Figure 4). The elevation for the reservoirs was thus set as the difference between this ground surface elevation and 350ft.

Setting Pump data

The pumps on North well and South well have 30HP and 25 HP rating respectively. These both are submersible pumps. Since this is the only information available on the pumps, they were assumed to be located at an elevation of 405 ft above the mean sea level. This elevation of the pump was also selected based on the information provided by Mr. Bruce Coldiron, where he assumed that the submersible pump was at least 400 ft deep. The most important parameter simulating the operation of the pumps is the pump curve. The information regarding the manufacturers was unknown, therefore suitable pump curves for the flow and the head required were found. Figure 11 shows the series of pump curve used (Flint and Walling, 2009). In *Components* option, the user defines the Pump Curves. Then the user can create more than one pump curve. Each pump curve has a unique ID. The option of multipoint data curve was selected and all the data points corresponding to the selected pump curve were entered in the table. The graph of the curve can be previewed in the window below the table, as shown in Figure12. Then the pump on the drawing was selected. Double clicking on the object opens the Properties Editor. In the Properties Editor, under the Pump Definition, the user can specify the pump curve by using the scroll bar and selecting the ID of the desired pump curve created. This sets the environment for simulating the pumps.

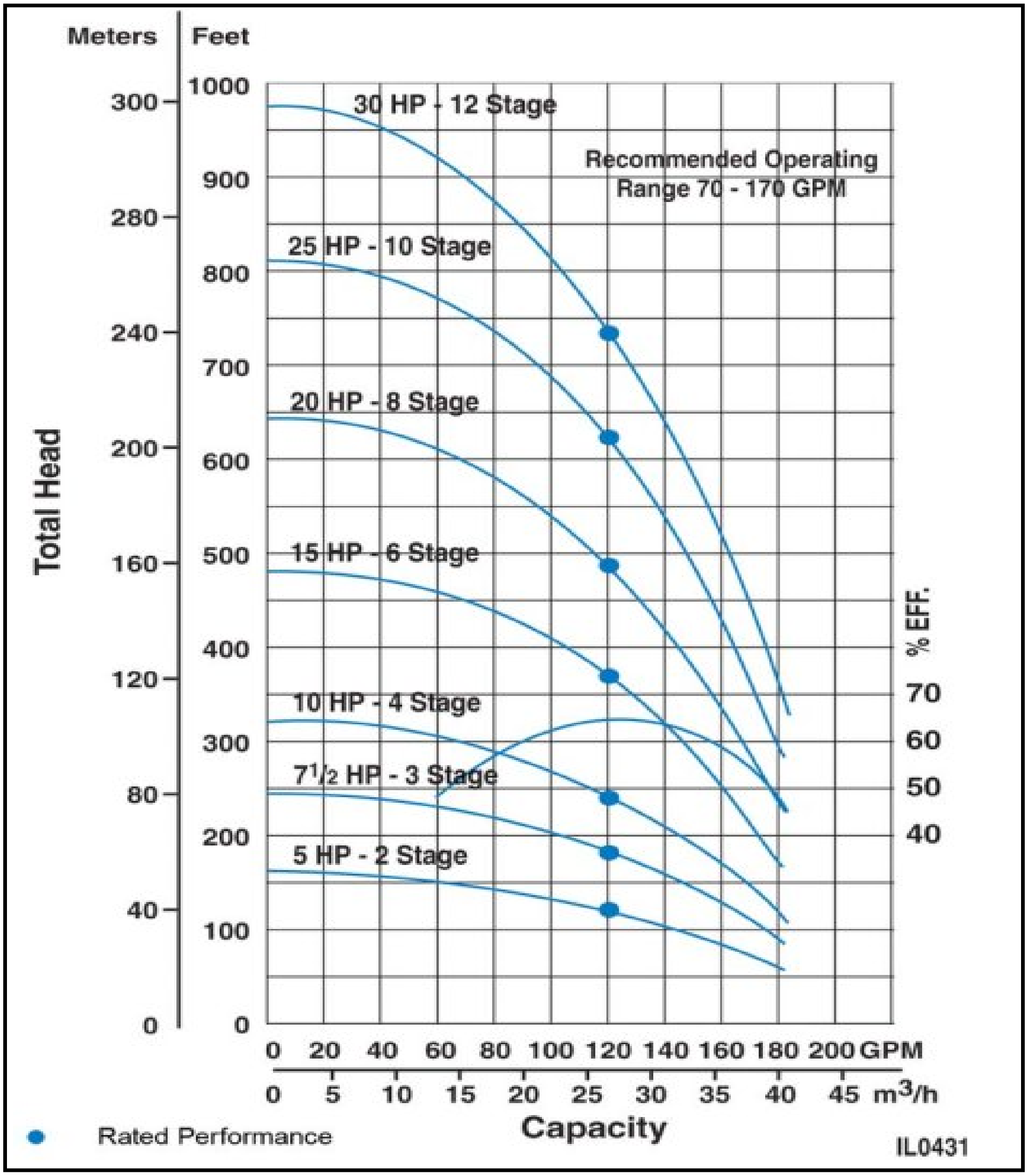

Figure 11: Pump curve for 30HP submersible pump (Source: Flint and Walling, 2009)

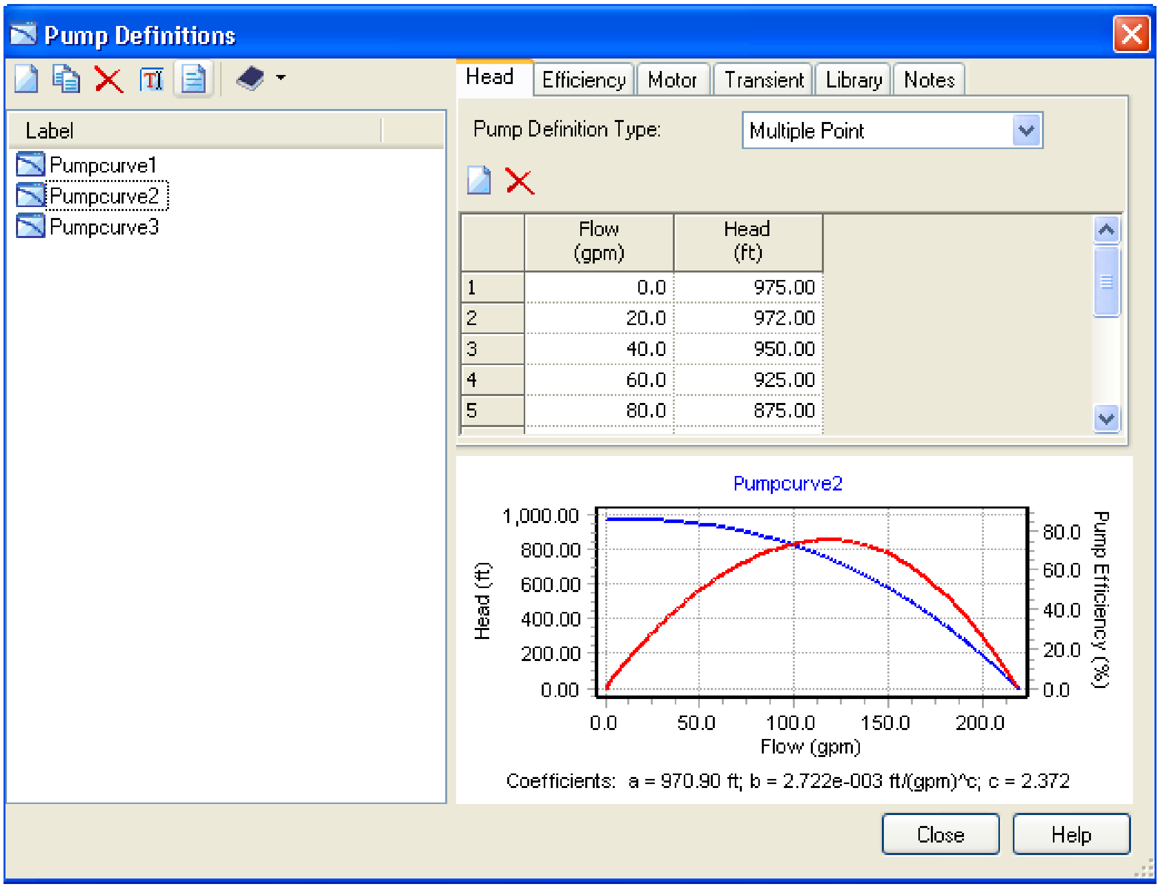

Figure 12: Screenshot of the Pump Definition Window.

Assigning base demands to each node

Each node was assigned a demand manually. In the Properties Editor of the nodes, under the *Demand* option, the user can click on the ellipsis (..). Then a window opens which shows demand in gpm and demand pattern. The value of the base demand was entered under the demand column for all the nodes.

Assigning roughness coefficients to pipelines

Hazen-William roughness factors were used to incorporate frictional losses. WaterCAD has an engineering library where different friction factors for different pipe materials are stored. In the Properties Editor for pipes one can select the pipe material. The attribute tables for pipe lines shapefile had the information regarding the pipe material. By default, WaterCAD considers that the pipeline is new ductile iron pipe. Generally, pipe made of materials such as steel, PVC and asbestos cement do not tend have as much deposition or corrosion as cast iron pipes (North American Pipe Corporation, 2009; Niquette 1999). The detailed hand drawn maps classified according to the age were provided by the city. These maps were used to adjust the friction factors. This is summarized in Figure 13. Figure 13 shows a table of Hazen-William friction factors classified by line sizes and age and degree of attack. Attack on the pipe is defined as the corrosion of the pipe and greater the ‘C’ factor greater is the smoothness and the carrying capacity of the pipe (Haestad Methods, 2003). This table was used to assign the respective friction factors.

C-factor Values for Discrete Pipe Diameters						
Type of Pipe	1.0 in. (2.5 cm)	3.0 in. (7.6 cm)	6.0 in. (15.2 cm)	12 in. (30 cm)	24 in. (61 cm)	48 in. (122 cm)
Uncoated cast iron - smooth and new		121	125	130	132	134
Coated cast iron - smooth and new		129	133	138	140	141
30 years old						
Trend 1 - slight attack		100	106	112	117	120
Trend 2 - moderate attack		83	90	97	102	107
Trend 3 - appreciable attack		59	70	78	83	89
Trend 4 - severe attack		41	50	58	66	73
60 years old						
Trend 1 - slight attack		90	97	102	107	112
Trend 2 - moderate attack		69	79	85	92	96
Trend 3 - appreciable attack		49	58	66	72	78
Trend 4 - severe attack		30	39	48	56	62
100 years old						
Trend 1 - slight attack		81	89	95	100	104
Trend 2 - moderate attack		61	70	78	83	89
Trend 3 - appreciable attack		40	49	57	64	71
Trend 4 - severe attack		21	30	39	46	54
Miscellaneous						
Newly scraped mains		109	116	121	125	127
Newly brushed mains		97	104	108	112	115
Coated spun iron - smooth and new		137	142	145	148	148
Old - take as coated cast iron of same age						
Galvanized iron - smooth and new	120	129	133			
Wrought iron - smooth and new	129	137	142			
Coated steel - smooth and new	129	137	142	145	148	148
Uncoated steel - smooth and new	134	142	145	147	150	150

Figure 13: Table showing the C- factors for different line sizes

(Source: Haestad Methods 2003)

Type of Pipe	C-factor Values for Discrete Pipe Diameters 1.0 in. (2.5 cm)	3.0 in. (7.6 cm)	6.0 in. (15.2 cm)	12 in. (30 cm)	24 in. (61 cm)	48 in. (122 cm)
Coated asbestos cement - clean		147	149	150	152	
Uncoated asbestos cement - clean		142	145	147	150	
Spun cement-lined and spun bitumen- lined - clean		147	149	150	152	153
Smooth pipe (including lead, brass, copper, polyethylene, and PVC) - clean	140	147	149	150	152	153
PVC wavy - clean	134	142	145	147	150	150
Concrete - Scobey						
Class 1 - Cs = 0.27; clean		69	79	84	90	95
Class 2 - Cs = 0.31; clean		95	102	106	110	113
Class 3 - Cs = 0.345; clean		109	116	121	125	127
Class 4 - Cs = 0.37; clean		121	125	130	132	134
Best - Cs = 0.40; clean		129	133	138	140	141
Tate relined pipes - clean		109	116	121	125	127
Prestressed concrete pipes - clean				147	150	150

Lamont (1981)

Figure 13: Table showing the C- factors for different line sizes continued

(Source: Haestad Methods 2003)

Assigning demand patterns

The *Components* tab has an option called 'Patterns' which opens the Pattern Manager window. The user can use this pattern manager to create water usage patterns based on daily, weekly and monthly use. For the maximum hourly demand, a multiplier of 5 was used. Generally, the peak hourly flow is 3-6 times the average daily flow (Haestad Methods, 2003). Also, a study of small community in Illinois showed that their peak hourly flow demands were about 6 times the average daily flow demand (Salvato, 1992). Thus to test for worst conditions, the factor of 5 was selected. The peak hours were considered to be 7 am to 9 am in the morning and 6 pm to 8 pm in the evening. A demand pattern for the school was created considering the number of students in the school and then the per capita consumption multiplied by the number of students was assigned to the node providing water to the school. The school was assumed to be open from 9 am to 4 pm on weekdays and was assigned a pattern of thrice the average residential demands during that period. The school remained closed from May to July, and during those months, demand was considered zero (Green, 2009). Fire flow pattern was assigned to all the hydrants. The base demand is 0 gpm. In case of fire the demand is 250 gpm. The pattern was assigned to provide this 250 gpm at peak hours between 7 am and 9 am. In Analysis tab, one can go into the Alternatives options. Under '*Demand Alternatives'* option one can assign the specific pattern and the base demand to a nodes.

Operating on Rules:

There needs to a set of binding conditions that will control the pump operation. According to the information provided by the city, the pumps are triggered by the automatic level switches in storage tanks. The highest permissible water level in the tank outside the town was 60ft and thus the rule written for this tank was

If Tank WT1 level >= 58 ft, Pump PMP-1 Status = Closed

Since the diameter of these tanks is really large it takes a long time to fill these tanks. Also it was necessary that these tanks remain half filled for maintaining the necessary pressures. Thus, the rule written to trigger the pump on was:

If Tank WT2 level <= 30 ft then Pump PMP-1 Status = Open

These rules were entered using the *Controls* options under the Components tab

Figure 14 shows the figure of the distribution network in the town

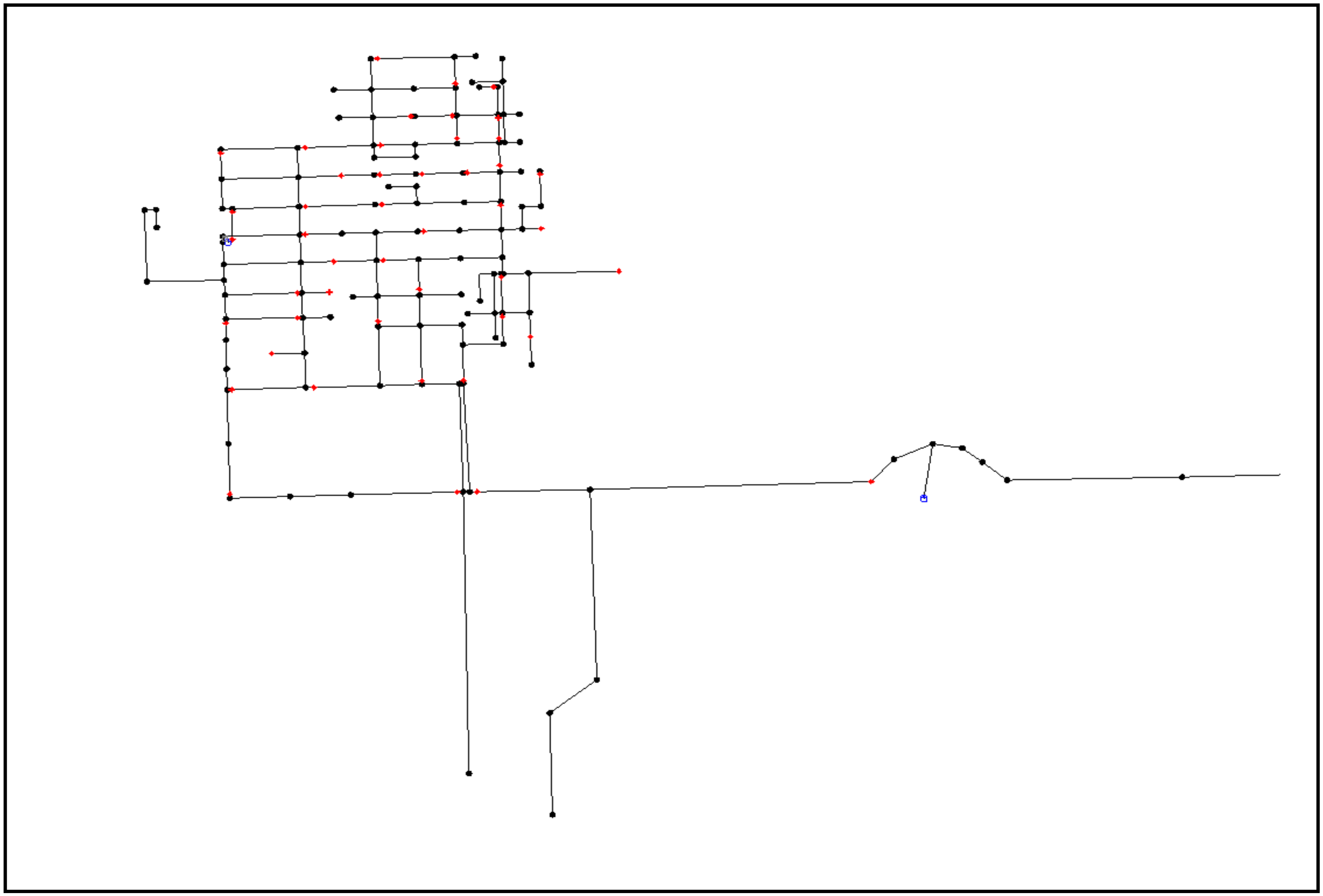

Figure 14: Distribution Network of in the town

CHAPTER IV

ANALYSIS AND FINDINGS

Introduction:

In this chapter, the discussion of the particular findings after the analysis of the distribution system was done will be presented. Any system will behave acceptably, if its parameters operate within a certain acceptable range. Similarly, to check the performance of the distribution system of Oilton, the following conditions were checked in the model:

- Provision of average day flow maintaining the pressures.
- Provision of peak hourly flows maintaining the pressures.
- Provision of fire flow during peak hours in the day.
- Unusually high pressures and low pressure in the system.
- Tank refilling at the start of the pump cycle.
- Low velocities in the pipes.
- Water quality with respect to water age and chlorine residuals in the system.

Assumption for Analysis of the Current Scenario

The conditions assumed under the current scenario were as follows: Both the tanks were half full at the start of the simulation. When the level in the tank WT1 dropped below 30 ft, it triggered of PMP-1 and when the level in Tank WT1 is greater than 58ft, then the pump was turned 'OFF'. The only data available on the pump was that it was a 30 HP submersible pump. Since no other information was available on the pump a suitable pump curve was selected. The pattern pump curve is as shown in Figure 15. The operating point of this pump is 170 gpm at 450 ft and it was modeled as a constant speed pump. Also, as mentioned previously in Chapter 3, there was no data available on the static level of the well. Therefore the level of the water was assumed to be 350 ft below the ground surface. It was assumed that the well can tolerate high pumping rates, considering the fact that Oilton has never had problems with the wells running dry. With reference to the conversation with Mr. Bruce Coldiron, the South well was not being pumped. Based on this information, the South Well was considered inactive throughout the analysis. WT1= Standpipe in the town; WT2= Standpipe outside the town.

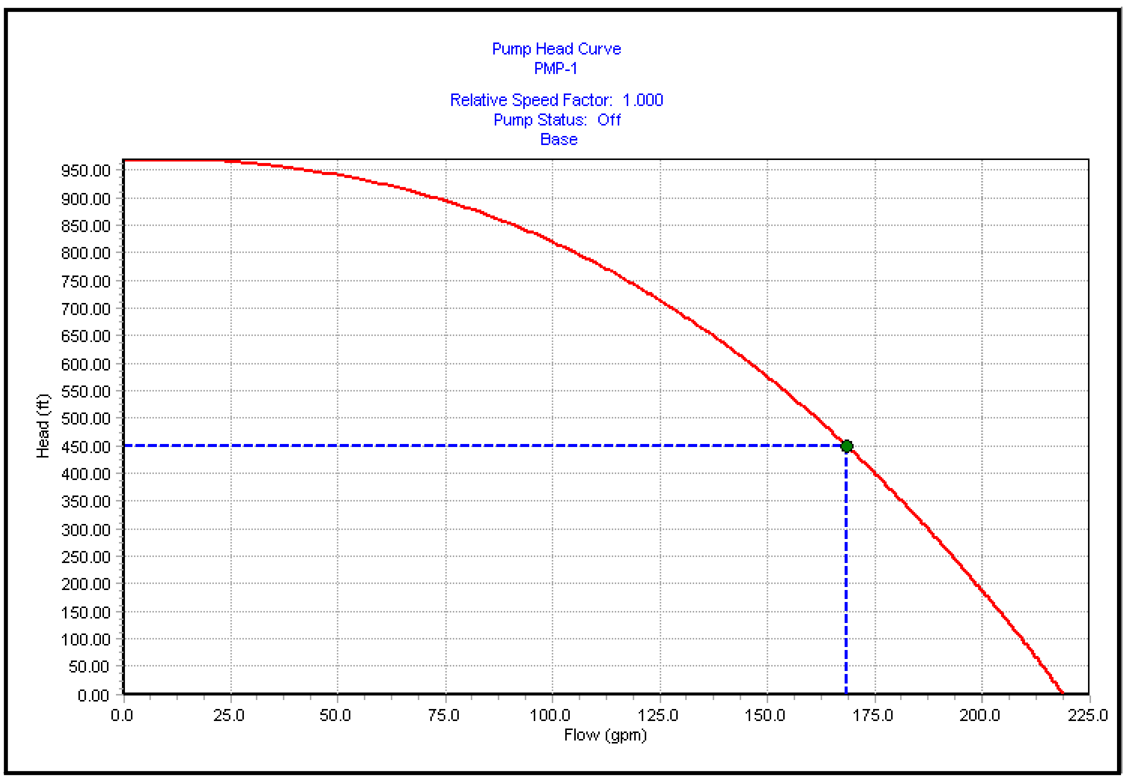

Figure 15: Pump curve for the operating conditions of the pump.

Analysis for base flow conditions

The system was initially checked for base flow conditions. Figure 16 shows the graph of pump flows and the fluctuations in the tanks levels WT1 and WT2 for base flow conditions. Since the controls were set to start the pump if the level in Tank WT1 was below 30ft, therefore the pump status was “ON” at the start of the simulation. It was observed that in the first pump cycle the tank WT1 reached its maximum level in 3 and ½ days. In the next successive pump cycles, the pump ran for 2 and ¾ days and remains off for about 1 and ¾ days. The highest water level allowed in tank WT2 is 126ft. This level was never reached because of the level control set on tank WT1. The highest water level observed in tank WT2 was 95 ft and the level fluctuated between 95 and 73 ft during the pump cycles. Under the conditions of base flow demands there were no unusually high pressures observed anywhere in the town, neither there were any low pressure areas. The pressures in the town increased with the start of the pump cycle and then stabilized and varied between 30 and 42 psi for every successive pump cycle. The velocities in the pipe were in the range of 0 to 3.3 ft/s. The water age was also checked for the base flow conditions; the water age took the maximum time to stabilize. It took about 39 days for the water age to stabilize in tank WT1 and about 67 days for the water age to stabilize in tank WT2. The oldest water calculated in tank WT1 was 10 days old and in tank WT2 was about 17 days. Since the age of the water appeared to be rising for an extended period simulation of one month, the simulation period was increased to 90 days. Figure 17 shows the graph of the water age in tanks WT1 and WT2.

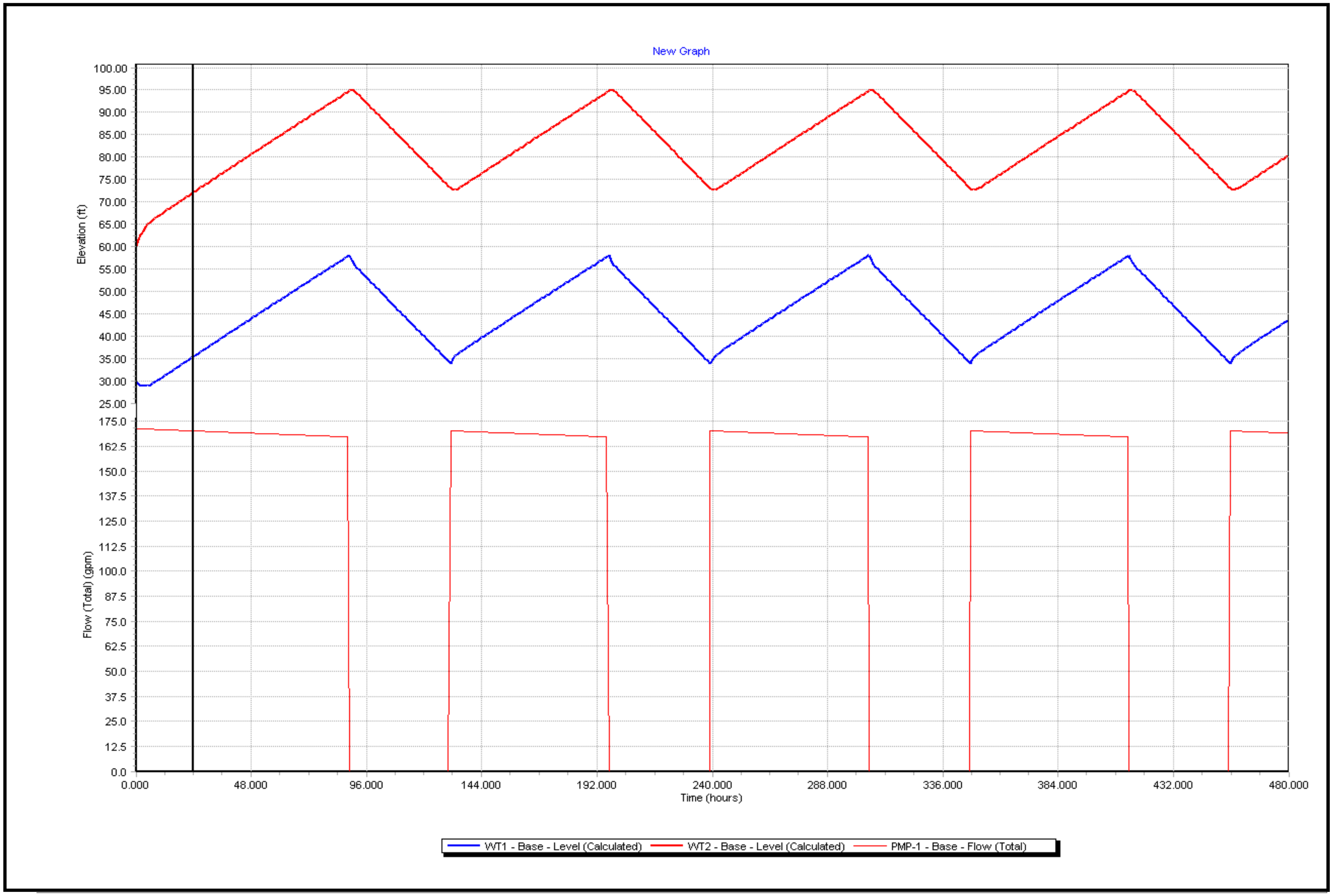

Figure 16: Graph of pumps cycles and level fluctuations in the tank under base flow conditions

However, it is more important to check for the chlorine residuals in these tanks. Oilton uses a dose of 4mg/l of chlorine for disinfection. The initial concentration of chlorine at the start of the simulation throughout the system was modeled to be 0 mg/L to test for the worst case. Figure 18 shows the graph of the chlorine residuals in both the tanks and thus gives a detailed picture of water quality calculated in these tanks. According to the simulation, the chlorine residuals in the tanks were observed to be above the lowest allowable limit, which is 0.2 mg/L as stated by the Oklahoma department of Environmental Quality (ODEQ, 2009). Thus under base flow conditions the tanks were observed to have sufficient chlorine residuals to meet ODEQ standards.

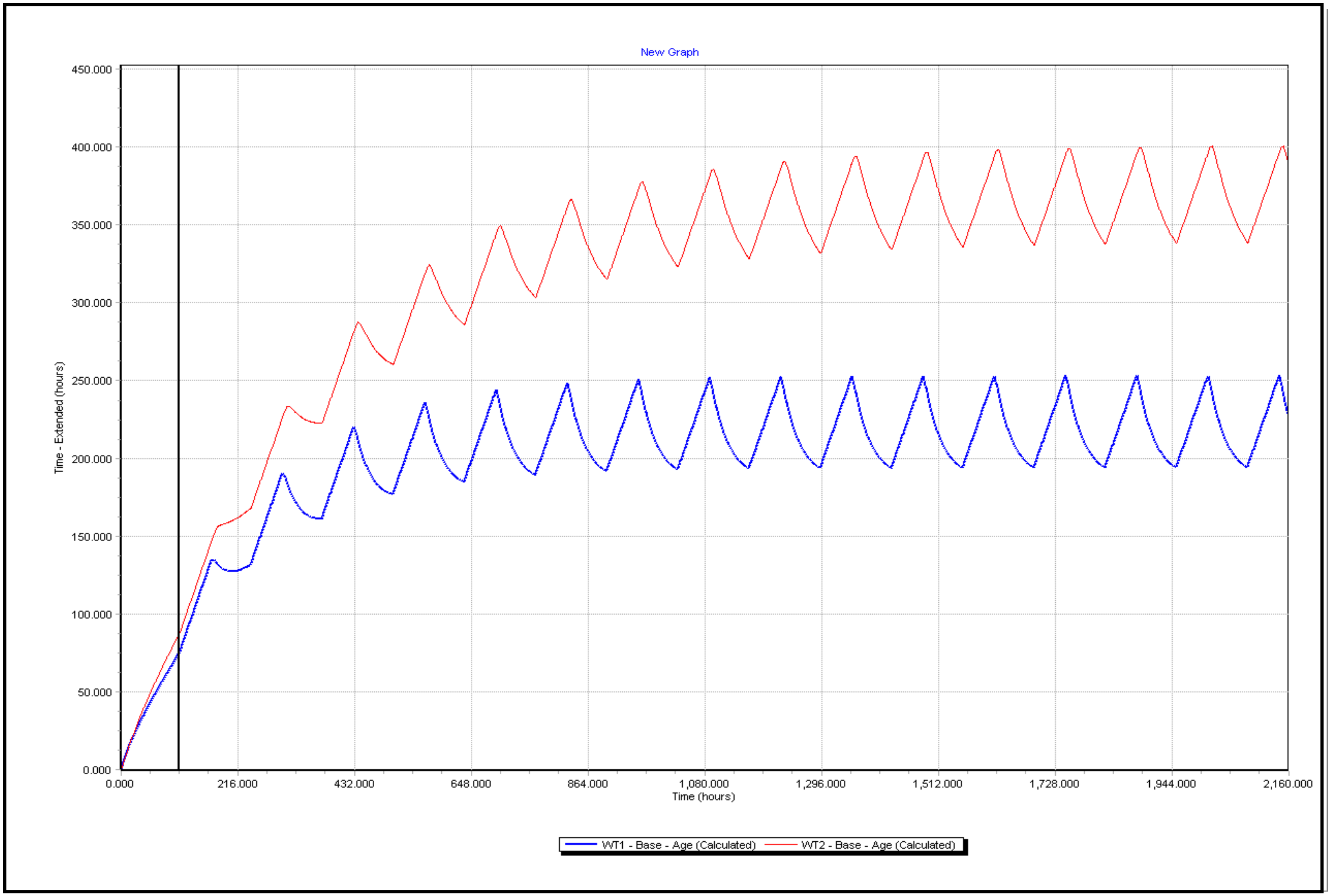

Figure 17: Graph of water age in the storage tanks WT1 and WT2

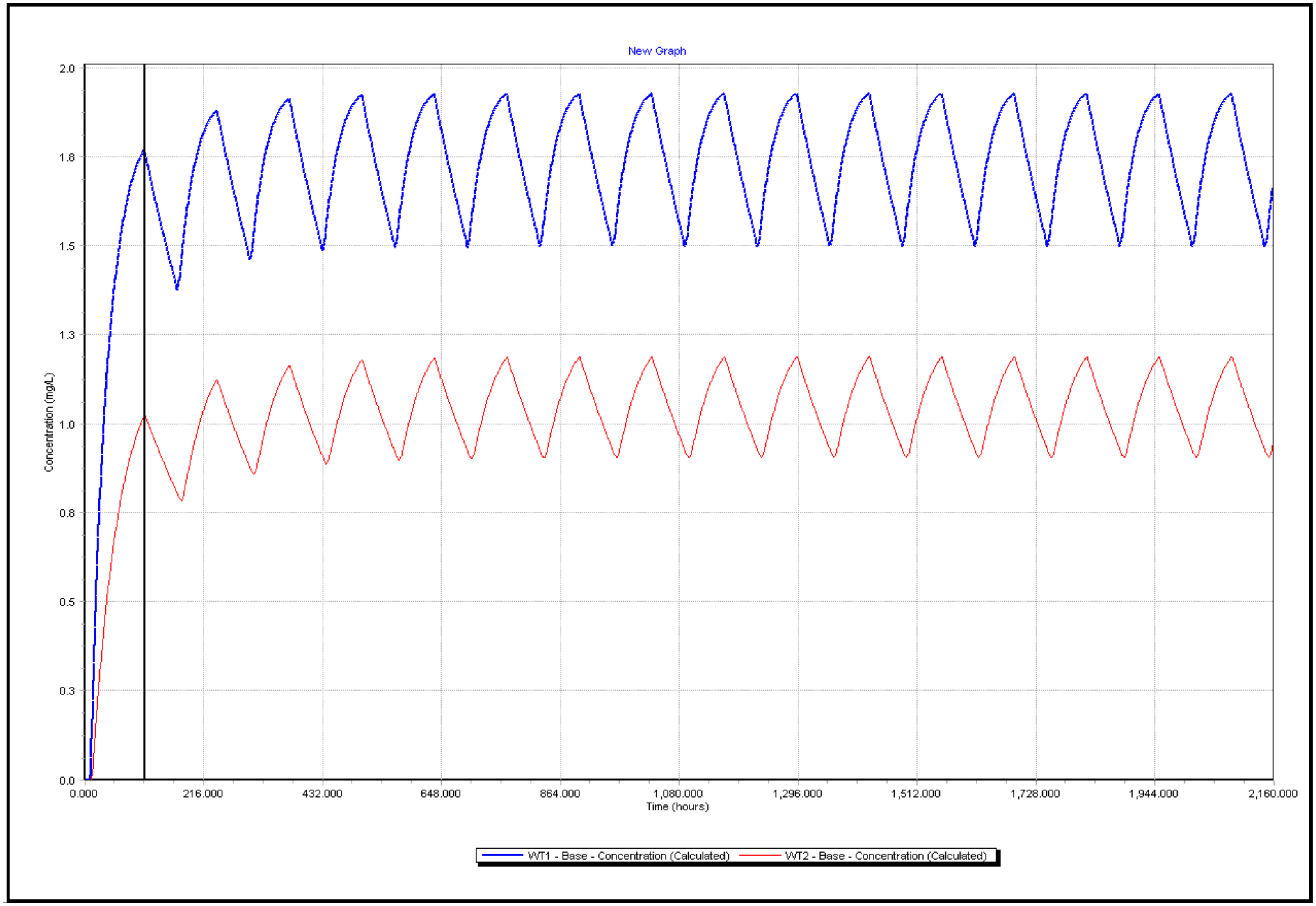

Figure 18: Graph of chlorine residuals in the tanks WT1 and WT2

Analysis of Peak flow conditions

The peak flow factors are generally lie between 3-6 times the average daily flow (Haestad Methods 2003).As previously explained in Chapter 3, for Oilton it was assumed to be 5 to check for the most severe case. Under this condition, tank WT1 level fell from 30 ft at the start of the simulation dropped to 25 ft. The peak period was considered to be from 7am to 8am. During this period the total demand by the system was calculated as 5 x 82 gpm = 410 gpm. During the peak periods, the tanks did not fill up. Figure 19 shows the graph of the levels in the tanks during the peak period.

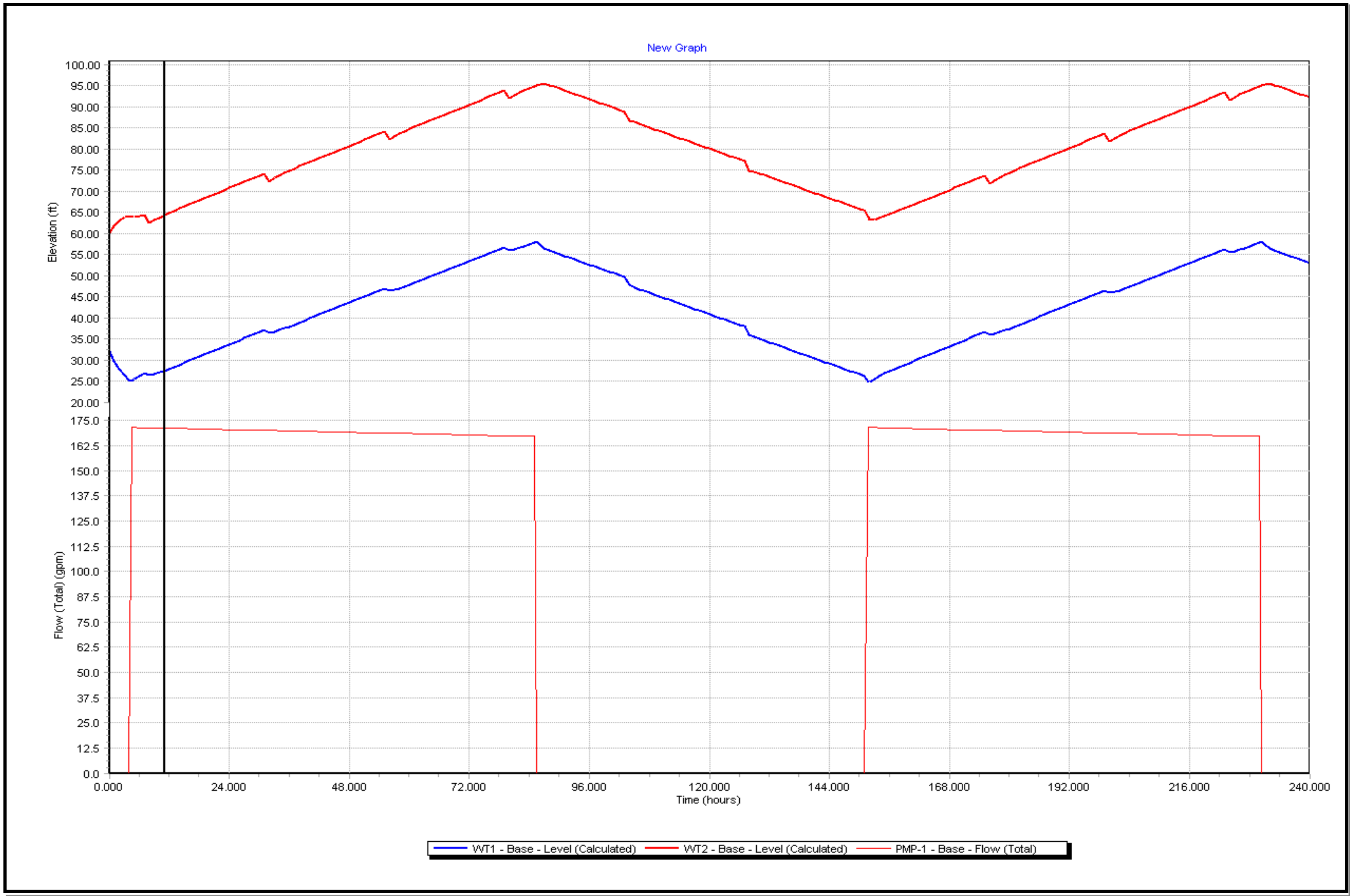

Figure 19: Graph of pumps cycles and level fluctuations in the tank under peak flow conditions

It is more critical to test for fire flow adequacy during the peak flow condition because ISO mitigation fire flow test conditions specify that for class 8 or better, the system should be able to provide a flow of 250 gpm at peak flow conditions maintaining a pressure of minimum of 20 psi (ISO Mitigation Online, 2009). City of Oilton has an ISO Fire Class 6. During this time the total water demand by the system would be equal to (5 x 82) + 250 gpm = 660 gpm. One hydrant in the town failed to satisfy this condition. This hydrant is located on the western most water line of the town. The pressure dropped below 12.5 psi when the demand was 250 gpm as shown in Figure 20. To see if the pressure conditions could be improved around node, the level in Tank WT2 was raised to its maximum of 126 ft. When tank WT2 was initially full, the pressure at the hydrant was about 23 psi at the start of the simulation. However, the condition of Tank 1 remaining full did not occur most of the times and under thus, the hydrant failed to provide 250gpm at 20 psi. Figure 21 shows the pressure at the fire hydrant for a demand of 250 gpm when the level in tank WT2 was initially at 126 ft. An extended period simulation of 240 hours was carried out to see the different pressures at the hydrant at different levels in this tank. When the level in the tank was greater than 80 ft, the hydrant could provide a pressure of 20psi and greater. Overall, all other hydrants, especially those located at key regions in town, like near the school and on Main Street, satisfied the fire flow demands.

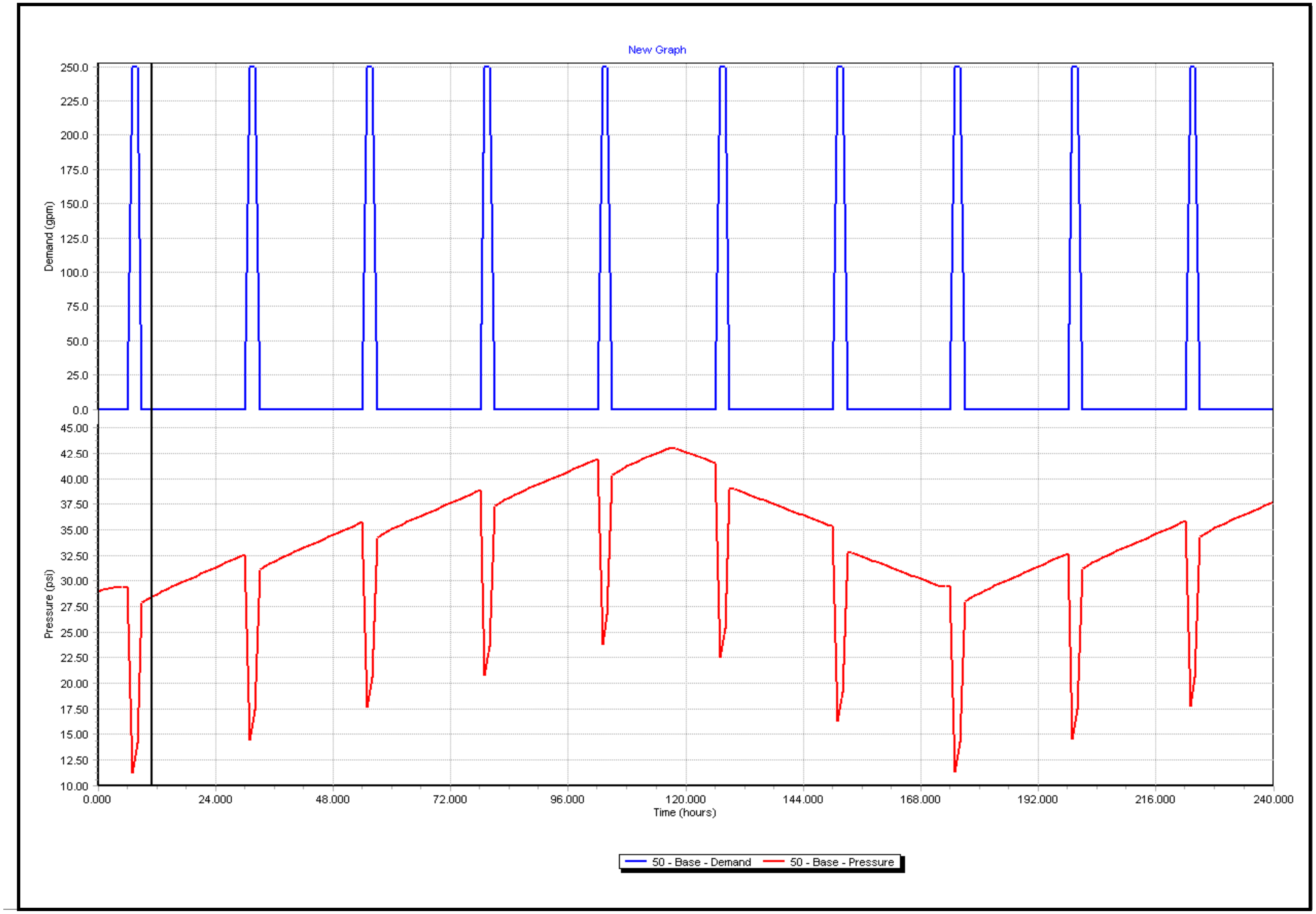

Figure 20: Fire flow hydrant test for hydrant 50 located on the western-most pipeline

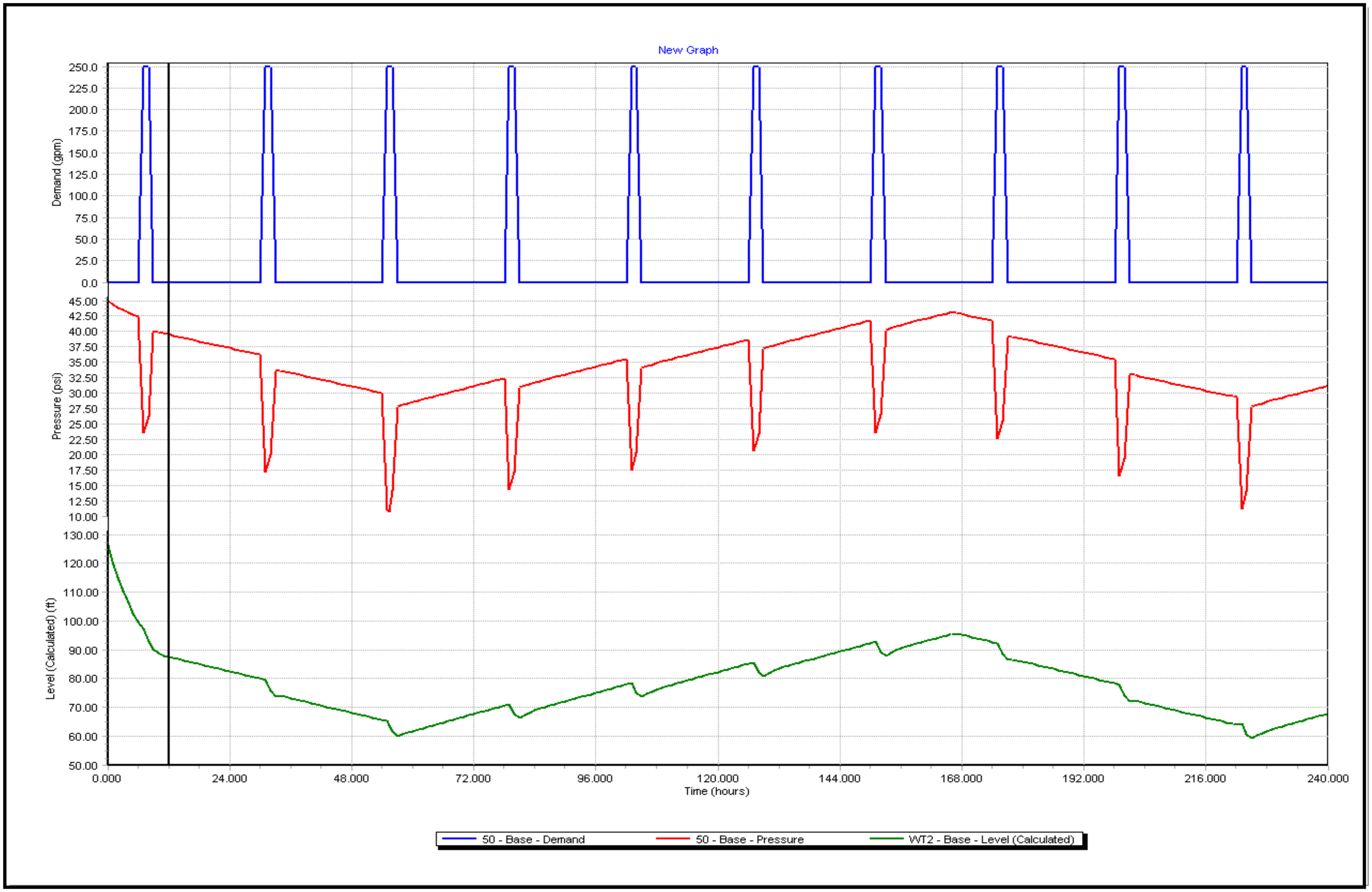

Figure 21: Graph of extended period simulation to see if hydrant satisfies the test when level in the tank WT2 is intially full

Diurnal Analysis

It is more appropriate to analyze this system under the daily flow conditions to understand its dynamics. Thus, a daily flow pattern was applied to every node. Figure 22 shows the pattern for water usage over the course of the day

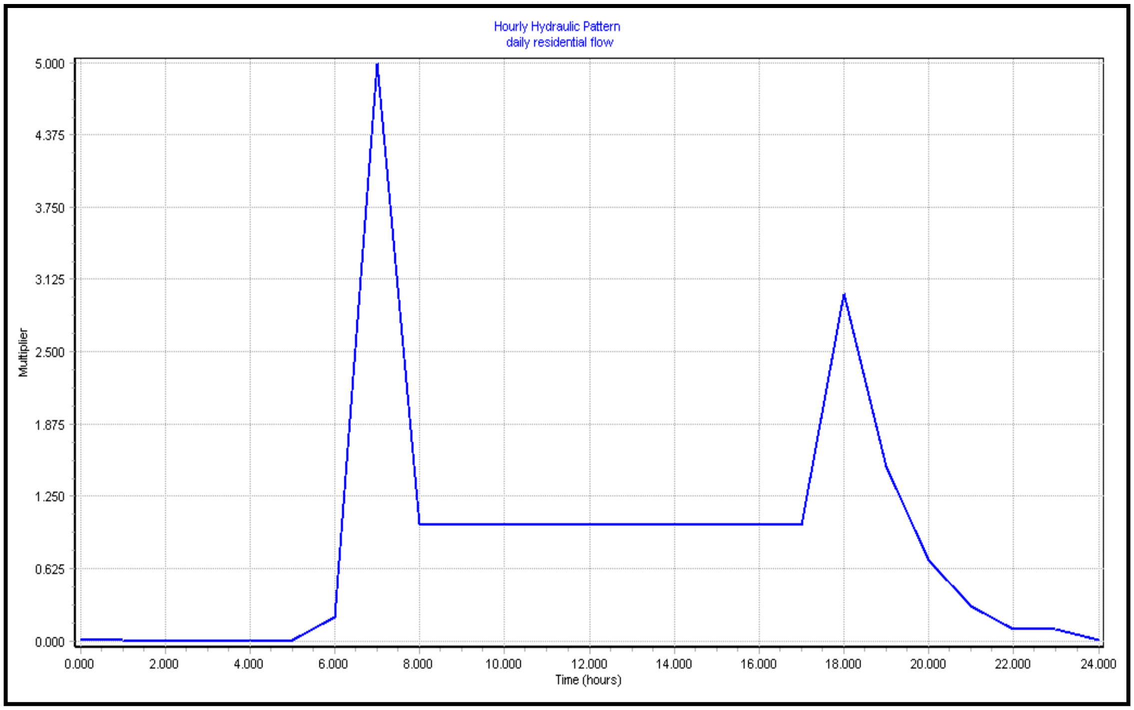

Figure 22: Diurnal pattern for current scenario

Under this set of conditions, the tank WT1 took 2 and 1/2 days to reach its maximum level of 58ft. Therefore the pump ran for 60 hours and remained off for about 55 hours. In successive pump cycle the pump ran for 42 hours and then remained off for 60 hours. Figure 23 shows the graph of the pump cycles and the fluctuations in tank WT1. Additionally, in the diurnal analysis the water demand from the school was taken into consideration and accordingly the demand pattern for the school was applied to the node J-38 which supplies water to the schools. The location of this node is shown in Figure 24. The school is assumed to be open from 9am to 4pm and the demand assigned to the school is the total number of enrollments times the per capita usage. The number of student enrollments are 369 (Green, 2009) and the per capita water consumption of the city is 0.065 gpm. Thus the base flow demand at the school node was 24 gpm. The pattern is assumed to be fixed during the school hours for case of simplicity.

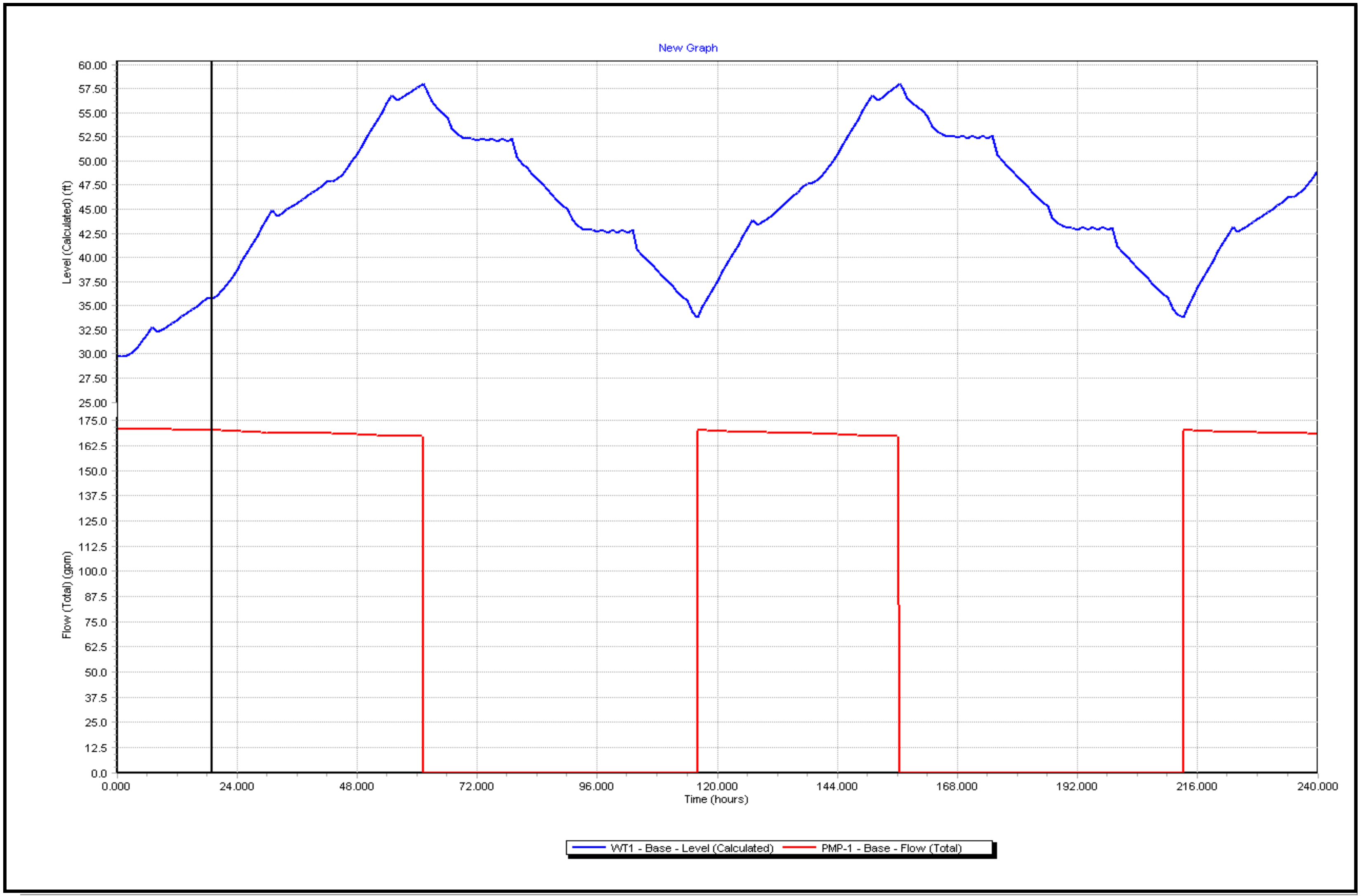

Figure 23: Graph of pump cycles and change in water level in tank WT1 under diurnal pattern

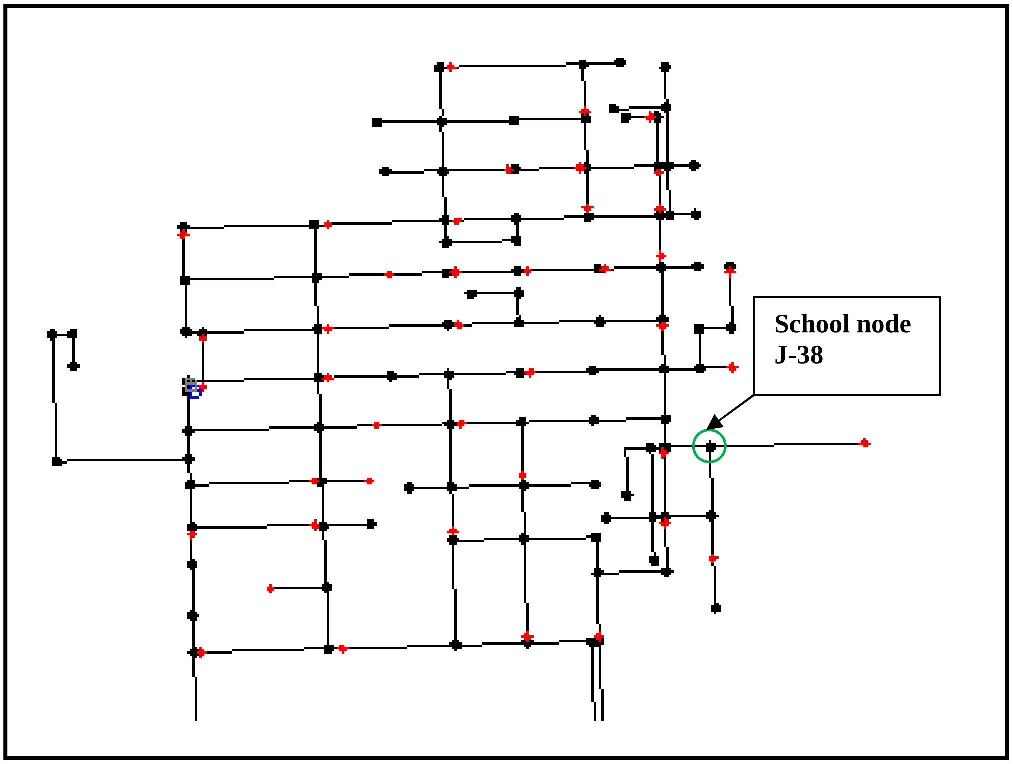

Figure 24: Location of School Node

For diurnal analysis of the town, the low pressures in the town were observed at two junctions located at the far southern end of the town. The lowest pressures recorded then were about 13.75 psi during peak demands. These junctions are located at a relatively higher elevation compared to the other junctions in the town, and hence they will experience low pressures.

Analyzing the velocities in the pipelines

The velocity range under diurnal flow pattern for the town was in the range of 0-2.24 ft/s. There are a number of pipes in Oilton that have almost reached the end of their usable life time (See Figure 10). The friction factors in these pipes were adjusted considering their age. For example, there is a cast iron line in the town which runs on

E 1st street. It has a remaining life of 2 yrs (See Figure 10). This line was considered to have appreciable attack and accordingly was given a C- factor of 41 (See Figure 13). The maximum headloss in the pipe was 0.1 ft. When this pipe was replaced by a new pipe having a C-factor of 130, then the head loss was observed to be 0.03 ft increasing the velocity in the pipe from 0.11ft/s to 0.175 ft/s (1.6 times). This is a considerable increase in the amount of velocity, especially since the velocities are generally low in the town, and the pipes never run full because of low demands. There are a number of cast iron pipes which have a remaining usable life less than or equal to 5 years. These pipes eventually will have to be replaced to improve the performance of the system.

Water age and water quality analysis for current condition

Water age was an important factor to be observed, since the city has large storage tank capacity. Once the tanks are full, they can supply water to the town for 2 and ½ days under normal day conditions maintaining the normal working pressures greater than 30 psi. Figure 25 shows the graph of the water age in tank WT1 and Figure 26 shows the water age in tank WT2 for the diurnal pattern. Since initially an extended period simulation of 10 days showed the water age continuously increased in tank WT2, a simulation for one month was carried out. It still showed some increase. Thus a simulation for two months was run and the age stabilized after 30 days in tank WT1 and after 42 days in tank WT2. The maximum water age in tank WT1 was 10 days and that in WT2 was about 15 days.

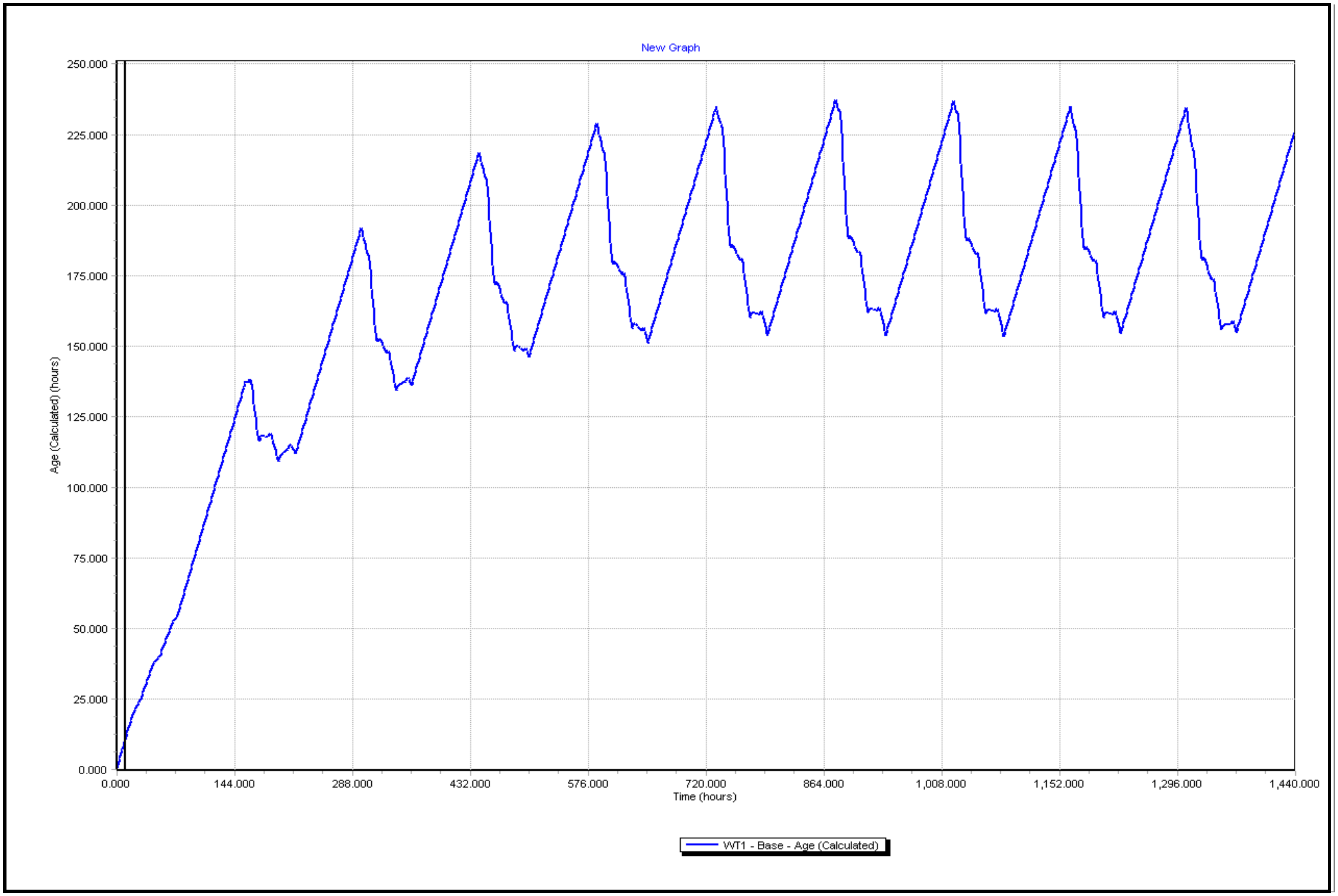

Figure 25: Graph of water age in Tank WT1

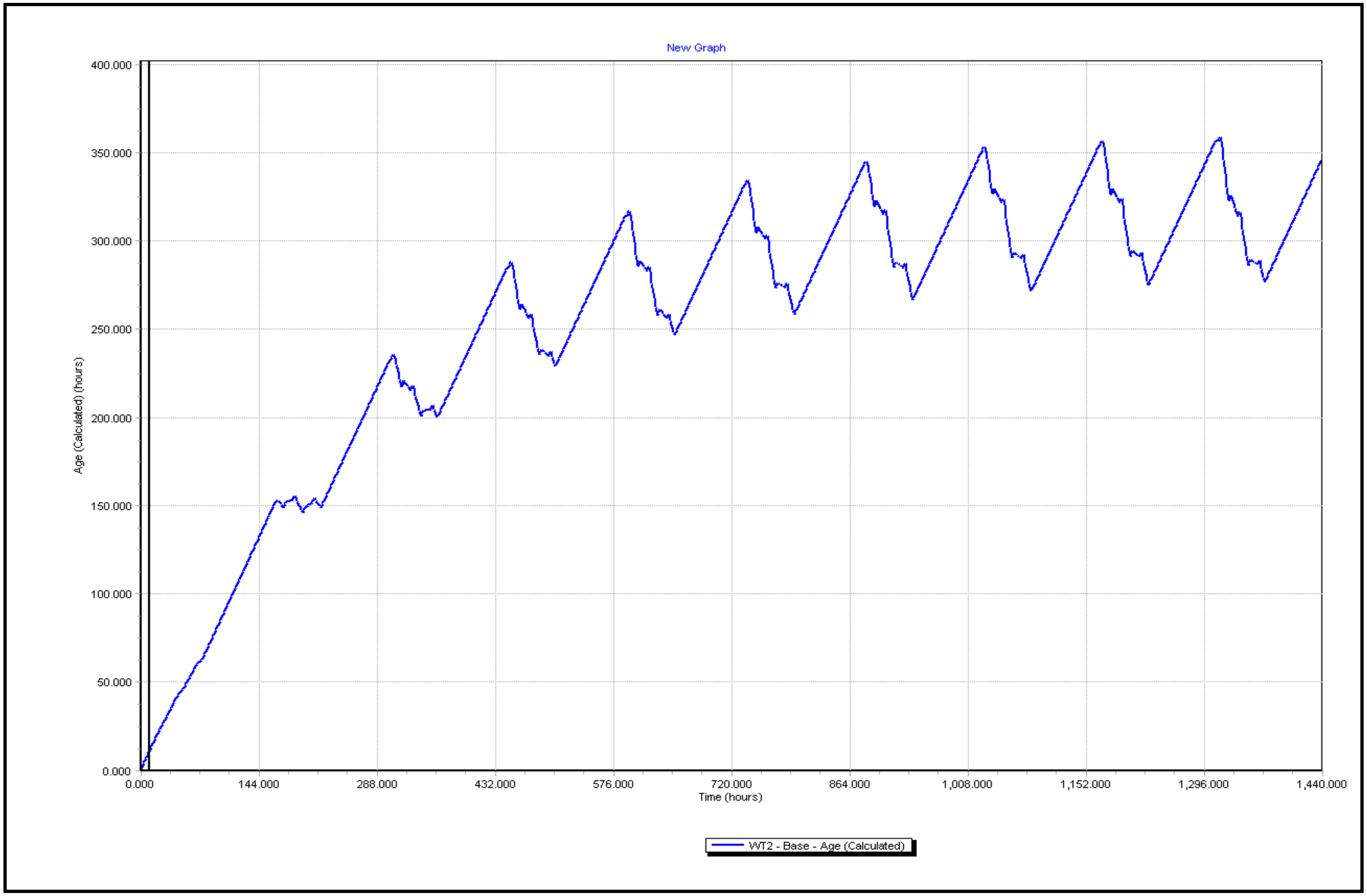

Figure 26: Graph of water age in Tank WT2

As mentioned before, Oilton uses chlorine gas for disinfection at a concentration of 4mg/L. The constituent’s concentration for a period of 2 months in both the tanks is as shown in Figures 27 and 28. The chlorine residual concentration increases in both tanks and then fluctuates with the re-filling and the draining of the tanks. However, this concentration is well above the required lowest limit of 0.2 mg/L (ODEQ 2009). The lowest chlorine residual level in the tank WT1 is about 1.495 mg/l and that in tank WT2 is about 0.975 after stabilization. Hence bacterial contamination or growth in these tanks would not be a problem.

The chlorine residual concentration was observed to drop to the lowest level of 0.8 mg/L near the school node. However, this occured when the school was out of operation. The concentration levels of chlorine residuals were low at the node J-130, which is at far south end of the town. At the start of the simulation, for a period of 72 hours, the concentration of chlorine residuals at this node were observed to be 0 mg/L. The highest chlorine residual concentration calculated at this node was 0.8 mg/L and lowest level calculated was 0.175 mg/L.

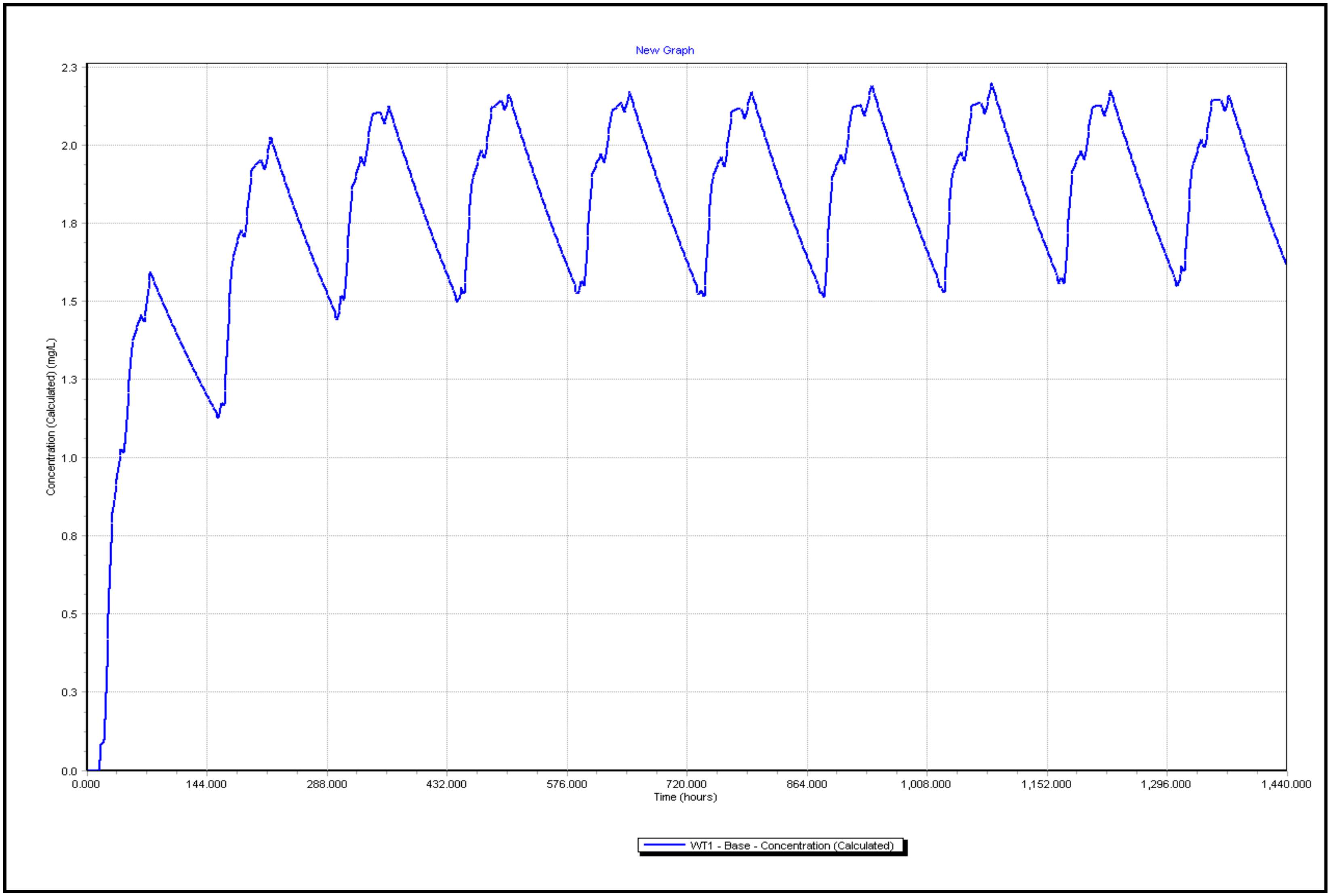

Figure 27: Chlorine residual concentration in Tank WT1

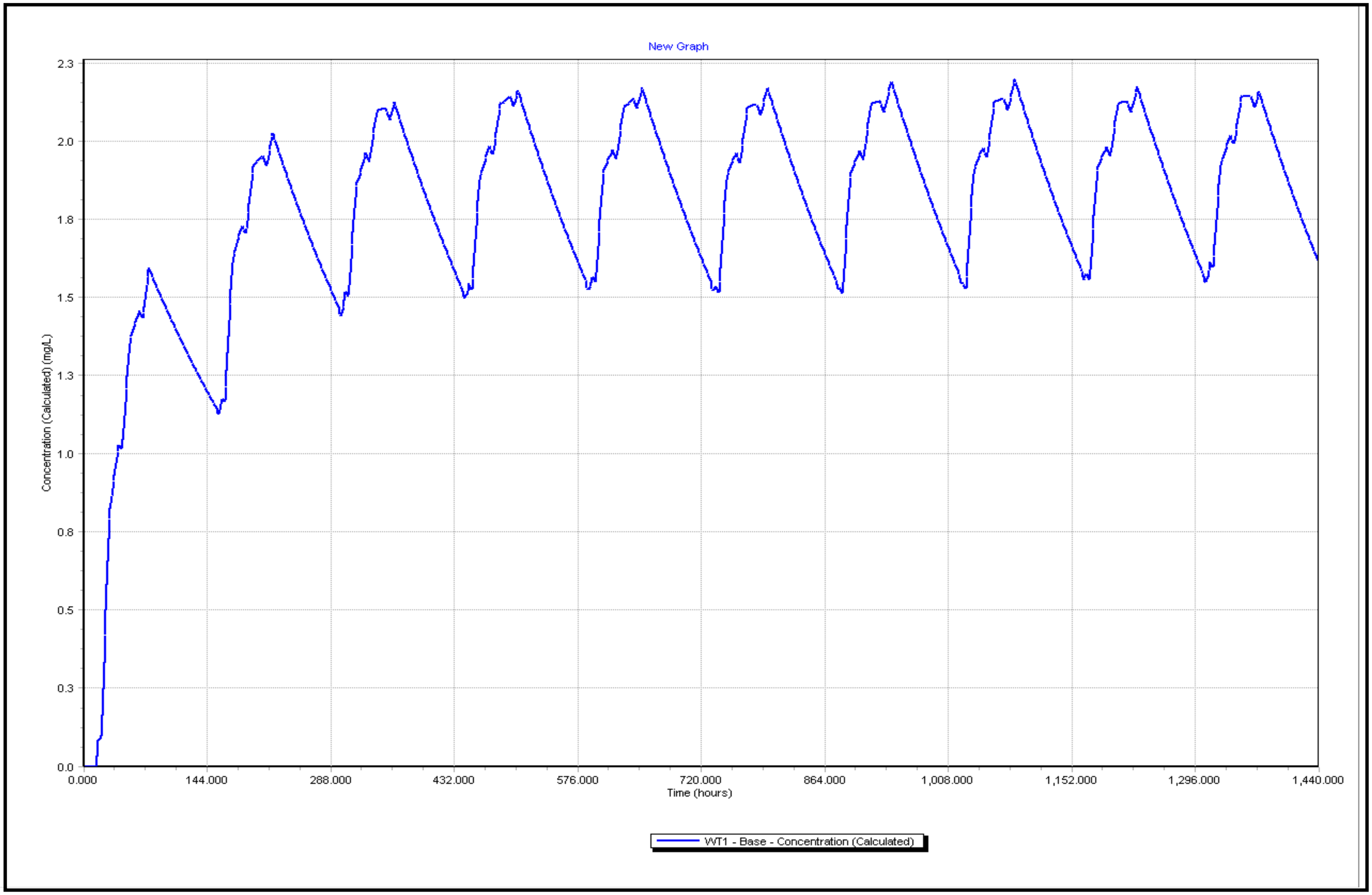

Figure 28: Chlorine residual concentration in Tank WT2

CHAPTER V

RECOMMENDATIONS AND CONCLUSION

Comments on specific findings

This section will highlight the specific findings in the analysis of the system the followed by recommendations to improve the distribution system. Though normal working pressures or water age didn't seem to be a very big problem in the city, clearly it was observed that the city has a very large storage supply of about 2 and half days for a small water demand. For the existing pump operation, which is set to start when the tank are empty and set to shut off when the tanks fill up, takes a very long for the tanks to fill. The tanks should ideally fill up within 6 to 12 hours of pump cycle (Salvato 1992). The inspection report of the tanks from the town mentions that the cost of replacing each tank on an average would be about USD 500,000. It is unreasonable to replace these tanks. Thus for these tanks to fill relatively quickly, a larger pump would be required. Accordingly various 50 HP pumps were tried. The pump curve for a representative pump is shown in Figure 29.

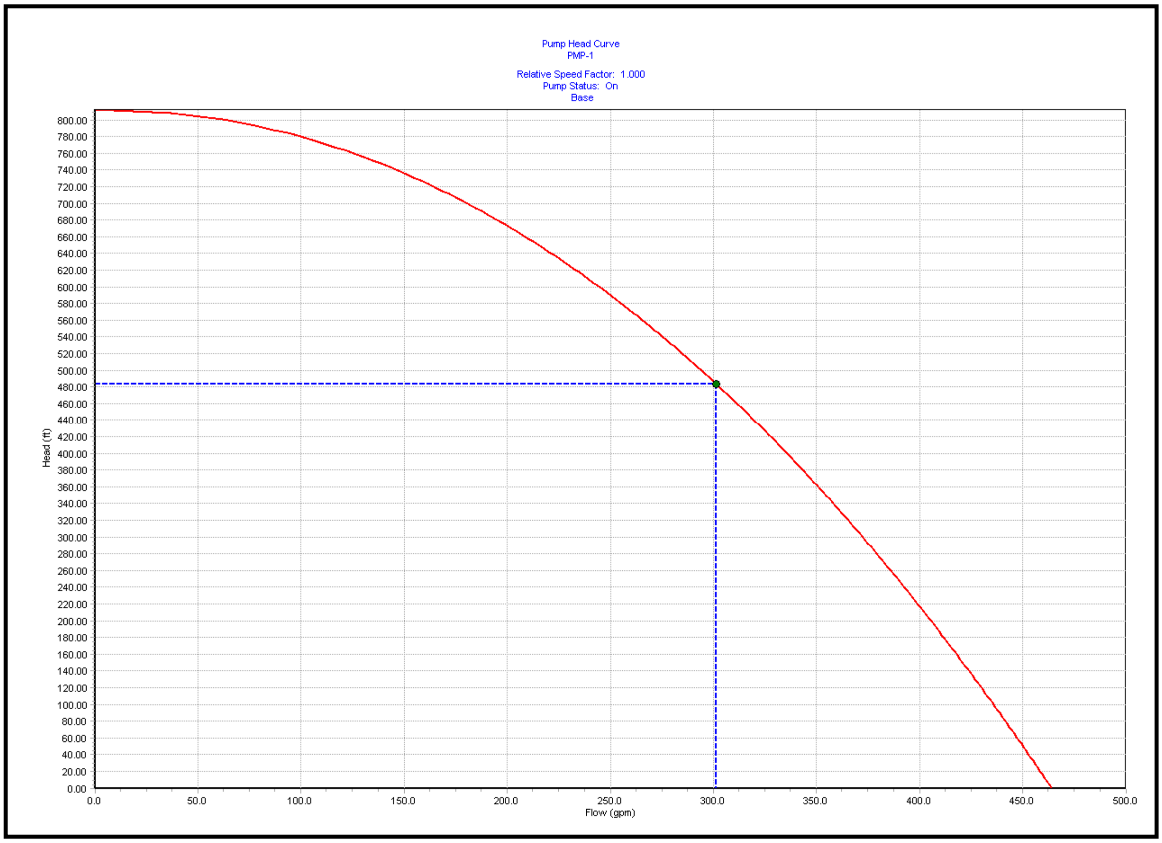

Figure 29: Pump curve for the recommended pump

The pump provided a flow of 300 gpm at 400 ft. The graph of the pump cycle for this pump is shown in Figure 30. From the graph, it can be clearly seen that the pump operates for about 26 hours and then remains off for about 60 hours or 2.5 days. For successive pump cycles, the pump runs only for about 16 to 18 hours. Using a large pump reduces the time required to fill the tanks by 3 times. This is very significant because, it would provide better condition where the pump operates for a short period and tanks take a long time to drain. Water quality is not going to be an issue in any case, because they would be filling out and emptying a volume of about 30ft in both the tank within 3 days. Alternatively, if they want to use the same pump, then they could run their pumps for duration of 12 hours. Figure 31 shows the graph of the pump cycles and the tank level fluctuations. Under this condition the tank now operates within a level difference of maximum 12ft. The concentration of chlorine residuals was found to remain in the acceptable range. Thus this solution works if they do not wish to operate their tanks at the highest levels. However, the best solution considering that the city has redundant storage system, would be a larger pump which can fill the tanks and then remain closed for a longer duration as mentioned in the previous solution.

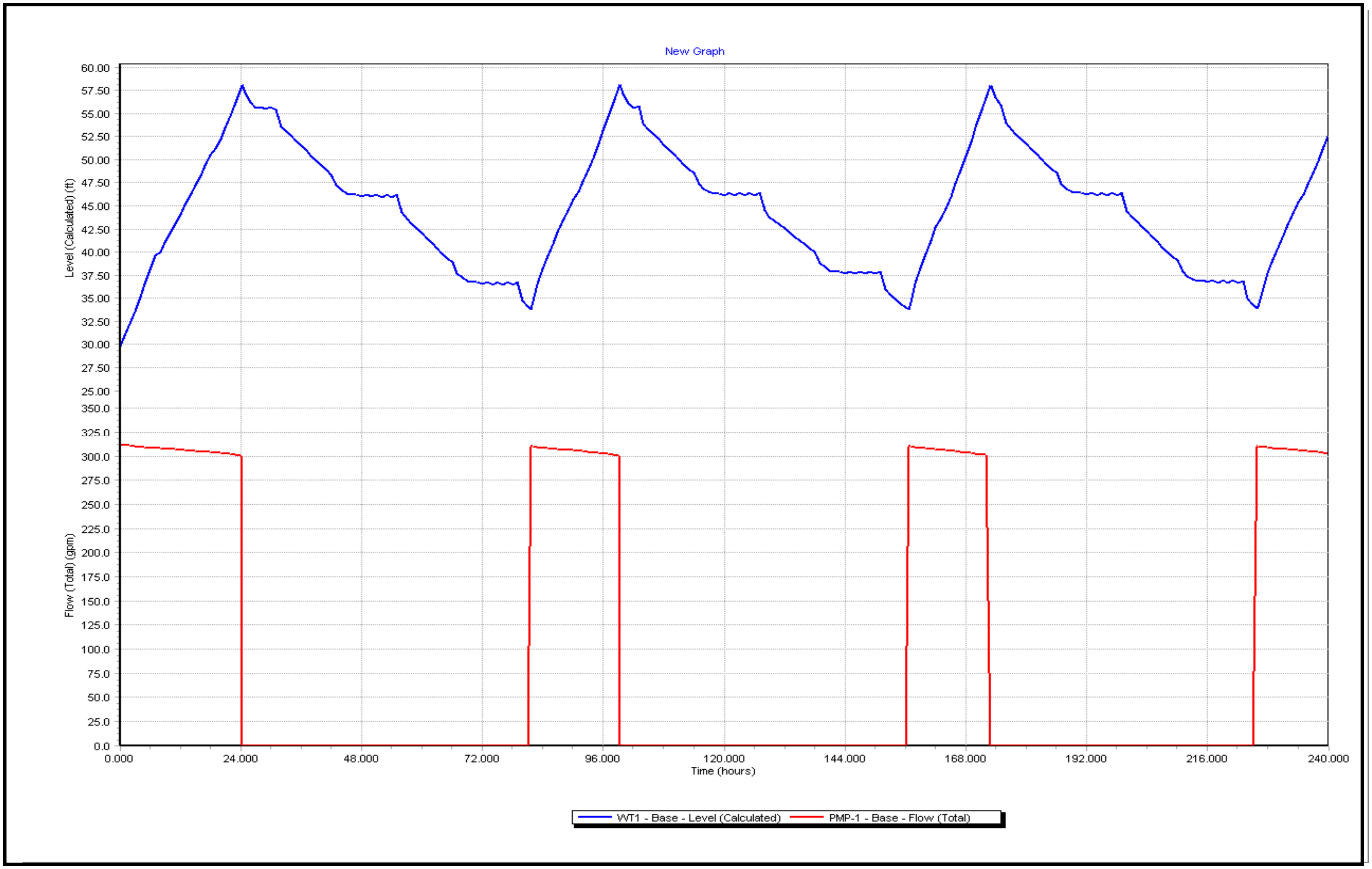

Figure 30: Pump cycle for recommended pump

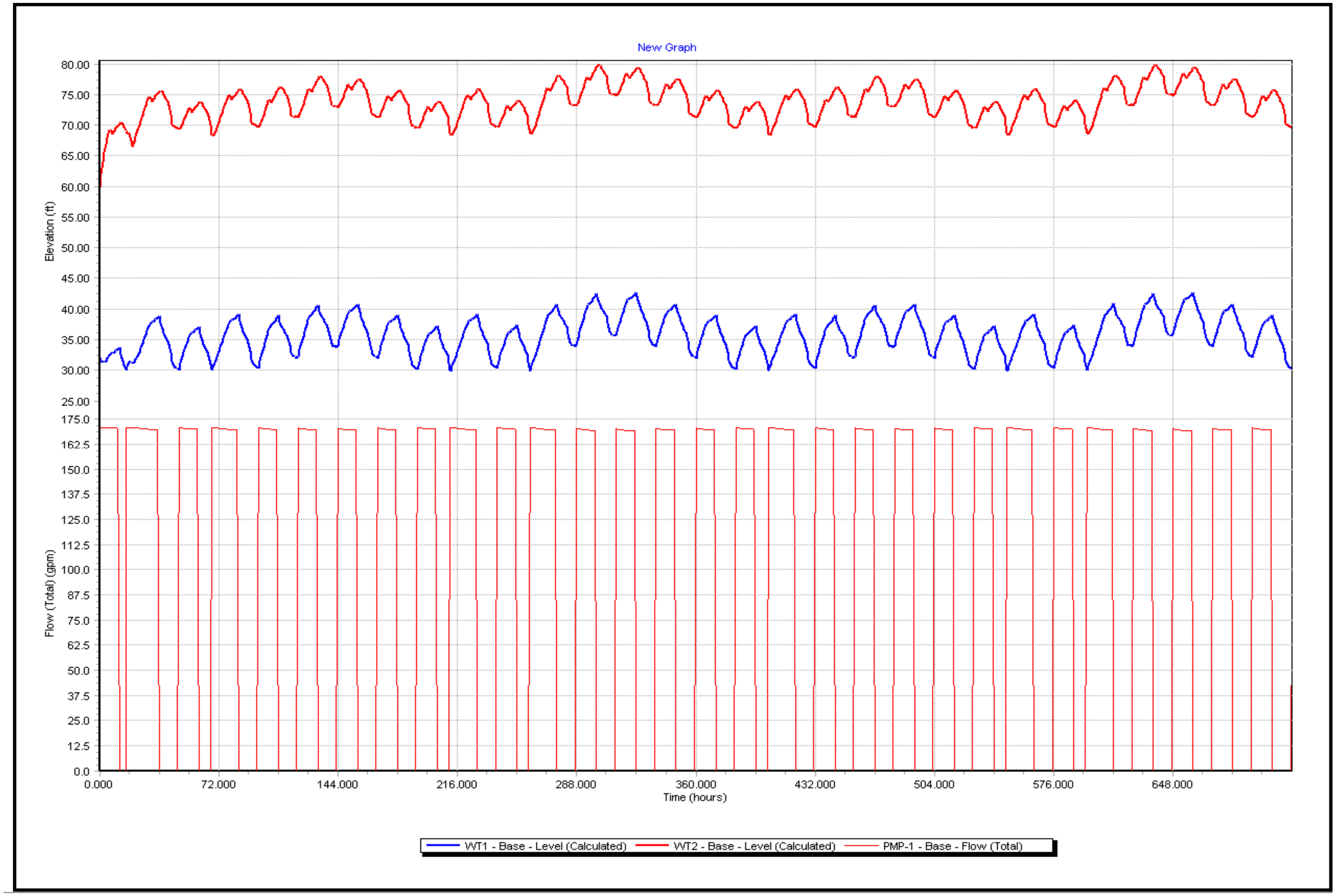

Figure 31: Graph of the pump cycles and tank levels when existing pump is run for 12 hours daily

The second important finding was the fire hydrant in the western most pipeline not being able to supply 250 gpm at a minimum pressure of 20 psi. Even with a larger pump, it cannot provide fire flows at the required pressure. This pipe line is an old cast iron pipeline and has about 15 years of its utility life left. Cleaning of this pipeline could be implemented to marginally improve the flow through the pipeline. However, it is most advisable to install a pipe with larger diameter. Replacing the line with a 4” line helps increase the pressure in this region and the hydrant can then provide a pressure of about 21 psi and a pressure of about 25 psi is obtained if the line is replaced with a 6” line. Water quality is not a problem except that the north well has iron in it. Iron is regulated as a secondary standard by EPA, since iron affects the taste and appearance, not the safety of the water (EPA, 2009). The six treatment options available are summarized in the Table 3.

Table 3: Treatment options for removal of iron from drinking water (Colter, 2006).

Sr no	Treatment	Comments	Price
1.	Faucet Attachments for individual houses	This is a cheap solution and can filter out iron if present in small amounts. Else, it will clog the faucet filters.	\$20 to \$100
2.	Aeration and filtration	This treatment is best suited if the iron concentration is greater than 25 mg/L. It is ineffective for organic iron. The start up of this process costs about USD 1000.	\$200 to \$5000
3.	Ion exchange/ water softeners	This process is efficient in removing iron present at low concentrations of less than or equal to 5 mg/L. This process is also ineffective in removal of organic iron.	\$200 and up

Sr no	Treatment	Comments	Price
4.	Phosphate treatment	This method can be used for removal of iron having concentration up to 3 mg/L. However, it increases the nutrient levels.	\$300 and up
5.	Chemical oxidation and filtration	This process can treat iron concentrations up to 10 mg/L. However it involves use of chemicals like chlorine, potassium permanganate and hydrogen peroxide. These chemicals should be handled carefully.	\$500 and up
6.	Oxidizing filters	These filters use manganese green sand as filter media and effectively remove 99% of iron which is present in concentration up to 15 mg/L.	\$500 and up

The data of the exact concentration of the iron the north well is not known and hence a suitable process can be selected after the reports from the testing laboratories are available.

Additional recommendations for the system:

- Pressure readings need to be taken at more than one location to verify the result of the simulation.
- Pump efficiency curves of the existing pump need to be obtained to verify the results of the simulation.
- If convenient and affordable, the City should carry out a test to find out the specific capacity of their well and also perform a drawdown test to have a better knowledge of their supply system.
- It is most recommended that the houses located at the far southern end of the town have additional pressure tanks installed near their homes and use booster pumps to solve the problems of poor pressures at their locations.
- Since the simulation indicated low chlorine residuals at these homes, water at these homes should be tested periodically for chlorine residuals and coliforms.

Recommendations for further study:

In a very recent communication with Mr. Green, the city has planned to shut down the north well due the water rights dispute and that south well would soon be ready to operate (Green, 2009). The research study was performed under the conditions, where the south well was out of service and the north well was fully functioning. Further information can be collected on this change in well operation and results can be presented. Though the city does not expect any commercial or industrial growth a future analysis of the system can be done or the system can be modeled as the state regulation of 1 gpm per service connection at a minimum pressure of 25 psi. Energy optimization studies can be performed to control the pumps so that electricity costs can be reduced.

Conclusions

To finally conclude this study, the city of Oilton has a fairly good distribution system in place. However, the capacity of the storage tanks is far greater than the present water demands, which would necessitate the use of larger capacity pump in order to fill the tanks quickly. Overall improvements in the transmissions lines and other appurtenances can be done on as-needed basis considering the growth of the town in future years.

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APPENDICES

APPENDIX A.

Calculations for Assigning Water Demands to each node

Population of the town: 1200 people

Number of houses in City of Oilton: 418

Average Number of People in each house: 2.87

Total water consumption: 118,000 gallons per day

Total water consumption in gpm: 81.94gpm

NODE	No. of houses around the node	population served by node	DEMAND in gpm	demand in l/s
N1	9	25.83732057	1.764354	0.111313
N2	5	14.35406699	0.980197	0.061841
N3	3	8.612440191	0.588118	0.037104
N4	0	0	0	0
N5	5	14.35406699	0.980197	0.061841
N6	5	14.35406699	0.980197	0.061841
N7	2	5.741626794	0.392079	0.024736
N8	3	8.612440191	0.588118	0.037104
N9	5	14.35406699	0.980197	0.061841
N10	7	20.09569378	1.372275	0.086577
N11	5	14.35406699	0.980197	0.061841
N12	4	11.48325359	0.784157	0.049472
N13	2	5.741626794	0.392079	0.024736
N14	2	5.741626794	0.392079	0.024736
N15	4	11.48325359	0.784157	0.049472
N16	3	8.612440191	0.588118	0.037104
N17	1	2.870813397	0.196039	0.012368
N18	1	2.870813397	0.196039	0.012368
N19	1	2.870813397	0.196039	0.012368
N20	2	5.741626794	0.392079	0.024736
N21	2	5.741626794	0.392079	0.024736
N22	3	8.612440191	0.588118	0.037104
N23	3	8.612440191	0.588118	0.037104

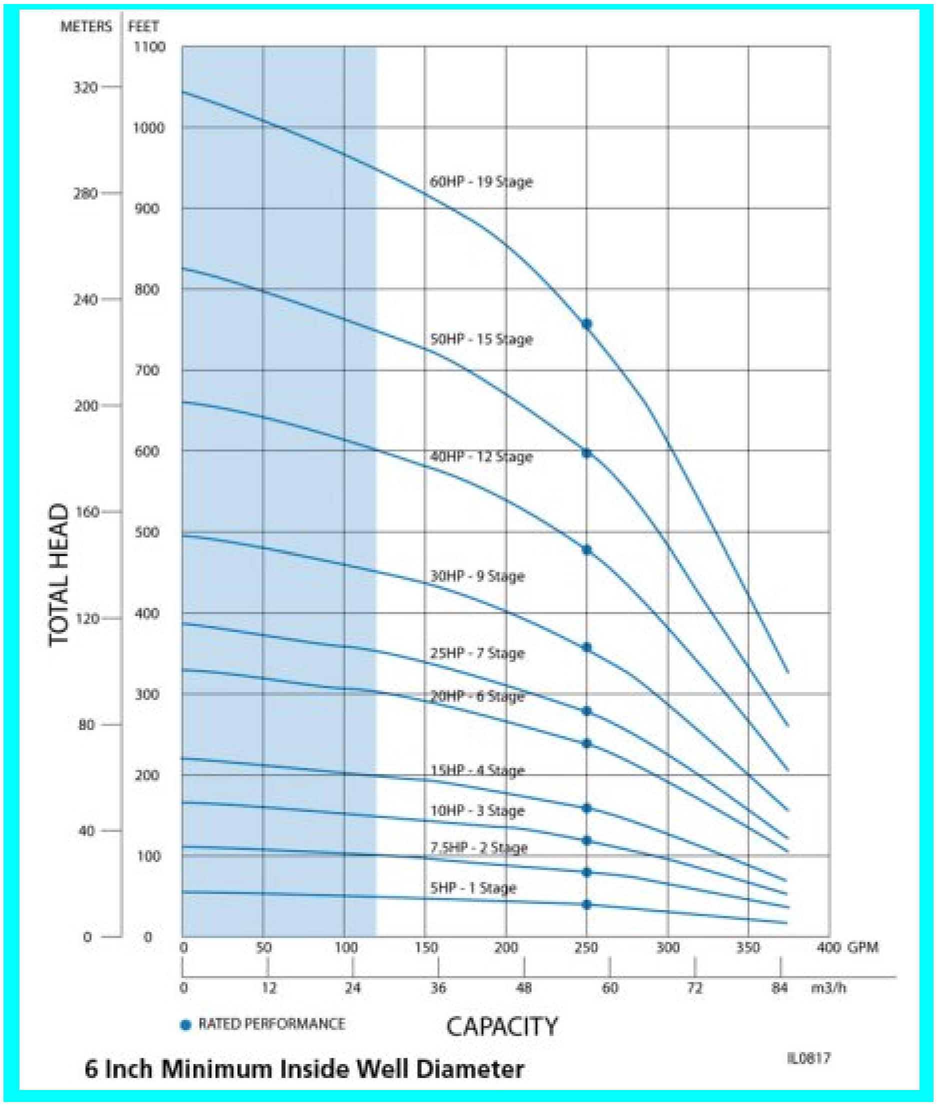
N24	4	11.48325359	0.784157	0.049472
N25	2	5.741626794	0.392079	0.024736
N26	2	5.741626794	0.392079	0.024736
N27	1	2.870813397	0.196039	0.012368
N28	6	17.22488038	1.176236	0.074209
N29	7	20.09569378	1.372275	0.086577
N30	2	5.741626794	0.392079	0.024736
N31	3	8.612440191	0.588118	0.037104
N32	3	8.612440191	0.588118	0.037104
N33	2	5.741626794	0.392079	0.024736
N34	4	11.48325359	0.784157	0.049472
N35	1	2.870813397	0.196039	0.012368
N36	0	0	0	0
N37	7	20.09569378	1.372275	0.086577
N38	4	11.48325359	0.784157	0.049472
N39	0	0	0	0
N40	1	2.870813397	0.196039	0.012368
N41	1	2.870813397	0.196039	0.012368
N42	3	8.612440191	0.588118	0.037104
N43	3	8.612440191	0.588118	0.037104
N44	2	5.741626794	0.392079	0.024736
N45	6	17.22488038	1.176236	0.074209
N46	3	8.612440191	0.588118	0.037104
N47	3	8.612440191	0.588118	0.037104
N48	6	17.22488038	1.176236	0.074209
N49	4	11.48325359	0.784157	0.049472
N50	4	11.48325359	0.784157	0.049472
N51	5	14.35406699	0.980197	0.061841
N52	5	14.35406699	0.980197	0.061841
N53	2	5.741626794	0.392079	0.024736
N54	2	5.741626794	0.392079	0.024736
N55	3	8.612440191	0.588118	0.037104
N56	6	17.22488038	1.176236	0.074209
N57	8	22.96650718	1.568315	0.098945
N58	7	20.09569378	1.372275	0.086577
N59	3	8.612440191	0.588118	0.037104
N60	6	17.22488038	1.176236	0.074209
N61	2	5.741626794	0.392079	0.024736
N62	6	17.22488038	1.176236	0.074209
N63	0	0	0	0
N64	0	0	0	0

N65	0	0	0	0
N66	0	0	0	0
N67	2	5.741626794	0.392079	0.024736
N68	9	25.83732057	1.764354	0.111313
N69	3	8.612440191	0.588118	0.037104
N70	4	11.48325359	0.784157	0.049472
N71	5	14.35406699	0.980197	0.061841
N72	2	5.741626794	0.392079	0.024736
N73	4	11.48325359	0.784157	0.049472
N74	7	20.09569378	1.372275	0.086577
N75	6	17.22488038	1.176236	0.074209
N76	5	14.35406699	0.980197	0.061841
N77	8	22.96650718	1.568315	0.098945
N78	3	8.612440191	0.588118	0.037104
N79	6	17.22488038	1.176236	0.074209
N80	4	11.48325359	0.784157	0.049472
N81	2	5.741626794	0.392079	0.024736
N82	2	5.741626794	0.392079	0.024736
N83	2	5.741626794	0.392079	0.024736
N84	0	0	0	0
N85	2	5.741626794	0.392079	0.024736
N86	2	5.741626794	0.392079	0.024736
N87	2	5.741626794	0.392079	0.024736
N88	3	8.612440191	0.588118	0.037104
N89	3	8.612440191	0.588118	0.037104
N90	3	8.612440191	0.588118	0.037104
N91	4	11.48325359	0.784157	0.049472
N92	3	8.612440191	0.588118	0.037104
N93	2	5.741626794	0.392079	0.024736
N94	6	17.22488038	1.176236	0.074209
N95	4	11.48325359	0.784157	0.049472
N96	6	17.22488038	1.176236	0.074209
N97	9	25.83732057	1.764354	0.111313
N98	6	17.22488038	1.176236	0.074209
N99	4	11.48325359	0.784157	0.049472
N100	4	11.48325359	0.784157	0.049472
N101	5	14.35406699	0.980197	0.061841
N102	4	11.48325359	0.784157	0.049472
N103	3	8.612440191	0.588118	0.037104
N104	2	5.741626794	0.392079	0.024736
N105	3	8.612440191	0.588118	0.037104

N106	0	0	0	0
N107	4	11.48325359	0.784157	0.049472
N108	3	8.612440191	0.588118	0.037104
N109	3	8.612440191	0.588118	0.037104
N110	0	0	0	0
N111	0	0	0	0
N112	5	14.35406699	0.980197	0.061841
N113	0	0	0	0
N114	**0**	**0**	**0**	**0**
N115	0	0	0	0
N116	0	0	0	0
N117	21	60.28708134	4.116826	0.259731
N118	7	20.09569378	1.372275	0.086577
N119	3	8.612440191	0.588118	0.037104
N120	1	2.870813397	0.196039	0.012368
N121	1	2.870813397	0.196039	0.012368
N122	0	0	0	0
N123	0	0	0	0
N124	0	0	0	0
TOTAL	**418**	**1200**	**81.94444**	**5.169875**

APPENDIX B

Pump Curves for the recommended pump:

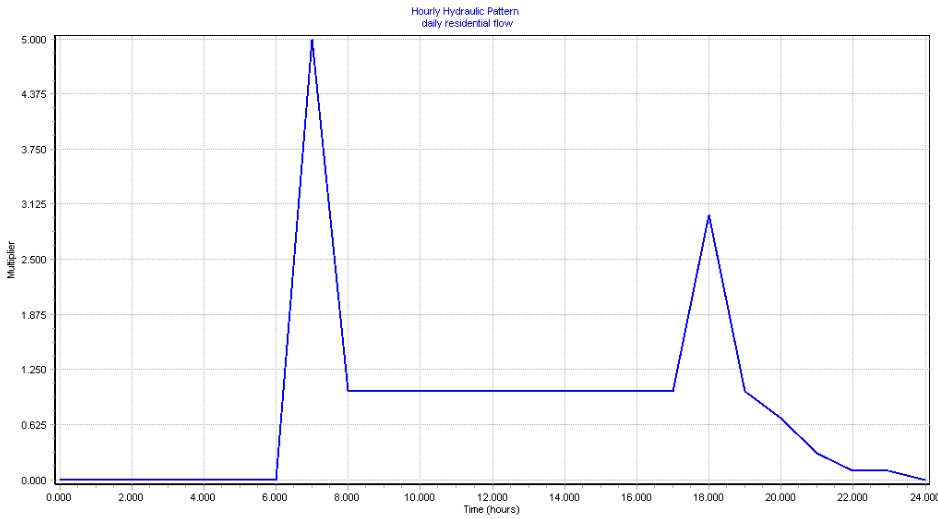


(Source: Flint and Walling 50 HP Submersible Pump , Dean Bennett Supply, 2009)

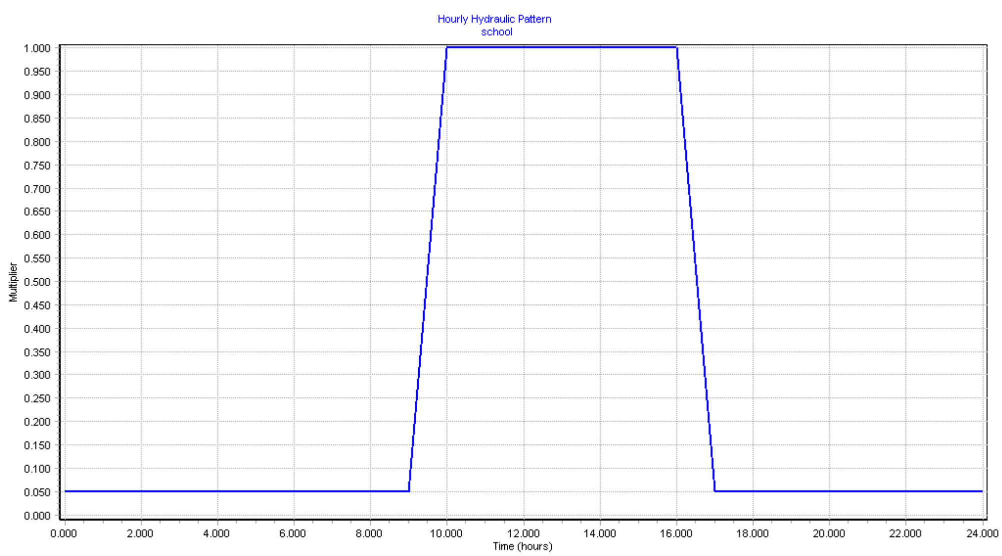
APPENDIX C

Patterns for all the nodes and School node pattern:

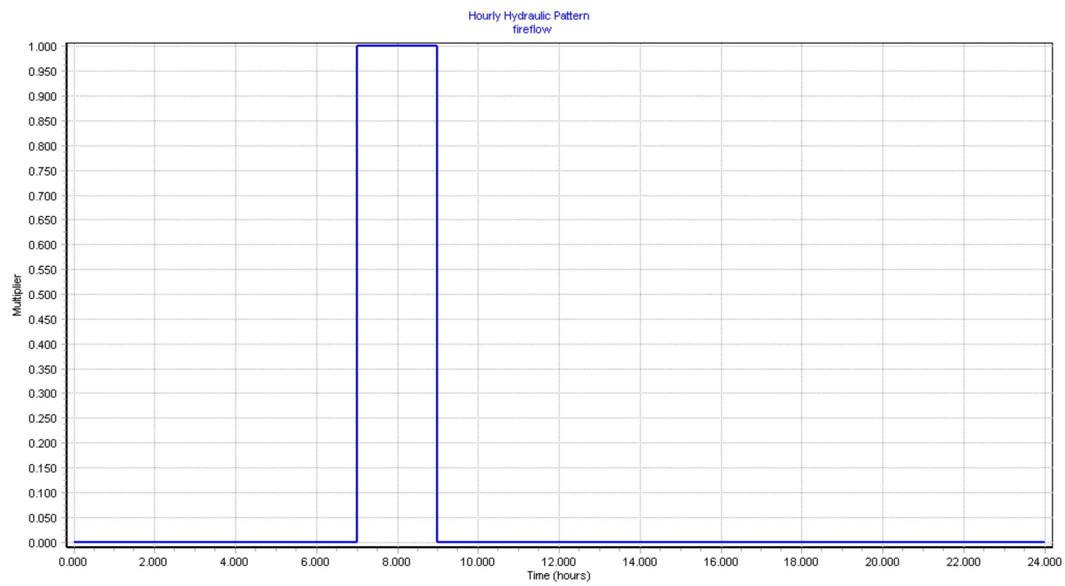
Diurnal Pattern for existing conditions:



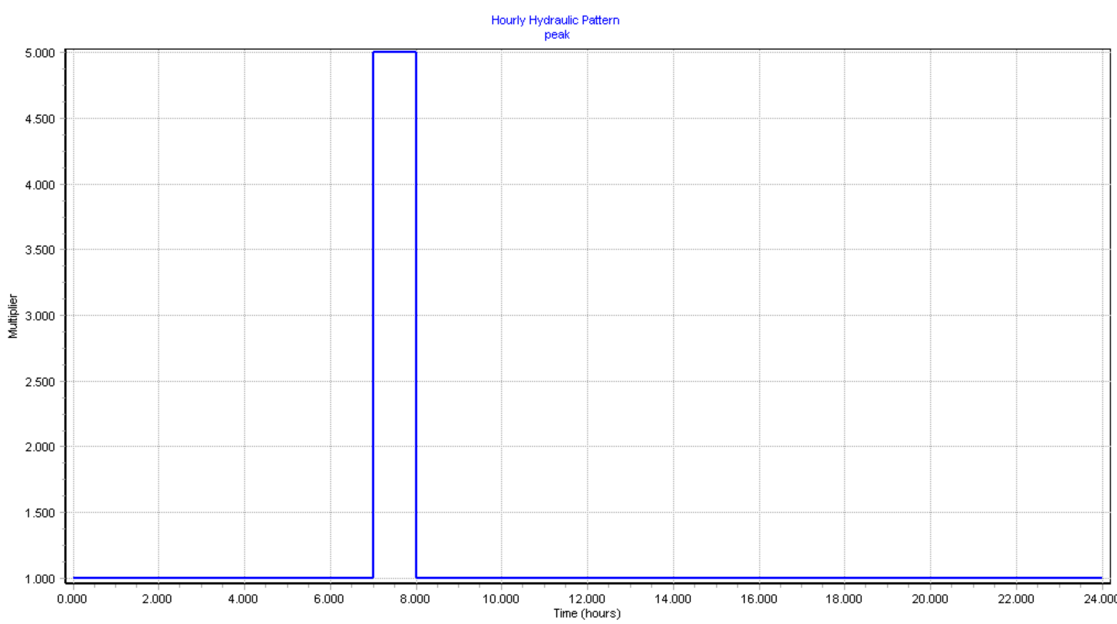
Pattern for school node:



Pattern for fire flow conditions:



Pattern for peak flow demand:



APPENDIX D

Control statements:

Controls for existing condition:

Controls Summary

Licensed for Academic Use Only

Logical Control: LC675

IF {"WT1" Level >= 58.00 ft}
THEN {"PMP-1" Pump Status = Off }
PRIORITY 5

Logical Control: LC678

IF {"WT1" Level <= 30.00 ft}
THEN {"PMP-1" Pump Status = On }

Logical Control: LC689

IF {"WT2" Level <= 73.00 ft}
THEN {"PMP-1" Pump Status = On }
PRIORITY 4

Control conditions when the pump runs for 12 hours:

Controls Summary

Licensed for Academic Use Only

Logical Control: LC688

IF {"WT1" Level >= 58.00 ft}
THEN {"PMP-1" Pump Status = On }
PRIORITY 5

Logical Control: LC691

IF {"WT1" Level <= 30.00 ft}
THEN {"PMP-1" Pump Status = On }
PRIORITY 5

Logical Control: LC694

IF {"Clock Time" = 12:00 AM }
THEN {"PMP-1" Pump Status = On }
PRIORITY 4

Logical Control: LC696

IF {"Clock Time" = 12:00 PM }
THEN {"PMP-1" Pump Status = On }
PRIORITY 4

Logical Control: LC698

IF {"WT2" Level <= 73.00 ft}
THEN {"PMP-1" Pump Status = On }

APPENDIX E

Calculation summary of the Diurnal Analysis for existing pump:

Calculation Summary (1: Base)
Licensed for Academic Use Only

Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
All Time Steps(244)	True	810	0.0009357	78.7
0.000	True	7	0.0001937	0.0
0.100	True	4	0.0001101	171.0
1.000	True	3	0.0000540	171.0
2.000	True	3	0.0000561	171.0
3.000	True	3	0.0000632	171.0
4.000	True	3	0.0000660	170.9
5.000	True	3	0.0000609	170.8
6.000	True	4	0.0007603	170.8
7.000	True	3	0.0000223	170.8
8.000	True	4	0.0006878	170.8
9.000	True	3	0.0001021	170.7
10.000	True	3	0.0000198	170.7
11.000	True	2	0.0004839	170.6
12.000	True	2	0.0001944	170.6
13.000	True	2	0.0000714	170.5
14.000	True	2	0.0000282	170.5
15.000	True	2	0.0000213	170.4
16.000	True	2	0.0000220	170.3
17.000	True	4	0.0000678	170.3
18.000	True	3	0.0000686	170.3
19.000	True	5	0.0004617	170.2
20.000	True	3	0.0000225	170.1
21.000	True	3	0.0000236	170.1
22.000	True	2	0.0005411	170.0
23.000	True	2	0.0002338	169.9
24.000	True	2	0.0001467	169.7
25.000	True	2	0.0000535	169.6
26.000	True	2	0.0000392	169.5
27.000	True	2	0.0000392	169.4
28.000	True	2	0.0000419	169.3
29.000	True	2	0.0000413	169.2
30.000	True	4	0.0003041	169.1
31.000	True	3	0.0000413	169.1
32.000	True	4	0.0005701	169.1
33.000	True	3	0.0001442	169.0
34.000	True	2	0.0009179	169.0
35.000	True	2	0.0004002	168.9
36.000	True	2	0.0001550	168.9
37.000	True	2	0.0000554	168.8
38.000	True	2	0.0000272	168.8
39.000	True	2	0.0000236	168.7
40.000	True	2	0.0000214	168.6
41.000	True	4	0.0000811	168.6
42.000	True	3	0.0000828	168.6
43.000	True	5	0.0003658	168.5

Calculation Summary (1: Base)

Licensed for Academic Use Only

Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
44.000	True	3	0.0000230	168.5
45.000	True	3	0.0000277	168.4
46.000	True	2	0.0005562	168.3
47.000	True	2	0.0002368	168.2
48.000	True	2	0.0001495	168.0
49.000	True	2	0.0000545	167.9
50.000	True	2	0.0000406	167.8
51.000	True	2	0.0000424	167.7
52.000	True	2	0.0000380	167.6
53.000	True	2	0.0000394	167.5
54.000	True	4	0.0002929	167.4
55.000	True	3	0.0000446	167.4
56.000	True	4	0.0005576	167.4
57.000	True	3	0.0001561	167.3
57.800	True	3	0.0008877	0.0
58.000	True	3	0.0000495	0.0
59.000	True	5	0.0001054	0.0
60.000	True	4	0.0001333	0.0
61.000	True	3	0.0001224	0.0
62.000	True	2	0.0001205	0.0
63.000	True	2	0.0001041	0.0
64.000	True	1	0.0001616	0.0
65.000	True	3	0.0000718	0.0
66.000	True	2	0.0001183	0.0
67.000	True	4	0.0001454	0.0
68.000	True	4	0.0002275	0.0
69.000	True	4	0.0006387	0.0
70.000	True	5	0.0007948	0.0
71.000	True	5	0.0004958	0.0
72.000	True	5	0.0003880	0.0
73.000	True	6	0.0003233	0.0
74.000	True	7	0.0003051	0.0
75.000	True	7	0.0004592	0.0
76.000	True	7	0.0005469	0.0
77.000	True	7	0.0005600	0.0
78.000	True	4	0.0001168	0.0
79.000	True	3	0.0001348	0.0
80.000	True	4	0.0000821	0.0
81.000	True	3	0.0006563	0.0
82.000	True	3	0.0001040	0.0
83.000	True	2	0.0001306	0.0
84.000	True	2	0.0001262	0.0
85.000	True	1	0.0002458	0.0
86.000	True	1	0.0001061	0.0
87.000	True	1	0.0001103	0.0
88.000	True	1	0.0001119	0.0

Calculation Summary (1: Base)

Licensed for Academic Use Only

Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
89.000	True	3	0.0000685	0.0
90.000	True	2	0.0001091	0.0
91.000	True	4	0.0001321	0.0
92.000	True	4	0.0002086	0.0
93.000	True	4	0.0006429	0.0
94.000	True	5	0.0008758	0.0
95.000	True	5	0.0005238	0.0
96.000	True	5	0.0006392	0.0
97.000	True	6	0.0005461	0.0
98.000	True	7	0.0005214	0.0
99.000	True	7	0.0005750	0.0
100.000	True	7	0.0006154	0.0
101.000	True	7	0.0006160	0.0
102.000	True	4	0.0001062	0.0
103.000	True	3	0.0001403	0.0
104.000	True	4	0.0000936	0.0
105.000	True	3	0.0005970	0.0
106.000	True	3	0.0001198	0.0
107.000	True	2	0.0001129	0.0
108.000	True	2	0.0001148	0.0
109.000	True	1	0.0002532	0.0
110.000	True	1	0.0001083	0.0
111.000	True	1	0.0001085	0.0
112.000	True	1	0.0001204	0.0
113.000	True	3	0.0000754	0.0
114.000	True	2	0.0001160	0.0
115.000	True	4	0.0001336	0.0
116.000	True	4	0.0001966	0.0
117.000	True	4	0.0006515	0.0
118.000	True	5	0.0008763	0.0
119.000	True	5	0.0005040	0.0
120.000	True	5	0.0003900	0.0
121.000	True	6	0.0003021	0.0
121.900	True	5	0.0000454	170.4
122.000	True	2	0.0002100	170.4
123.000	True	3	0.0001391	170.3
124.000	True	3	0.0000374	170.1
125.000	True	2	0.0002953	170.0
126.000	True	1	0.0002743	170.0
127.000	True	3	0.0000471	169.9
128.000	True	1	0.0005771	169.9
129.000	True	3	0.0001505	169.9
130.000	True	2	0.0008636	169.8
131.000	True	2	0.0003758	169.8
132.000	True	2	0.0001438	169.7
133.000	True	2	0.0000532	169.7

Calculation Summary (1: Base)
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Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
134.000	True	2	0.0000255	169.6
135.000	True	2	0.0000196	169.5
136.000	True	2	0.0000187	169.5
137.000	True	4	0.0000730	169.5
138.000	True	3	0.0000764	169.4
139.000	True	5	0.0004039	169.4
140.000	True	3	0.0000232	169.3
141.000	True	3	0.0000256	169.2
142.000	True	2	0.0005504	169.1
143.000	True	2	0.0002352	169.0
144.000	True	2	0.0001471	168.9
145.000	True	2	0.0000530	168.8
146.000	True	2	0.0000416	168.7
147.000	True	2	0.0000353	168.5
148.000	True	2	0.0000368	168.4
149.000	True	2	0.0000367	168.3
150.000	True	4	0.0002975	168.3
151.000	True	3	0.0000403	168.2
152.000	True	4	0.0005615	168.2
153.000	True	3	0.0001508	168.2
154.000	True	2	0.0009357	168.1
155.000	True	2	0.0004079	168.1
156.000	True	2	0.0001568	168.0
157.000	True	2	0.0000560	168.0
158.000	True	2	0.0000245	167.9
159.000	True	2	0.0000213	167.9
160.000	True	2	0.0000210	167.8
161.000	True	4	0.0000907	167.8
162.000	True	3	0.0000931	167.7
163.000	True	5	0.0003415	167.7
164.000	True	3	0.0000235	167.6
165.000	True	3	0.0000244	167.5
166.000	True	2	0.0005635	167.4
167.000	True	2	0.0002402	167.3
167.400	True	4	0.0000679	0.0
168.000	True	4	0.0001562	0.0
169.000	True	7	0.0003482	0.0
170.000	True	6	0.0002799	0.0
171.000	True	7	0.0003017	0.0
172.000	True	7	0.0004832	0.0
173.000	True	7	0.0005849	0.0
174.000	True	4	0.0001085	0.0
175.000	True	3	0.0001399	0.0
176.000	True	4	0.0000779	0.0
177.000	True	3	0.0006054	0.0
178.000	True	3	0.0001248	0.0

Calculation Summary (1: Base)
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Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
179.000	True	2	0.0001153	0.0
180.000	True	2	0.0001073	0.0
181.000	True	1	0.0002503	0.0
182.000	True	1	0.0001143	0.0
183.000	True	1	0.0001159	0.0
184.000	True	1	0.0001114	0.0
185.000	True	3	0.0000692	0.0
186.000	True	2	0.0001206	0.0
187.000	True	4	0.0001289	0.0
188.000	True	4	0.0002230	0.0
189.000	True	4	0.0006137	0.0
190.000	True	5	0.0008782	0.0
191.000	True	5	0.0004601	0.0
192.000	True	5	0.0003357	0.0
193.000	True	6	0.0002830	0.0
194.000	True	7	0.0003164	0.0
195.000	True	7	0.0004656	0.0
196.000	True	7	0.0005605	0.0
197.000	True	7	0.0005536	0.0
198.000	True	4	0.0001190	0.0
199.000	True	3	0.0001341	0.0
200.000	True	4	0.0000841	0.0
201.000	True	3	0.0006588	0.0
202.000	True	3	0.0001114	0.0
203.000	True	2	0.0001293	0.0
204.000	True	2	0.0001189	0.0
205.000	True	1	0.0002491	0.0
206.000	True	1	0.0001133	0.0
207.000	True	1	0.0001132	0.0
208.000	True	1	0.0001147	0.0
209.000	True	3	0.0000669	0.0
210.000	True	2	0.0001056	0.0
211.000	True	4	0.0001228	0.0
212.000	True	4	0.0002415	0.0
213.000	True	4	0.0005654	0.0
214.000	True	5	0.0008705	0.0
215.000	True	5	0.0004866	0.0
216.000	True	5	0.0006151	0.0
217.000	True	6	0.0004996	0.0
218.000	True	7	0.0005121	0.0
219.000	True	7	0.0005720	0.0
220.000	True	7	0.0006168	0.0
221.000	True	7	0.0006247	0.0
222.000	True	4	0.0001052	0.0
223.000	True	3	0.0001399	0.0
224.000	True	4	0.0000863	0.0

Calculation Summary (1: Base)
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Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
225.000	True	3	0.0006009	0.0
226.000	True	3	0.0001258	0.0
227.000	True	2	0.0001202	0.0
228.000	True	2	0.0001154	0.0
229.000	True	1	0.0002520	0.0
230.000	True	1	0.0001436	0.0
231.000	True	1	0.0001184	0.0
232.000	True	4	0.0001441	170.5
233.000	True	3	0.0008562	170.4
234.000	True	3	0.0001078	170.3
235.000	True	5	0.0000297	170.3
236.000	True	3	0.0000329	170.2
237.000	True	3	0.0000223	170.1
238.000	True	2	0.0004796	170.0
239.000	True	2	0.0001954	169.9
240.000	True	2	0.0001314	169.8

Flow Demanded (gpm)	Flow Stored (gpm)
67.5	11.2
0.0	0.0
0.0	171.0
0.0	171.0
0.0	171.0
0.0	171.0
0.0	170.9
0.0	170.8
197.8	-27.0
261.3	-90.5
103.1	67.6
79.1	91.6
79.1	91.6
79.1	91.5
79.1	91.5
79.1	91.4
79.1	91.4
79.1	91.3
79.1	91.2
158.2	12.1
158.2	12.1
67.2	103.0
39.6	130.6
15.8	154.2
7.9	162.1
4.0	165.9
0.0	169.7

Calculation Summary (1: Base)
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Flow Demanded (gpm)	Flow Stored (gpm)
0.0	169.6
0.0	169.5
0.0	169.4
0.0	169.3
0.0	169.1
197.8	-28.6
261.3	-92.2
103.1	65.9
79.1	89.9
79.1	89.9
79.1	89.8
79.1	89.8
79.1	89.7
79.1	89.7
79.1	89.6
79.1	89.5
158.2	10.4
158.2	10.4
67.2	101.3
39.6	128.9
15.8	152.5
7.9	160.4
4.0	164.2
0.0	168.0
0.0	167.9
0.0	167.8
0.0	167.7
0.0	167.6
0.0	167.4
197.8	-30.3
261.3	-93.9
103.1	64.3
79.1	88.2
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
158.2	-158.2
158.2	-158.2
67.2	-67.2
39.6	-39.6
15.8	-15.8

Calculation Summary (1: Base)
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Flow Demanded (gpm)	Flow Stored (gpm)
7.9	-7.9
4.0	-4.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
197.8	-197.8
261.3	-261.3
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
158.2	-158.2
158.2	-158.2
67.2	-67.2
39.6	-39.6
15.8	15.8
7.9	7.9
4.0	-4.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
197.8	-197.8
261.3	-261.3
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
158.2	-158.2
158.2	-158.2
67.2	-67.2

Calculation Summary (1: Base)
Licensed for Academic Use Only

Flow Demanded (gpm)	Flow Stored (gpm)
39.6	-39.6
15.8	-15.8
7.9	-7.9
4.0	-4.0
0.0	0.0
0.0	0.0
0.0	170.4
0.0	170.4
0.0	170.3
0.0	170.1
0.0	170.0
197.8	-27.8
261.3	-91.4
103.1	66.8
79.1	90.8
79.1	90.7
79.1	90.7
79.1	90.6
79.1	90.6
79.1	90.5
79.1	90.4
79.1	90.4
158.2	11.2
158.2	11.2
67.2	102.1
39.6	129.7
15.8	153.4
7.9	161.2
4.0	165.0
0.0	168.9
0.0	168.8
0.0	168.6
0.0	168.5
0.0	168.4
0.0	168.3
197.8	-29.5
261.3	-93.1
103.1	65.1
79.1	89.1
79.1	89.0
79.1	89.0
79.1	88.9
79.1	88.9
79.1	88.8
79.1	88.7
79.1	88.7

Calculation Summary (1: Base)
Licensed for Academic Use Only

Flow Demanded (gpm)	Flow Stored (gpm)
158.2	9.5
158.2	9.5
67.2	100.4
39.6	128.0
15.8	151.7
7.9	159.5
4.0	163.4
4.0	-4.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
197.8	197.8
261.3	-261.3
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
158.2	-158.2
158.2	-158.2
67.2	-67.2
39.6	-39.6
15.8	-15.8
7.9	-7.9
4.0	-4.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
197.8	-197.8
261.3	-261.3
103.1	103.1
79.1	-79.1
79.1	79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1

Calculation Summary (1: Base)
Licensed for Academic Use Only

Flow Demanded (gpm)	Flow Stored (gpm)
79.1	-79.1
79.1	-79.1
79.1	-79.1
158.2	-158.2
158.2	-158.2
67.2	-67.2
39.6	-39.6
15.8	-15.8
7.9	-7.9
4.0	-4.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
197.8	-197.8
261.3	-261.3
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	91.4
158.2	12.2
158.2	12.1
67.2	103.0
39.6	130.7
15.8	154.3
7.9	162.1
4.0	165.9
0.0	169.8

APPENDIX F

Calculation summary for diurnal analysis using recommended pump:

Calculation Summary (1: Base)
Licensed for Academic Use Only

Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
All Time Steps(247)	True	871	0.0009800	86.5
0.000	True	7	0.0001937	0.0
0.100	True	4	0.0000481	312.0
1.000	True	2	0.0003490	311.6
2.000	True	2	0.0003200	311.1
3.000	True	2	0.0002212	310.5
4.000	True	2	0.0001512	310.0
5.000	True	2	0.0001010	309.4
6.000	True	2	0.0000687	308.8
7.000	True	4	0.0008634	309.0
8.000	True	5	0.0003055	308.3
9.000	True	3	0.0000176	308.0
10.000	True	2	0.0001102	307.5
11.000	True	2	0.0000705	307.1
12.000	True	2	0.0000442	306.7
13.000	True	2	0.0000280	306.2
14.000	True	2	0.0000176	305.8
15.000	True	2	0.0000145	305.3
16.000	True	2	0.0000125	304.9
17.000	True	2	0.0000122	304.4
18.000	True	4	0.0003043	304.3
19.000	True	5	0.0001256	303.7
20.000	True	2	0.0008448	303.2
21.000	True	3	0.0000135	302.6
22.000	True	2	0.0004050	302.1
23.000	True	2	0.0000597	301.5
23.800	True	4	0.0000349	0.0
24.000	True	3	0.0000616	0.0
25.000	True	4	0.0001016	0.0
26.000	True	4	0.0006910	0.0
27.000	True	7	0.0003161	0.0
28.000	True	7	0.0004205	0.0
29.000	True	7	0.0005749	0.0
30.000	True	7	0.0005898	0.0
31.000	True	4	0.0001110	0.0
32.000	True	4	0.0000899	0.0
33.000	True	3	0.0005144	0.0
34.000	True	2	0.0006021	0.0
35.000	True	2	0.0001241	0.0
36.000	True	1	0.0008689	0.0
37.000	True	1	0.0001552	0.0
38.000	True	1	0.0001302	0.0
39.000	True	1	0.0001319	0.0
40.000	True	1	0.0001219	0.0
41.000	True	1	0.0001241	0.0
42.000	True	3	0.0002287	0.0

Calculation Summary (1: Base)
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Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
43.000	True	4	0.0001278	0.0
44.000	True	3	0.0003635	0.0
45.000	True	4	0.0003375	0.0
46.000	True	6	0.0008186	0.0
47.000	True	5	0.0002219	0.0
48.000	True	5	0.0007241	0.0
49.000	True	6	0.0005981	0.0
50.000	True	7	0.0005455	0.0
51.000	True	7	0.0006033	0.0
52.000	True	7	0.0006458	0.0
53.000	True	7	0.0006803	0.0
54.000	True	7	0.0006385	0.0
55.000	True	4	0.0001058	0.0
56.000	True	4	0.0000910	0.0
57.000	True	3	0.0006871	0.0
58.000	True	3	0.0001308	0.0
59.000	True	2	0.0001401	0.0
60.000	True	2	0.0001142	0.0
61.000	True	1	0.0003378	0.0
62.000	True	1	0.0001140	0.0
63.000	True	1	0.0001118	0.0
64.000	True	1	0.0001177	0.0
65.000	True	1	0.0001192	0.0
66.000	True	3	0.0002273	0.0
67.000	True	4	0.0001215	0.0
68.000	True	3	0.0003534	0.0
69.000	True	4	0.0003724	0.0
70.000	True	6	0.0008233	0.0
71.000	True	5	0.0002353	0.0
72.000	True	5	0.0007340	0.0
73.000	True	6	0.0005765	0.0
74.000	True	7	0.0005089	0.0
75.000	True	7	0.0006326	0.0
76.000	True	7	0.0006229	0.0
77.000	True	7	0.0006674	0.0
78.000	True	7	0.0006506	0.0
79.000	True	4	0.0001082	0.0
80.000	True	4	0.0001044	0.0
81.000	True	3	0.0006818	0.0
82.000	True	3	0.0001266	0.0
83.000	True	2	0.0001325	0.0
84.000	True	2	0.0001069	0.0
84.600	True	4	0.0004677	310.4
85.000	True	3	0.0001296	310.1
86.000	True	3	0.0002362	309.4
87.000	True	3	0.0000138	308.8

Calculation Summary (1: Base)
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Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
88.000	True	2	0.0007785	308.3
89.000	True	2	0.0003077	307.7
90.000	True	4	0.0004438	307.5
91.000	True	4	0.0006812	306.9
92.000	True	2	0.0009418	306.4
93.000	True	3	0.0000113	305.8
94.000	True	2	0.0003989	305.2
95.000	True	2	0.0000336	304.6
96.000	True	2	0.0001150	304.0
97.000	True	2	0.0000414	303.4
98.000	True	2	0.0000396	302.8
99.000	True	2	0.0000406	302.1
100.000	True	2	0.0000426	301.5
100.900	True	4	0.0001169	0.0
101.000	True	2	0.0004004	0.0
102.000	True	4	0.0001878	0.0
103.000	True	4	0.0001395	0.0
104.000	True	4	0.0001097	0.0
105.000	True	3	0.0008542	0.0
106.000	True	3	0.0001270	0.0
107.000	True	2	0.0001425	0.0
108.000	True	2	0.0001258	0.0
109.000	True	1	0.0004159	0.0
110.000	True	1	0.0001092	0.0
111.000	True	1	0.0001115	0.0
112.000	True	1	0.0001077	0.0
113.000	True	1	0.0001091	0.0
114.000	True	3	0.0002311	0.0
115.000	True	4	0.0001214	0.0
116.000	True	3	0.0003577	0.0
117.000	True	4	0.0003648	0.0
118.000	True	6	0.0008235	0.0
119.000	True	5	0.0002757	0.0
120.000	True	5	0.0007239	0.0
121.000	True	6	0.0005786	0.0
122.000	True	7	0.0004941	0.0
123.000	True	7	0.0006311	0.0
124.000	True	7	0.0006699	0.0
125.000	True	7	0.0006299	0.0
126.000	True	7	0.0006566	0.0
127.000	True	4	0.0001067	0.0
128.000	True	4	0.0000934	0.0
129.000	True	3	0.0006809	0.0
130.000	True	3	0.0001421	0.0
131.000	True	2	0.0001299	0.0
132.000	True	2	0.0001240	0.0

Calculation Summary (1: Base)
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Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
133.000	True	1	0.0003629	0.0
134.000	True	1	0.0001139	0.0
135.000	True	1	0.0001143	0.0
136.000	True	1	0.0000959	0.0
137.000	True	1	0.0000992	0.0
138.000	True	3	0.0002313	0.0
139.000	True	4	0.0001328	0.0
140.000	True	3	0.0003647	0.0
141.000	True	4	0.0003671	0.0
142.000	True	6	0.0008552	0.0
143.000	True	5	0.0002335	0.0
144.000	True	5	0.0007298	0.0
145.000	True	6	0.0005741	0.0
146.000	True	7	0.0005028	0.0
147.000	True	7	0.0006151	0.0
148.000	True	7	0.0006473	0.0
149.000	True	7	0.0006477	0.0
150.000	True	7	0.0006609	0.0
151.000	True	4	0.0001061	0.0
152.000	True	4	0.0000955	0.0
153.000	True	3	0.0006866	0.0
154.000	True	3	0.0001290	0.0
155.000	True	2	0.0001291	0.0
156.000	True	2	0.0001042	0.0
157.000	True	1	0.0003305	0.0
157.300	True	4	0.0004561	310.4
158.000	True	3	0.0005390	309.9
159.000	True	3	0.0000541	309.3
160.000	True	3	0.0000108	308.7
161.000	True	2	0.0005827	308.1
162.000	True	4	0.0006975	307.9
163.000	True	4	0.0005051	307.3
164.000	True	2	0.0009800	306.7
165.000	True	3	0.0000133	306.1
166.000	True	2	0.0003985	305.5
167.000	True	2	0.0000303	305.0
168.000	True	2	0.0001133	304.3
169.000	True	2	0.0000260	303.7
170.000	True	2	0.0000240	303.1
171.000	True	2	0.0000218	302.5
172.000	True	2	0.0000212	301.9
173.000	True	2	0.0000225	301.2
173.400	True	4	0.0000650	0.0
174.000	True	3	0.0001905	0.0
175.000	True	4	0.0002296	0.0
176.000	True	5	0.0000686	0.0

Calculation Summary (1: Base)
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Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
177.000	True	4	0.0001154	0.0
178.000	True	3	0.0006245	0.0
179.000	True	2	0.0003438	0.0
180.000	True	2	0.0001029	0.0
181.000	True	1	0.0007741	0.0
182.000	True	1	0.0001675	0.0
183.000	True	1	0.0001290	0.0
184.000	True	1	0.0001219	0.0
185.000	True	1	0.0001276	0.0
186.000	True	3	0.0002301	0.0
187.000	True	4	0.0001246	0.0
188.000	True	3	0.0003636	0.0
189.000	True	4	0.0003361	0.0
190.000	True	6	0.0008252	0.0
191.000	True	5	0.0002611	0.0
192.000	True	5	0.0007356	0.0
193.000	True	6	0.0005921	0.0
194.000	True	7	0.0005311	0.0
195.000	True	7	0.0006044	0.0
196.000	True	7	0.0006611	0.0
197.000	True	7	0.0006580	0.0
198.000	True	7	0.0006347	0.0
199.000	True	4	0.0001067	0.0
200.000	True	4	0.0000970	0.0
201.000	True	3	0.0006813	0.0
202.000	True	3	0.0001435	0.0
203.000	True	2	0.0001354	0.0
204.000	True	2	0.0001119	0.0
205.000	True	1	0.0003480	0.0
206.000	True	1	0.0001067	0.0
207.000	True	1	0.0001087	0.0
208.000	True	1	0.0001051	0.0
209.000	True	1	0.0001067	0.0
210.000	True	3	0.0002282	0.0
211.000	True	4	0.0001063	0.0
212.000	True	3	0.0003506	0.0
213.000	True	4	0.0003736	0.0
214.000	True	6	0.0008370	0.0
215.000	True	5	0.0002580	0.0
216.000	True	5	0.0007490	0.0
217.000	True	6	0.0006107	0.0
218.000	True	7	0.0005294	0.0
219.000	True	7	0.0006055	0.0
220.000	True	7	0.0006582	0.0
221.000	True	7	0.0006516	0.0
222.000	True	7	0.0006521	0.0

Calculation Summary (1: Base)
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Time (hours)	Balanced?	Trials	Relative Flow Change	Flow Supplied (gpm)
223.000	True	4	0.0001045	0.0
224.000	True	4	0.0001039	0.0
225.000	True	3	0.0006813	0.0
226.000	True	3	0.0001301	0.0
227.000	True	2	0.0001308	0.0
228.000	True	2	0.0001089	0.0
229.000	True	1	0.0003339	0.0
229.100	True	4	0.0004532	310.4
230.000	True	3	0.0009777	309.7
231.000	True	3	0.0000242	309.1
232.000	True	3	0.0000100	308.5
233.000	True	2	0.0004648	308.0
234.000	True	4	0.0002716	307.8
235.000	True	4	0.0005742	307.1
236.000	True	2	0.0009652	306.6
237.000	True	3	0.0000112	306.0
238.000	True	2	0.0003980	305.4
239.000	True	2	0.0000312	304.8
240.000	True	2	0.0001145	304.2

Flow Demanded (gpm)	Flow Stored (gpm)
67.4	19.2
0.0	0.0
0.0	312.0
0.0	311.6
0.0	311.1
0.0	310.5
0.0	310.0
0.0	309.4
0.0	308.8
419.5	-110.5
103.1	205.2
79.1	228.9
79.1	228.4
79.1	228.0
79.1	227.6
79.1	227.1
79.1	226.7
79.1	226.2
79.1	225.8
79.1	225.3
237.3	67.0
79.1	224.6
55.4	247.8
23.7	278.9

Calculation Summary (1: Base)
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Flow Demanded (gpm)	Flow Stored (gpm)
7.9	294.1
7.9	293.6
7.9	-7.9
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
419.5	-419.5
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
237.3	-237.3
79.1	-79.1
55.4	-55.4
23.7	-23.7
7.9	7.9
7.9	-7.9
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
419.5	419.5
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
237.3	-237.3

Calculation Summary (1: Base)
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Flow Demanded (gpm)	Flow Stored (gpm)
79.1	-79.1
55.4	-55.4
23.7	-23.7
7.9	-7.9
7.9	-7.9
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
419.5	-419.5
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	231.2
79.1	231.0
79.1	230.3
79.1	229.7
79.1	229.2
79.1	228.6
237.3	70.2
79.1	227.8
55.4	251.0
23.7	282.1
7.9	297.3
7.9	296.7
0.0	304.0
0.0	303.4
0.0	302.7
0.0	302.1
0.0	301.5
0.0	0.0
0.0	0.0
0.0	0.0
419.5	-419.5
103.1	-103.1
79.1	-79.1
79.1	79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	79.1

Calculation Summary (1: Base)
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Flow Demanded (gpm)	Flow Stored (gpm)
79.1	-79.1
79.1	-79.1
79.1	-79.1
237.3	-237.3
79.1	-79.1
55.4	-55.4
23.7	-23.7
7.9	-7.9
7.9	-7.9
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
419.5	-419.5
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
237.3	-237.3
79.1	-79.1
55.4	-55.4
23.7	-23.7
7.9	-7.9
7.9	-7.9
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
419.5	-419.5
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1

Calculation Summary (1: Base)

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Flow Demanded (gpm)	Flow Stored (gpm)
79.1	-79.1
79.1	231.3
79.1	230.8
79.1	230.1
79.1	229.6
79.1	229.0
237.3	70.6
79.1	228.1
55.4	251.4
23.7	282.4
7.9	297.6
7.9	297.0
0.0	304.3
0.0	303.7
0.0	303.1
0.0	302.5
0.0	301.8
0.0	301.2
0.0	0.0
0.0	0.0
419.5	-419.5
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
237.3	-237.3
79.1	-79.1
55.4	-55.4
23.7	-23.7
7.9	-7.9
7.9	-7.9
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
419.5	-419.5
103.1	-103.1

Calculation Summary (1: Base)

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Flow Demanded (gpm)	Flow Stored (gpm)
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
237.3	-237.3
79.1	-79.1
55.4	-55.4
23.7	-23.7
7.9	-7.9
7.9	-7.9
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
419.5	-419.5
103.1	-103.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	-79.1
79.1	231.3
79.1	230.6
79.1	230.0
79.1	229.4
79.1	228.9
237.3	70.5
79.1	228.0
55.4	251.2
23.7	282.3
7.9	297.5
7.9	296.9
0.0	304.2

VITA

Neha Mangesh Bhadbhade

Candidate for the Degree of

Master of Science

Thesis: PERFORMANCE EVALUATION OF A DRINKING WATER DISTRIBUTION SYSTEM USING HYDRAULIC SIMULATION SOFTWARE FOR THE CITY OF OILTON, OKLAHOMA

Major Field: Environmental Engineering

Biographical:

Personal Data: Daughter of Mangesh and Rupa Bhadbhade
Date of Birth: 12/22/1982
Country of Citizenship: India

Education: Earned a Bachelor's Degree in Engineering in Instrumentation and Control Engineering from University of Pune, India in 2004. Completed the requirements for the Master of Science or Arts in your major at Oklahoma State University, Stillwater, Oklahoma in December of 2009

Experience: Research and Development Engineer in Manas Microsystems Pvt Ltd in Pune, India. Teaching Assistant for Oklahoma State University in Stillwater, Oklahoma. Research Assistant for Oklahoma State University in Stillwater, Oklahoma.

Professional Memberships:
Chi Epsilon

Name: Neha Mangesh Bhadbhade Date of Degree: December 2009

Institution: Oklahoma State University Location: Stillwater, Oklahoma

Title of Study: PERFORMANCE EVALUATION OF A DRINKING WATER DISTRIBUTION SYSTEM USING HYDRAULIC SIMULATION SOFTWARE FOR THE CITY OF OILTON, OKLAHOMA

Pages in Study: 105 Candidate for the Degree of Master of Science

Major Field: Environmental Engineering

Scope and Method of Study:

The scope of the study was to evaluate the performance of drinking water distribution system of the City of Oilton, Oklahoma using hydraulic simulation software and recommend changes based on the findings which would help the City of Oilton operate their system with more efficiency. This study was a part of a larger project, the goal of which was to develop a cost-effective Decision Support Tool to help the Rural Water Districts of Oklahoma in the long term planning of their water infrastructure. The project was funded by the Oklahoma Water Resources Research Institute. The hydraulic simulation software used for the study was WaterCAD V8i which is a licensed software distributed by the Bentley Systems. The methodology incorporated briefly, was to identify the potential consumers around the demand nodes in the distribution network and assign these nodes base demands depending upon the number of consumers around that node. The behavior of the system was observed under base flow, peak flow, fire flow and diurnal flow conditions and its performance was evaluated based on mainly two things: hydraulic conditions and water quality.

Findings and Conclusions:

It was observed that the City of Oilton has very large storage tanks. The total storage capacity in the City is 950,000 gallons compared to a small average daily demand of 118,000 gallons. Since the supply of water to the town is based on gravity it was observed that only the top 20 ft of water volume in the tanks was used to maintain the normal working pressures above 30 psi. It was observed that the existing pump took excessively large time of 2 and ½ days to fill the tanks. Thus, the recommendation made to the City was to use a larger pump in order to fill the tanks at a faster rate. The other recommendations made were replacement of old pipes with new in order to have more flow through them and perform routine coliform tests to avoid state violations.

ADVISER’S APPROVAL: Dr. Dee Ann Sanders