Final Report

Oklahoma Water Resources Research Institute

Title: Decision Support Model for Evaluating Alternative Water Supply Infrastructure Scenarios

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Section 1: Problem and Research Objectives

Rural water systems often struggle to make decisions regarding their future, particularly when those decisions involve upgrading their infrastructure or consolidating / cooperating with other systems. This study demonstrates the development of a step-by-step methodology that provides assistance to rural water systems for planning and updating their water supply infrastructure. The objective of this study is to create a process that allows a rural water system to assess their own infrastructure and consider different avenues for funding potential enhancements. Specific steps involved in this process are discussed in depth in the report that follows, but a high-level overview includes the ability for a rural water system to perform the following:

1) Develop a list and sources of data required for modeling possible infrastructure upgrades (including maps / information on the infrastructure itself)
2) Create a hydraulic simulation model for the water system, using free or low-cost software
3) Determine problem areas and potential solutions to these problems
4) Estimate capital and operating costs for alternative solutions; gather information on potential funding sources and consider grant or loan-writing options

The methodology described in this project is generalizable to any number of rural water systems, including those using either surface or groundwater. While we initially hoped the tools and methods used under this methodology would be able to be performed by non-specialists, such as local water district managers, our experience indicates that some specialist oversight is likely necessary. This system of evaluation should still dramatically enhance the capability of rural water districts to understand the limitations of their current system and give updates to the local community well in advance of any infrastructure crisis.

Section 2: Background and Methodology

The 2007 Environmental Protection Agency (EPA) report of Drinking Water Infrastructure Needs Survey and Assessment stated that the United States would need an investment of 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, 2007). 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: (OWRB, 2009)

- 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 existing demands
- Identify the necessary changes in the system to meet future water demands

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. Four rural water systems were chosen, representing systems with above ground storage, below ground storage, groundwater sources, and surface water sources. The four systems chosen were Beggs, Oklahoma, Braggs, Oklahoma, Kaw City, and Oilton, Oklahoma. These systems represented a variety of infrastructure issues, including insufficient storage, old pipes, and low pressure areas. In addition to the options in the systems selected, two different water distribution models were used during the project. The locations of the towns are shown in Figure 1 while the population, source of water, and general problems for these towns is given in Table 1.

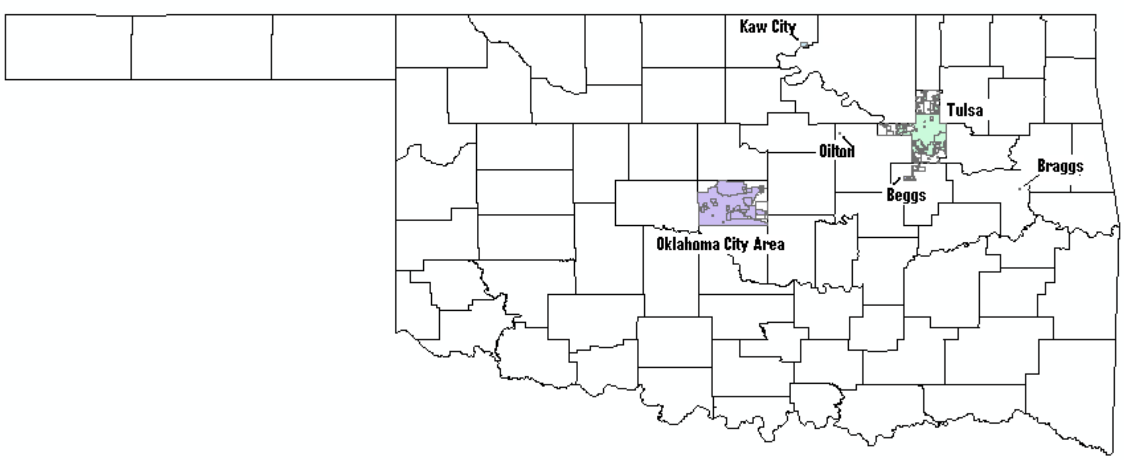

Figure 1. Location of Study Site Towns of Beggs, Braggs, Kaw City, and Oilton in Oklahoma

Table 1. Small Towns in Oklahoma Participating in Study of Water System Planning.

Item	Beggs	Braggs	Kaw City	Oilton
Population	1,364	1,030	400	1,200
Water Source [a]	S.W.	G.W.	G.W.	G.W.

Gallons/Day (thos)	161	75.6	80	118
Treatment	Conventional.	NR	NR	NR
Storage (thos. gal.)	175[b]	200	200	950
Issues: Old pipes	Yes	Yes	Yes	Yes
Insufficient storage	Yes	No	Yes[c]	No
Low Pressure Areas	Yes	Yes	No	Yes
Sufficient Fire Flow	No	Yes	Yes	No
Primary Standards	ok	ok	ok	ok
Secondary Standards	ok	ok	Mn	ok
Water Age	Some[d]	Some[d]	Some[d]	Some[d]

[a] Abbreviations used; S.W. = surface water, G.W. = ground water, NR = not required.
[b] 50,000 elevated plus 125,000 in ground tank.
[c] During summer tourist weekends.
[d] Generally in areas served by long un-looped pipes

Only one of the four towns had a digital pipeline data set. In some cases, the hand drawn pipeline maps were incomplete. The approach in this study was to develop a hydrological simulation model for the town and then use that model to address the problems shown above in Table 1. The following approach was followed.

1) Contact and meet with appropriate local officials such as the mayor, manager, and/or city engineer.
2) Obtain copies of pipe line maps noting length, diameter, age, material, and condition, if possible. Alternatively sketch pipeline maps onto Google or Tiger line drawings of the city. Handheld GPS units were used to verify the location of critical infrastructure such as wells, treatment plants, and water storage units.
3) Obtain available technical information about the pumps, (size, power, model, age, power consumption, and hours of operation) and other system components.
4) Develop and validate an EPANET or WaterCAD simulation model for the water system.
5) Use the EPANET or WaterCAD models to evaluate the ability of the system to meet time of day demands by spatial location. The EPANET hydrological simulation program was developed by EPA and is available at no cost. WaterCAD is a commercial system distributed by Bentley Systems.
6) Determine the ability of the water distribution system to meet fire flow demands at each hydrant (minimum 20 psi after two hours of 250 gpm flow).
7) Evaluate the type, amount, and time of infrastructure needs to meet projected population growth.

The above general steps were refined as discussed below for the four systems evaluated.

<u>Simulation Model Development</u>

The Oklahoma Water Resources Board (OWRB) has developed a set of GIS pipeline drawings for some 800 rural water systems in Oklahoma; however, these drawings typically do not include small towns such as the ones included in this project. The first step was to develop the geographical information system (GIS) drawings of the major pipelines serving the city.

Zonum Systems (2009) has developed several freeware interfaces to EPANET. One program, (EPANETZ) allows the user to digitize pipelines onto a Google Map of the town. Comparison of the Google map of the town with engineering drawings permits development of a digital infrastructure map with approximate (though not exact) location of pipelines. The program automatically creates the necessary linkages between nodes. The user must enter the pipe diameters and the node elevations. The GIS will provide estimates of the length of pipes, but actual lengths should be used when these are available. Two or more pipes are considered joined if they share the same node. One problem in getting EPANET to operate, is that slight differences in placement of pipe lines may generate multiple nodes which appear as a single node on one location. More expensive simulation programs link such nodes automatically. Excel macros were written to check the differences in latitude and longitude between nodes and ask the user if pipes having separate ending nodes within a specified radius should be connected, essentially requiring user verification for each unconnected node.

A second problem encountered was how to determine the elevation of each node, which is a required input for determining water flow. This is difficult for inexperienced users to accomplish in ARCVIEW or ARCMAP. However, a second relatively inexpensive GIS program, Global Mapper, was available that creates XYZ files (which include elevation) by simply overlaying the line drawing of the pipes on a USGS elevation file. Visual Basic macros were then used to add the elevations to the pipeline nodes. (Zonum Solutions (2009) now offers an online program to add elevations to nodes). The values relating to the depth of wells, height and volume of storage facilities, pump curves, rules for pump operation, and diurnal water use patterns must be added to the data set. The effect of corrosion in reducing pipe flows was also approximated after discussions with the city engineer.

The following three sections (sections 2.1 – 2.3) discuss the steps taken to evaluate the distribution systems in Beggs, Oilton, and Braggs, respectively. As indicated, free EPANET software was used in both Beggs and Braggs, while Oilton incorporated the for-fee WaterCAD software typically used by professional engineers. A discussion of the issues faced during each simulation is included. Additionally, the analysis of Beggs (which was completed prior to Oilton and Braggs) incorporates a methodology for assessing the cost of potential upgrades to the existing infrastructure. Finally, a fourth section (section 2.4) describes of the investigation for Kaw City. This analysis differs from the previous three projects in that it deals with the assessment of several options to improve the city's water supply (comparing costs of various new treatment plants) while also providing water to at least one other entity. The Kaw City analysis focuses more on the cost of construction and operation alternatives rather than simulation of the existing infrastructure.

Section 2.1: Simulation Model to Evaluate the Beggs Water Distribution System

The EPANET model developed for the City of Beggs will be used to illustrate the capabilities of the water simulation software to analyze problems and possible solutions for a small town (Lea, 2009).

Figure 2 shows the digitized pipeline for the City of Beggs overlaid on photo map of the city. The low pressure areas indicated by circles along with the areas where the age of water in the pipes was problematic in Figure 2 were confirmed by the city engineer. One area with pressure problems and inadequate fire flow was on the west end where the primary and secondary schools were located. A similar problem was encountered with the "Hilltop" area on the east. Both of these areas represent city expansions made after the initial water system was developed. The dead ends associated with several long pipes also failed the fire-flow test.

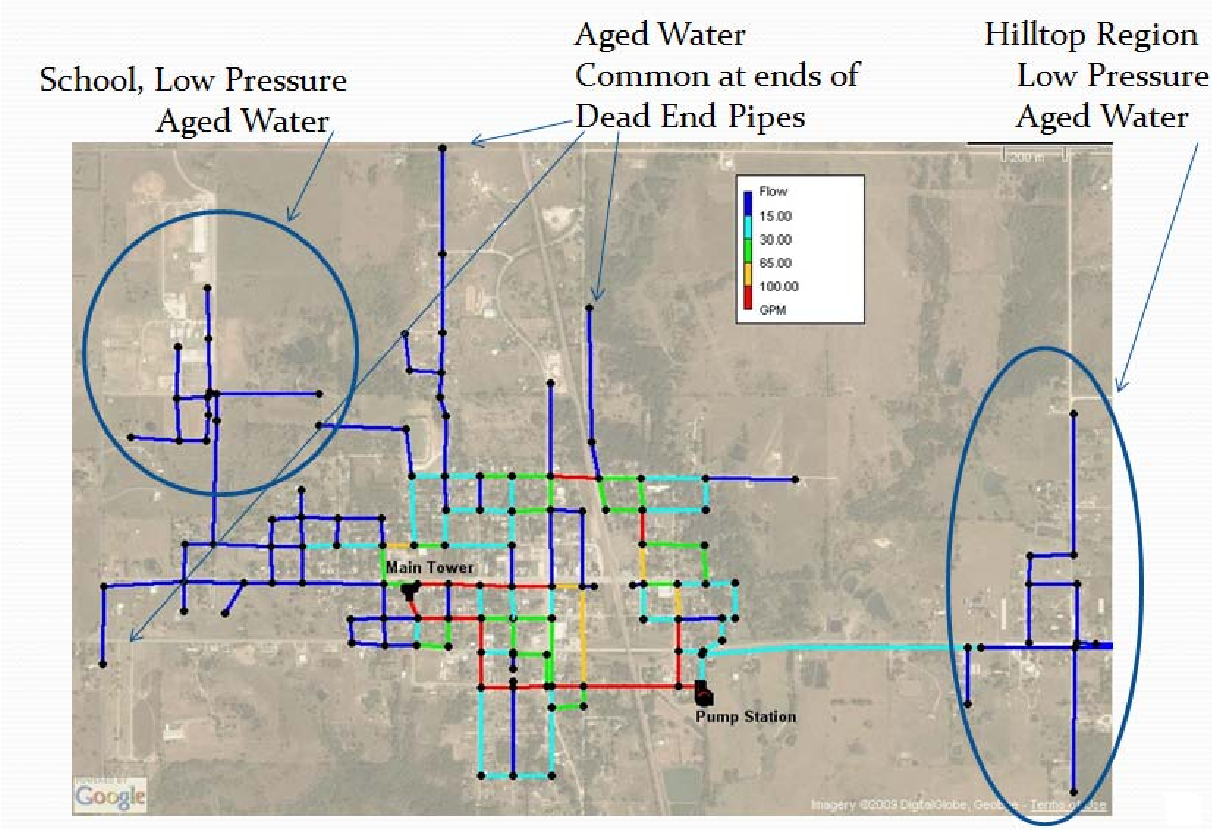

Figure 2. Digitized EPANET Model of the Water System for Beggs, Oklahoma Showing Pipeline Flow and Indications of Areas with Low Pressure and Areas Where Age of Water in the Pipes was Problematic.

The alternatives simulated to correct the problems shown in Figure 2 included installing new or modified pumps, a new water tower on the east end of town, replacing old pipes that had corrosive deposits, and / or adding new pipes to eliminate dead ends and create new water paths.

A set of simulations involving the addition of new pipes to convert the long single pipes shown in Figure 2 into loops indicated the problem of water age could be remedied most of the pressure and fire flow problems could be resolved. The pipes and water tower added during the simulations are shown in Figure 3.

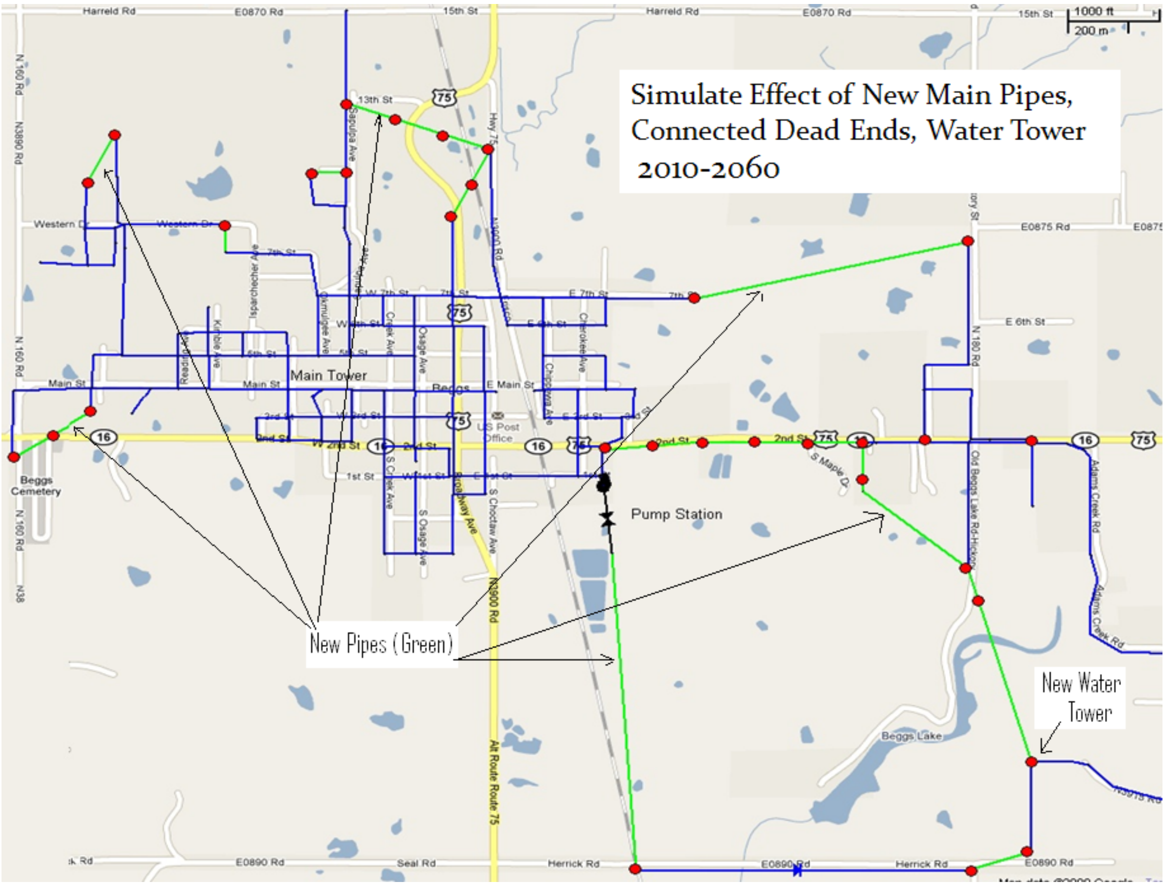

Figure 3. EPANET Model of the Water System for Beggs with Pipes added to Eliminate Dead Ends and the Location of a New Water Tower.

An important issue for a small town like Beggs, which is currently facing sewer upgrade problems, is the cost and the best order in which make modifications.

Cost estimates for pipe, pumps, and storage tanks were obtained from Means (2009) and adjusted were necessary to account for price changes since publication. Table 2 shows the prices used to the cost of installing PVC pipe of alternative diameters. A spreadsheet was used to develop the cost for the purchase and installation cost of alternative sizes of PVC pipe from 2 through 8 inches, using data on the cost of pipe, excavation, and backfilling used estimates from Means (2009).

Table 2. Costs Used for AWWA 160 SDR-18 PVC[a] Pipe, Trenching, and Backfilling

Diameter	Pipe	Trenching[b] and Backfill	Total Cost
Inches	$/LF	$/LF	$/LF
2	2.24	3.47	5.72
3	5.01	3.55	8.56
4	6.12	3.62	9.75
6	8.62	3.77	12.40
8	12.17	3.92	16.10

[a] Polyvinyl Chloride pipe.
[b] Assuming the pipe is placed in a two foot wide trench so the top of the pipe is 36 inches below the surface.

The problem of choosing the most economical diameter for single pipe to deliver a given volume with a designated head or pressure at the delivery can be determined by enumeration. For each diameter, add the annualized installation cost of the pipe to the annual cost of energy required to force the water through the pipe. Choose the diameter with the smallest annual total cost. Suppose it is necessary to purchase pipe that will deliver 100 gpm over a mile and up into an 80 foot tank. The amount of brake horsepower required is calculated as

$$\text{bhp} = \frac{\text{Head ft} * \text{GPM}}{(3960*\text{pe}*\text{me})},$$

where

pe is the pump efficiency, for example 0.7, and
me is the motor efficiency, for example 0.91.

If an electric motor is used, the amount of electricity used per year is 0.746 * bhp *8760 hours. The total feet of head required is equal to the 80 feet of lift into the tank plus the head (pressure) necessary to force 100 gpm of water through one mile of pipe of a given diameter. According to the Hazen-Williams formula, the head loss is,

$$\text{Hloss (ft)} = \frac{10.51\ (\text{GPM}/C)^{1.85}\ \text{Length}}{D^{4.87}}$$

Where

C is a Hazen-Williams friction coefficient, assumed to be 140 for PVC pipe
D is the inside diameter of the pipe in inches
Length is the length of the pipe.

The minimum annual cost involves a tradeoff between pipe size and energy cost. As the diameter of the pipe increases, the total cost of the pipe increases, but the energy required to force the water through the pipe decreases. A standard capital recovery factor was used to annualize the cost of the pipe. The annual capital cost for one mile of pipe (Table 2) and the annual pumping costs are added together in Figure 4. The least cost alternative is the four-inch diameter pipe that would cost $7,000 per year.

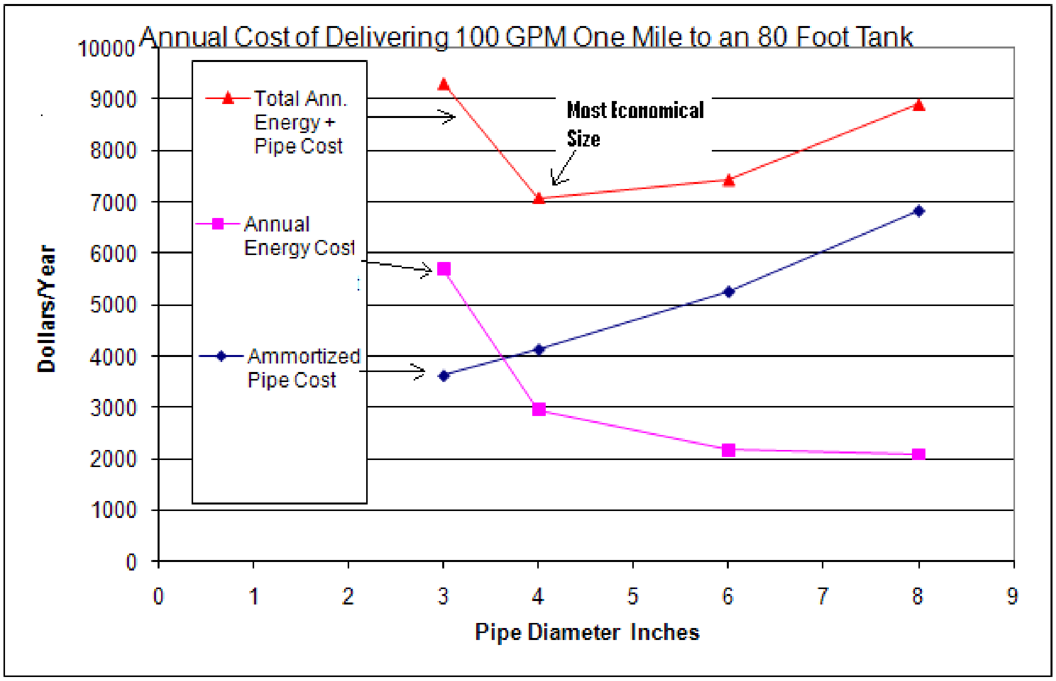

Figure 4. Comparison of Annual Total, Capital, and Energy Cost to Install One Mile of PVC Pipe with a 20-year Life to Deliver 100 GPM to an 80 Foot Tank when Electricity Costs are $0.10 per kwh and the Interest Rate is Five Percent.

However, in a water system the problem is more complicated since a new pipe will be used in a net work with other pipes. Also, Oklahoma mandates require that if a fire hydrant is attached to the pipe, the minimum diameter would have to be six inches. Alternative simulation runs were used to compare the system performance in terms of pressures and energy cost before and after each change in the distribution system infrastructure. The capital costs associated with different solutions were calculated outside the simulation.

The ability to meet fire flow requirements at each fire hydrant node was tested by adding a 250 gpm demand to each node in turn and testing the pressure after a two hour simulation. The full set of fire node tests were repeated after each set of infrastructure changes. Excel macros were again used to write out the simulation input data, run the simulation, retrieve the results of each simulation, and determine the number of fire flow and other failures in the system. A set of incremental infrastructure investments was developed that maximized the number of new fire hydrant nodes meeting the fire flow test per dollar spent. The results are shown in Table 3. In Table 3, the greatest initial improvement per investment dollar came from adding the two major pipes in the eastern part of Beggs. At the bottom of Table 3, the additional water tower in eastern Beggs, added onto the previous changes, had the fewest improvements per dollar spent.

Table 3. Order of Changes in Beggs Water Distribution System to Maximize Fire-Flow Compliance per Dollar Invested.

Order	Description of Changes	Cost
1	Install two major pipes in East Beggs	$69,000
2	Add three additional pipes in East Beggs to finish addressing Hilltop pressure problems	$60,000
3	Add remaining pipes to eliminate targeted dead ends	$57,000
4	Add Additional Fire Hydrants	$60,500
5	Add 50,000 gallon water tower in East Beggs	$167,000
	Total All Changes	$415,000

For a detailed analysis of each step of the Beggs analysis (including all data sources, software modeling inputs, and cost estimation methodology), see Lea (2009).

Section 2.2: Simulation Model to Evaluate the Oilton Water Distribution System

The City of Oilton is located in Creek County and is approximately 54.6 miles to the west of Tulsa. 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 five 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. The main pipeline that brings water to the city is an eight-inch asbestos cement pipeline. There are two main distribution pipes in the town, one of which is an eight-inch PVC pipeline while the other is an 8 inch asbestos cement pipeline. All other mains and sub-mains are in the range of 1 to 6 inches. A summary of the statistics of the Oilton water distribution system is shown below in Table 4. Figure 5 shows the map of the town. Figure 6 shows a schematic of the distribution system.

Table 4: Oilton System Statistics
• Source: 2 deep (approx. 500 ft) wells • Pumps: Single submersible pump per well • Total Storage: 950,000 gal • Pumping Rate: 118,000 gal/day (81 gpm) • Population Served: 1200

<u>Selection of hydraulic simulation software for Oilton, Oklahoma</u>

The hydraulic simulation software used for this part of the study was WaterCAD V8i distributed by Bentley Systems. The aim of this project was to provide an economic tool which would be affordable to rural water districts. However, after completion of the previous study carried out for the Beggs water system, it was evident that the free hydraulic simulation software used (EPANET) was 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. Although rural water system personnel are not likely to be able to use WaterCAD, most professional civil engineers do have knowledge of the software and a demonstration of its applicability to rural systems can potentially aid future efforts to assist these communities.

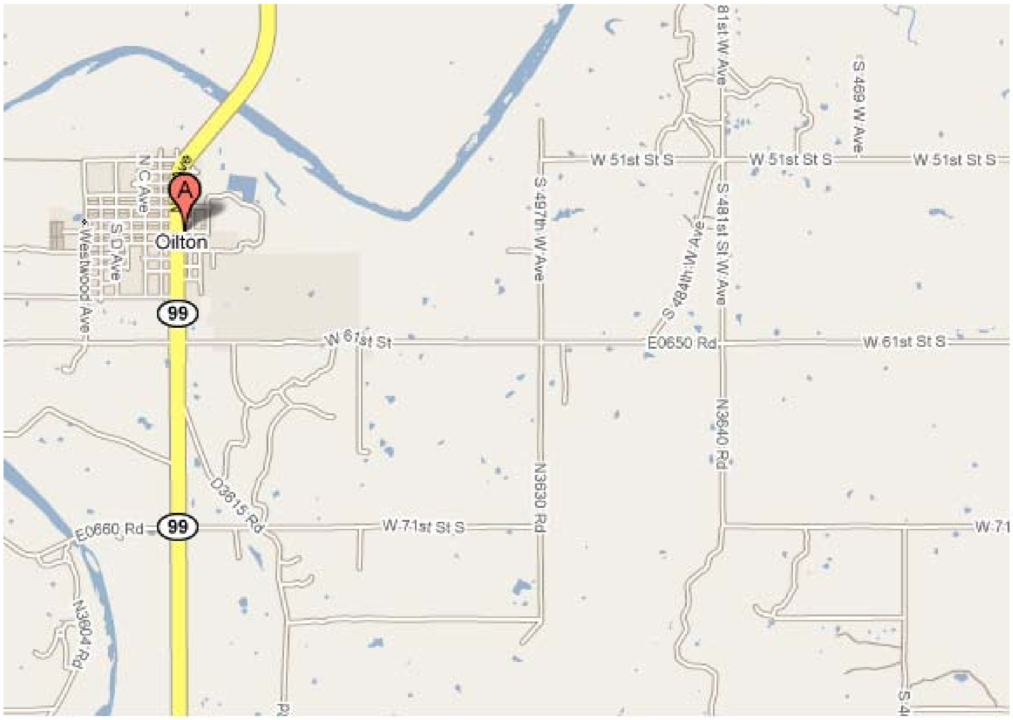

Figure 5: Map of Oilton, Oklahoma, Area (Google Maps, 2009)

To use the simulation software, the following steps were followed:

1. Pipelines were digitized, from information gathered on location (x-y coordinates), length, and diameter.
2. Facilities were located, including treatment plants, wells, pumps, and towers/standpipes.
3. Unknowns at this point included
    - Elevation Changes along pipeline
    - Location of Users along pipeline
    - Demand allocation along pipeline
    - Age, Condition, Materials

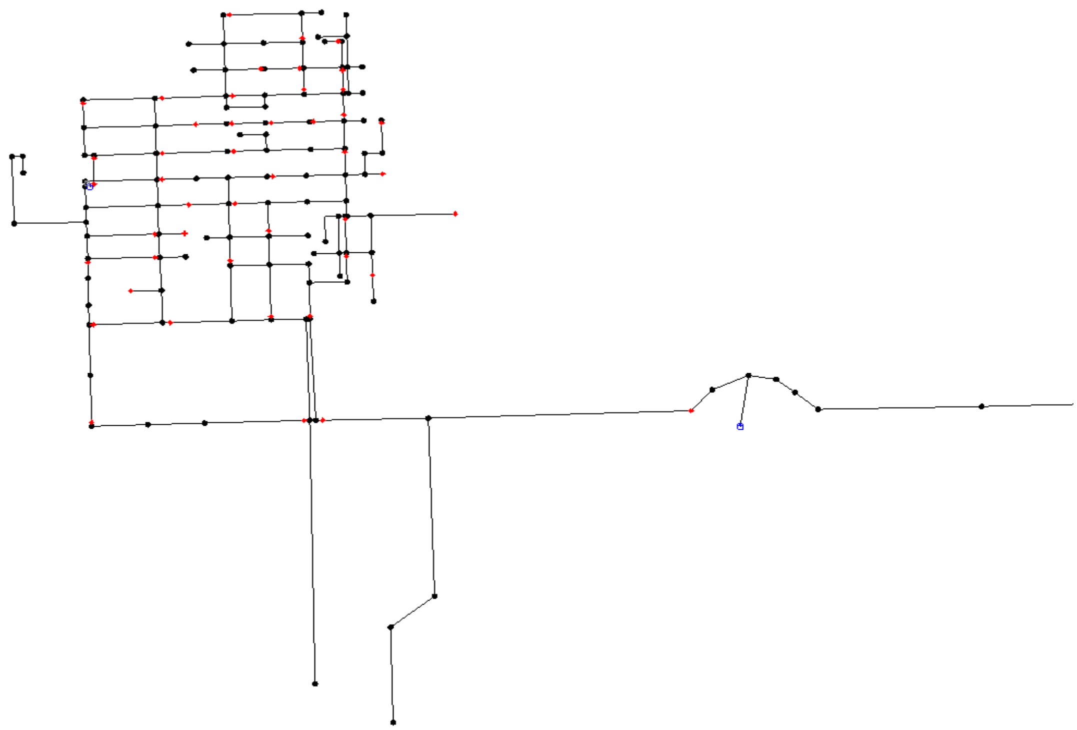

Figure 6: Schematic of Oilton, Oklahoma, Water Distribution System (Bhadbhade, 2009)

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). Note that this elevation data source is different than that used for the Beggs simulation. 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, and the Census 2000 Block data was used to match households to potential nodes. 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.

<u>Oilton Simulation Results</u>

- Very large storage results in long water age and excessively long (several days) pump cycles to fill the tanks.
- However, most storage volume is unusable due to low pressures that result when water in standpipes is dropped more than 30 ft from the top of the tanks.
- Excessively long, low-demand lines result in high water age and low disinfectant residuals at dead ends.

For a detailed analysis of the Oilton simulation using WaterCAD, see Bhadbhade (2009).

Section 2.3: Simulation Model to Evaluate the Braggs Water Distribution System

Braggs is located in eastern Oklahoma, 56 miles south east of Tulsa (Figure 7). The population of the city is 308. The largest section of the existing water distribution system was installed in 1982 and has been serving the local population and 650 people in surrounding areas for the last 27 years.

BRAGGS, OKLAHOMA

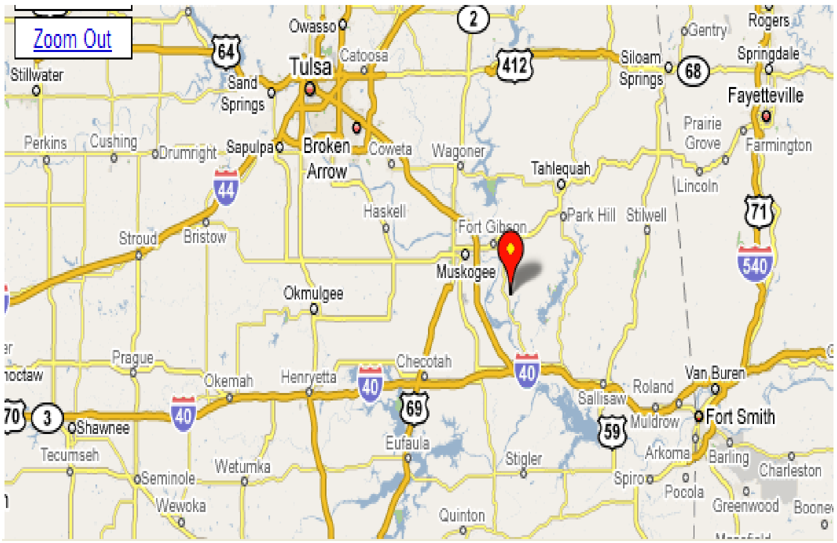

Figure 7: Map of the Braggs, Oklahoma, area (Google Maps, 2009)

Currently the system has 416 service connections and serves 1030 people from its primary water source which is ground water artesian wells. The distribution system network consists of three water towers; one located in the center, one at the north end and one on the south end of the city, giving a total storage capacity of 200,000 gallons. A summary of the statistics for the Bragg distribution system is shown in Table 5.

Table 5: Braggs System Statistics

- Source: Artesian Wells
- Pumps: 3 identical working in parallel
- Total Storage: 200,000 gal
- Pumping Rate: 75,600 gal/day (52.5 gpm)
- Service Connections: 416
- Population Served: 1030

The piping consists mainly of long two inch branch pipes which are interconnected by a few four and six inch supply mains.

The map of the Braggs water distribution system was obtained from the Water Information Mapping System (WIMS) on the Oklahoma Water Resources Board (OWRB) website at http://www.owrb.ok.gov/maps/server/wims.php. WIMS is an Internet-based map server that requires a supported web browser. The Braggs system is shown in Figure 8.

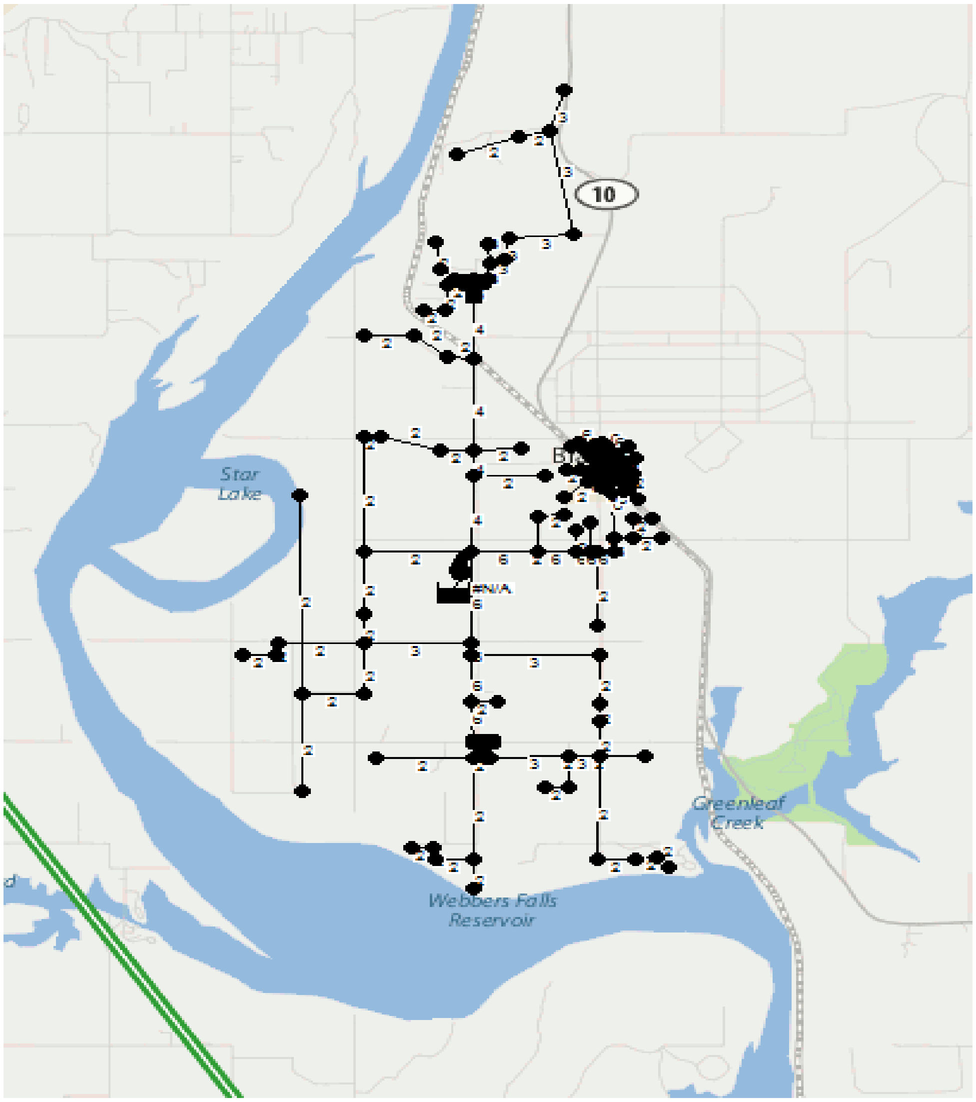

Figure 8: Schematic of Water Distribution Pipelines in Braggs, Oklahoma, Service Area from EPANETZ

Information regarding the age of the system, problems related to inadequate flows, low water pressure, leakages and bursts water usage patterns and equipment information for pumps was obtained from interviews with the plant operator at Braggs, Oklahoma. Water usage data were obtained from the Oklahoma Department of Environmental Quality (ODEQ) records. The records included information regarding the total water pumped daily from the treatment plant, the pH and the doses of the different chemicals added to the water prior to distribution over an eight year period from January 2001 to April 2009.

Hydraulic modeling using EPANET for Braggs, Oklahoma

The process of modeling a network using EPANET involves input of the parameters or variables that most closely describe the operation of the actual system. These parameters include the shape of the tanks, the pump curve which describes the operation of the pump and an infinite reservoir. Other input parameters required for the model to run include the maximum and minimum water levels and an initial water level in the tank. The three water tanks at Braggs are all cylindrical in shape.

There are three identical pumps at Braggs, each delivering 150gpm at 208ft of head. The pumps operate in parallel delivering the same head and are set to sequentially come on line in order to meet increasing flow requirements for the system. The pumps were modeled according to the information received from the system operator. Usually a single pump is switched on when the pressure drops below 65 psi and is switched off when the pressure exceeds 80psi. Therefore, rule based controls were set within the EPANET model to ensure that the first pump was switched on when the pressure dropped below 65psi and switched off when the pressure increased to 80psi. Pump 2 was modeled to switch on if the pressure dropped further as would be the case in the event of a fire. Pump 3 was treated as a standby for the system in case pumps 1 or 2 failed to operate and was not included in the hydraulic modeling process.

The greatest percentage of the pipes at Braggs were installed in 1982 when the currently existing PVC pipes were installed to replace deteriorated cast iron pipes that had been previously installed in the 1940's. Therefore, most of the pipes are almost 30 years old. The operator noted that they had not replaced any pipes recently.

Braggs Simulation Results

- Technical work necessary to use the EPANET software took several months. The software is not user friendly and technical support is non-existent.
- Relatively good records from the operator resulted in good match between simulation and the limited physical system measures (flows and pressures).
- Water age was high and disinfectant residual was predicted to be low in the long dead ends. Looping did not help, since it merely increased the flow paths and further lowered velocities.

For a detailed analysis of the Braggs simulation using EPANET, see Senyondo (2009).

Section 2.4: Cost comparison of alternative treatment plant facilities in Kaw City

Kaw City (located in Kay County has had water problems since the 1990's because of the collapse of one of its wells and from the poor taste of the groundwater. The poor taste is attributed to high levels of minerals such as manganese and iron in groundwater. Chapman (2003) found that the levels of Barium were 0.265 mg/l, iron was 0.071 mg/l and manganese was 0.121mg/l. These chemicals are above the Oklahoma Environmental Secondary Standard. Chapman (2003) also notes the importance of constructing a water treatment plant with capacity of 125 gpm and to use alternative treatment systems to treat the water in order to achieve the quality of the water below secondary standard. Table 6 shows the analysis of the untreated water and the levels that EPA and the State require after water treatment.

Table 6: Organic Compound level in groundwater before and after treatment

Chemicals/Organic compound	Test from the well on the bridge no. 3 (2000).	EPA and State Standard/ primary required level in (units)
Total dissolved solids	637 mg/l	500 mg/l
Turbidity	0.76 NTU	Surface water standard 95% must be < 0.5
iron	0.071 mg/l	0.3 mg/l
manganese	0.121 mg/l	0.05 mg/l
Barium	0.265 mg/l	2 mg/l
Hardness	514 mg/l	Existing hardness is only 152 mg/l

Source: Chapman & Associates (2003)

Another problem with Kaw City's water is its taste and odor. Kaw City officials have specified that they would like to solve the problem of the poor condition and taste of their water while also considering the possibility of selling water to nearby communities (including the city of Shidler). Accordingly, the city requested assistance in estimating the cost of establishing a new well, new water treatment plant, and the necessary extension of pipelines. The presence of Kaw Lake creates an additional tourist demand for water, especially during summer weekends.

To increase the quality of water to solve both the high demand problem and to provide quality drinking water for domestic and other uses (necessary to meet U.S. EPA and Safe Drinking Water Standards), there is a need to develop a comprehensive solution by building a water treatment plant and to use appropriate water treatment systems to treat the water for drinking. The purpose of water treatment is to condition, change and remove the contaminants, to supply safe and good tasting drinking water acceptable to consumers or users (Spellman, 2003). The base water demand for Kaw City and Shidler is 150 gpm. The building of the plant will provide a volume of 216,000 (150*60*24) gpd for the two cities. It will serve customers with its own water and provide portable water to the people of Shidler. Residents hypothesize that this will boost the economic activities of the city, particularly tourism. Because construction costs can be large and vary dramatically by plant type, the city wants to examine both the benefits and costs of the treatment plant and its operations.

This portion of the study estimates the cost of building an alternative treatment plant facility for a reliable supply of drinking water to the people of Kaw City. The focus is on choosing the best (least cost) treatment system while improving quality of the water from the well to the city and other potential buyers. Due to the nature of the chemical compounds in the groundwater, two main treatment systems that is nanofiltration (reverse osmosis) and Aeralator® would be considered because it is the most effective systems of treating water.

Economies of scale dictate that the capacity of the treatment plant needs to increase in order to supply both Kaw City and Shidler. Increasing the capacity of the treatment plant and supply to serve these cities is economically viable and better than building small capacity to serve only Kaw City. Moreover, the expansion of the size and supply will reduce the cost of building treatment plant, the cost of treatment of the water and the distribution of water.

The general of objective of this portion of the study is to determine and compare the cost of building alternative treatment plant facilities in Kaw City. Specific objectives of this research include:

1. To determine the total discounted investment capital cost and annual capital cost of the two possible sizes and types of water treatment plants.
2. To determine the annual operating cost of the two possible sizes and types of water treatment plants.
3. To determine the cost of a new well and the cost of the transmission line from monitoring well to the treatment plant (at the “greenhouse” site) and from the treatment plant to the existing pipeline at Washunga bay.
4. To compare the discounted amortized capital cost and plus the amortized operating cost for two sizes and two types of treatment plant.
5. Determine the cost of replacing the entire Kaw City distribution pipeline.

<u>Estimation of Cost</u>

The best systems among the various water alternative treatments may be selected on the basis of cost of construction, other capital, operating and cost associated with capital and plant maintenance cost over a designated planning area.

Capital Cost

Capital costs are the costs for the physical assets of the project. Capital costs are part of the fixed component of the total cost. They are normally incurred at one time but also include cost of rehabilitation or replacement of equipment during the life of the system. Capital costs are typically estimated for equipment, materials, construction, and other assets. Capital costs can be estimated using a recently developed model (Sethi, 1997; Sethi and Wiesner, 2000) that divides water system costs into major capital cost components. These categories include pipes and valves, membranes, pumps, electrical and instrumentation, tanks, frames, and miscellaneous items (including buildings, electrical supply, treated water storage and pumping, etc.). Total construction cost includes all costs related to construction contract, overhead and profit of the contractor (Kawamura, 2000).

Generally, there are economies of size so as the capacity of the system increases the unit cost of capital declines. Therefore, the per gallon capital cost of water treatment for only Kaw city with capacity of 86,000 gpd may higher would a combined system for Kaw City and Shidler with capacity of 216,000 gpd. Some of existing low-pressure membrane water treatment plants are indeed small, with capacities of less than 3,800 m^3/d (1-mgd). As plant capacity increases, per capital costs typically decrease, due to economies of size associated with manufactured equipment and other facilities. Therefore, for large treatment plants, the annualized capital costs may become similar to the operating costs.

Operation Cost

Operation costs are the variable cost components in the project cost. They include costs incurred in running the day-to-day business or project. For a water treatment plant, operating costs include costs for chemicals, maintenance, energy, taxes, and insurance. Generally, the costs for maintenance, taxes, and insurance are estimated merely as a percentage of the total capital cost. Labor costs are based on the manpower needed and the average salary. The manpower requirements for each design are can be calculated according to EPA documentation (USEPA, 1971).

According to Sethi and Wiesner (2000), operating costs can be systematically calculated for the energy utilized by pumps, for membrane replacement, and for chemicals. Costs related to other components, like concentrate disposal and labor, are highly dependent on factors such as geography, scale, and application of the membrane process. Operation and maintenance costs of water treatment plant normally consist of labor, supervision and administration, power, chemicals, maintenance, repairs, and miscellaneous supplies and services. Other factors that can influence the maintenance and operation cost include the policy of the owners, the complexity of the system, the local environment, and weather. Operating cost can also be increased due to continuing inflationary trends of labor, power, and equipment (Kawamura, 2000).

Distribution Cost

Water is delivered to consumers through transmission pipelines and distribution mains. Trunk lines are the major pipelines that represent the major trunk lines used to deliver water. They connect the treatment plant to the pumping station and to the distribution system. Pumping stations, pipelines, and labor energy comprise the major costs of distribution. The distribution works include the meters, pipelines, and storage facilities (water tanks or reservoir) necessary to convey the water from the transmission system to the consumer (Clark, 1981). As a result, the cost of distribution depends upon the quantity consumed by individuals at various distances from the plant. Clark (1964) noted the energy cost depended upon the flow and distance pumped.

Anticipated Contribution

The result of this portion of the study will assist the Kaw tribe and Kaw City in planning for their water treatment plant and for the distribution of the water to the customers in the city or the area and the cities around it. The results will also give the insight of the power needed to supply certain amount (in gallons) of water a day and a minute (gallons per day or gallons per minute). It will enable city to project the number of water (gallons per day) for future increase in population, cost of equipment like pipes, installation cost and maintenance cost. Moreover, the study will help the city to choose the best (and more cost effective) treatment system.

In addition, the study will help Kaw City and Shidler to solve their long term water problems resulting from poor taste and high amount of minerals in the water, and water shortage in the city (especially during weekends) due to tourism activities in the area.

Data Collection

The data used to estimate costs for distribution of water, the capital and operation costs for water treatment plants, and the pattern of water demand in Kaw City were collected from various sources. The data on costs of water treatment systems were obtained from manufacturers. The costs for pipeline materials and installation were obtained from Means Construction Cost Estimates (RSMeans, 2009).

The data on the layout of the city pipelines including the diameter, the length of the pipe and the materials like fire hydrant collected from drawings provided by the city engineer, which provided an in-depth layout of the existing pipelines in the city and the one connecting Shidler. The treatment plant and monitoring well design are also obtained from the City Engineer through the Department of Environment, Kaw Nation.

The estimated current and projected population of the city were obtained from the website, http://www.census.gov, and Oklahoma Department of Commerce (USCS 2008; ODOC 2008).

Data Details

The study requires detailed information on cost for trenches, pipes, and energy. The study also requires knowledge of the effect of specific variables such as diameter of the pipe, width and depth of the trench, horsepower, distance of the pipeline and overall capital and operating costs. The mains areas of the estimation include the well and pipelines to the treatment plant, the treatment plant, and the distribution system. In each area, costs are divided into fixed (or capital) and operating (or variable) costs.

Estimation of Cost of Water Treatment Systems

Two different sizes of each alternative treatment system will be. For example, one size of the estimated nanofiltration (reverse osmosis) treatment plant system will serve a population of four hundred (400), (Kaw City only); while the other size will serve approximately one thousand people (1,000) (both Kaw City and Shidler). Because of economics of scale, a plant that serves both Kaw city and Shildler is more likely to be economically viable. The cost estimates in this study will be summarized in three main categories: (1) capital cost, (2) operating cost and (3) distribution cost.

Capital Cost

Capital costs are mainly for construction cost and cost of treatment equipment. Once installed these become the fixed component of cost. Capital costs are expected to be incurred mostly at the beginning of the planning process and in future years when the equipment is replaced or renovated. Capital cost can be calculated as the sum of material cost and equipment cost, trench cost, fixed pipe cost and contingency cost. Contingency is a proportion of construction cost estimated as a lump sum cost. The proportion of the contingency depends on the contractor or the estimator of the project but usually ranges from 2% to 5% (Roberts, 2008).

The cost of equipment is a major part of the total capital cost for a water treatment system. The estimation of equipment cost is based on the size, type and quantity of equipment needed to complete the project. The cost of equipment is estimated by multiplying the quantity of equipment by its current price. Some of the equipment can be rented or leased (Roberts, 2008). The materials needed for water treatment include pipes, fire hydrants and others. This category also includes membranes, pumps, pipes and valves, electrical and instrumentation, tanks and frames, and miscellaneous items such as buildings, electrical supply, and treated water storage. Some data are adjusted using the Engineering News Record's Construction Cost Index (ENR CCI) ratio. The ENR CCI value is determined by averaging index values for various equipment. For example, to update a representative cost of 2002 (ENR CCI value $6,538), the cost of 2002 would be multiplied by the ratio of $7,872 over $6,538. The ENR CCI values are based on material and labor construction costs of all major cities across the US. The index measures the amount of money it would cost to purchase a theoretical quantity of services and goods in one year, as opposed to another. The approach of accounts for the individual economies of scale related to different equipment and facilities, and thereby considered an overall economy of scale for the entire membrane system (Sethi, 2000).

Estimation of Pipe Cost

The pipe cost is part of fixed component of cost. Pipe cost is a function of its diameter. Mathematically,

FPC = IP*Dia*MF………… (1)

where FPC = the Fixed Pipe cost, Dia =Diameter of the pipeline and MF = Mortgage factor, IP= Investment Cost of pipe

Trench Cost

Trench cost is the cost of excavating the trench to lay pipes. The trench cost is a function of width and an exponential of depth of the trench. The larger the size of the pipe, greater the width of the trench will be. The depth of the trench varies associated with the size of the diameter of the pipe. T_i is Trench Cost, Di= the depth, D^2_i = square depth and δ_i= the coefficients. The model is:

$$T_i = a + bD_i + ce^D \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(2)}$$

T_i is the cost of trenching

D_i is the depth of the trenching which varies with the cost of pipe.

e is natural logarithms

a, b and c are the parameters of the model and are estimated using regression. Budgets are first constructed based on different trench depths. Then, as equation (2) indicates, regression was used to estimate trenching cost as a function of depth. Since the width of the trench was held constant, it did not have effect on the ordinary least squares (OLS) estimation of the trench cost. R^2 will be calculated to show the goodness of fit of the depth in relation to unit cost of the pipe.

In addition, the total cost of excavation and backfill includes the cost of the trench, packing, and backfill. Trenching cost can expressed as the sum of the cost for backfill, packing, and trench cost.

$$ExB_f = T_i + P_i + B_i + OC_i \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3)}$$

where ExB_f is the Total cost of Excavation and backfill

T_i is the trench cost, P_i the cost of packing on the sides of the pipe in the trench, B_i is cost backfill and OP_i is other cost such as bedding, surveying or blasing..

Estimation of Operation Cost

The operation cost represents the variable part of the cost of the treatment plant, and represents the cost incurred in running the day-to-day activities of the plant. It comprises of chemical cost, energy cost, staff, maintenance, monitoring and labor cost. Total operation cost can be calculated as the sum of the above stated costs. Labor cost can be calculated base on the number of hour per work. It will be estimated base on the current wage of the labor per hour. In estimating operation cost, there general assumptions include:

a. The number of operation hours in a year, usually 8760 hours. (365*24=8760 hours).

b. The unit cost of electricity use during the operations. This has significant effects on the cost of operations. The unit cost electricity consumed in this study is $0.108.

c. The capacity of the well. In this study the capacity is the 150 gpm for Kaw City and Shidler.

d. The unit cost of potassium permanganate use to control odor, and taste in the water is $1.60.

e. The unit cost of chlorine use to kill bacteria in the water is $0.50 per lb.

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Chemical Cost

Chemical costs are the cost for those chemicals used in the water treatment plant. However, this cost depends on the quantity of the chemical use during treatment process and the price of the chemical per pound. When the price per pound of the chemical used for treatment increase, the cost will also increase. In the estimation of chemical cost, there are some baseline assumptions that should be followed:

a). The unit cost of chlorine (in $) should be clearly stated. The unit cost of chlorine is $0.50/lb. This cost will give the cost of the chlorine that will be use in treatment of water base on the quantity of the chlorine use. The chlorine is the most important chemical as far as treatment of water is concern which is use to kill bacteria in water.

b). Another assumption is the cost of the potassium permanganate ($KMnO_4$) use during the treatment. Potassium permanganate ($KMnO_4$) is used primarily to control taste and odors, remove color, control biological growth in treatment plants, and remove iron and manganese.

c). The third assumption to be considered is the unit cost of the scale inhabitor. The unit cost is $1.15lb. The scale inhibitors specifically develop to manage the problems associated with hard water, specifically hardness salts and the formation of scale in a wide range of commercial and industrial process environments.

The chemical cost is estimated base on P_i= price of $KnMnO_4$, (Potassium permanganate), Q_i= quantity of $KnMnO_4$, δi = Scale inhibitor, S_i=cost of inhibitor, α_i=cost of chlorine and C_i is Chlorine. Chemical cost *(C_N)* is calculated as

$$C_N=P*Q + \delta_i * S_i + \alpha_i * C_i \ldots\ldots\ldots\ldots\ldots\ldots \text{(4)}$$

Energy Cost

The Energy cost is the cost of energy needed to run the pumps or treatment plant and other facilities. The energy cost can be estimated with the use of both water horsepower and the Brake Horse power method. In estimating energy cost, the following assumptions should be considered:

a. Pump efficiency should be range from 50-85% efficiency. Pumping efficiency is water horsepower divided by brake horsepower. Mathematically, *Pump efficiency = Whp/Bhp*

b. The efficiency of the electric motor efficiency is also ranging from 80-95%. Motor efficiency is the quotient of Bhp to Mhp where Mhp is Motor horsepower. Algebraically, *Motor efficiency = Bhp/Mhp.* (Spellman, 2000).

Water Horsepower *(Whp=GPM*Head/3960)* is the theoretical power required to pump a given volume of water from a well and through a pipe. The amount of head (pressure) that must be supplied by a pump is equal to the sum of the pumping lift and head loss in the pipeline. The headloss in the pipeline may be calculated by the Hazen-Williams formula as, (Spellman, 2000),

$$\text{Head Loss} = \frac{10.51*(GPM/C)^{2}*Dist}{(Dia)^{4.87}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$

where *Dia* is the diameter of the pipe in inches, *Dist*= distance of the pipe in feet, *C*= coefficient of roughness for type of pipe.

Horsepower (Bhp) is defined as the horsepower supplied to the pump from the motor. It depends on the water horsepower. It can be calculated as

$$Bhp = \frac{GPM*Head\ (pr)}{3960*Peff*Meff} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

where *GPM* is gallon per minute, *Peff* is Pumping efficiency, *Meff* is Motor efficiency and Head (pr) is the pressure flow.

EC is Energy Cost, *GPM* is gallons per minute, *Hd* is head loss, Pe is Pump efficiency, Me is motor efficiency, *KwBhp* is kilowatt per brake horse power, *hpy* is hour per year, and *pelec* electricity cost

$$EC= \frac{\{(GPM*Hd)*Kwbhp*hpy*pelec\}}{3990*Pe*Me} \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Estimation of Cost of Drilling the Well

The necessary depth of drilling a well for Kaw City can be estimated based on a previous monitoring well drilled by CRC & Associates, Inc of Tulsa, OK. The monitoring well is located in the north of Kaw Lake near Washunga Bay. The cost of the drilling of the well is part of construction cost or capital cost. Therefore, the costs of the materials and the equipment which were used in the process of the drilling will be the main focus.

In this estimation, certain features of drilling of the well such as the depth of the hole and diameter (size) of the hole taken into consideration. Previous estimates suggest that typical hole diameter is 8", the length of the hole from the casing to the bottom cap level is 120' and the casing diameter is 4" (CRC & Associates, Inc). Therefore, it is assumed that the length of the pipe (specifically PVC 4") will be 120 feet (120'). To estimate the cost of drilling the well accurately, the quantity of each equipment and material will be multiply by the current prices from the Construction cost data (RSMeans, 2009).

Description and Method of Treatment System

The Aeralater® water treatment process is designed to remove high levels of iron and/or manganese from water. The Aeralator® treatment system is divided into three main sections: aeration, detention and filtration (four filter cells) (Figure 9). The system has been described as three in one system because it performs three functions in a single unit. The type II AERALATER® is considered as a modified conventional treatment system for Kaw City.

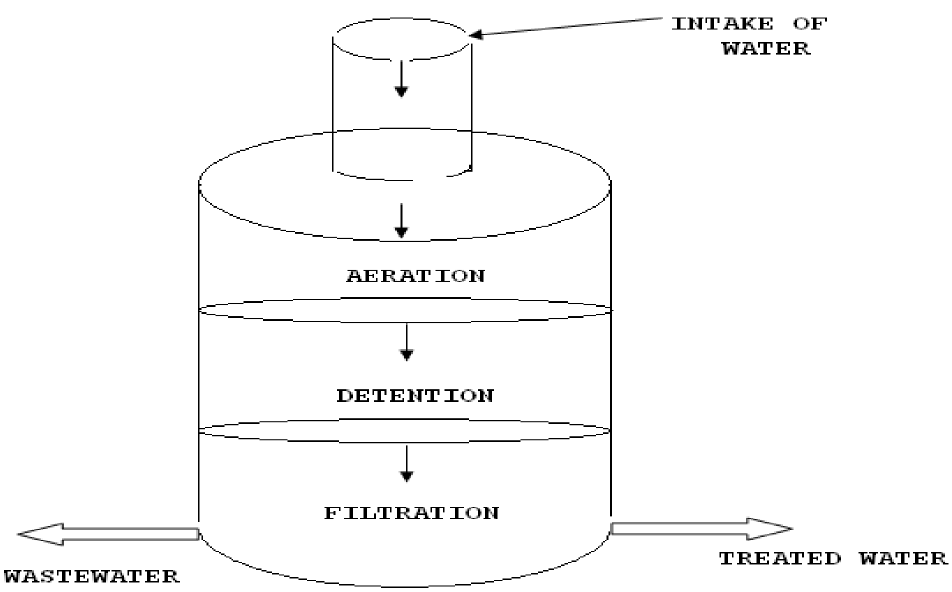

Figure 9. The Flow System of Aeralator ® Treatment Process

The Aeralater® is a complete self-contained filter plant for treating water. It combines aeration, detention, and filtration functions. The treatment processes involves aeration, iron manganese oxidation (with the oxidant added at inlet piping to the Aeralator® system), detention and gravity filtration (with four filter cells). Water from the well (groundwater) enters the top of the Aeralater® and pass through inlet hole (PVC pipe) to the aeration section. After aeration water moves to detention area where oxidation and flocculation of iron and manganese occurs.

The static mixer is mounted in between aeration and detention in order to speedup oxidation process. The probes in the detention tank are used to control the operation of pumps and chemical feeders to control the reaction.

The oxidized iron and manganese water is distributed to the four filter cells through simple piping arrangement. The filtered water later passes through low pressure rate. These filters contain Anthra/sand to remove the manganese., The media is advertized as an alternative to greensand. After the raw water has passed through these processes, multiplates with low headloss are used to collect the filtered water. A similar process is used to automatically backwash the filters and remove the wastewater. The filtered water is then pumped to the elevated storage tank.

Description of Nanofiltration Water Treatment System

As an alternative to the Aeralator®, the nanofiltration system under consideration has a two-stage array system (Figure 10). The system was constructed by Fluid Processes Inc. and the spiral-wound membranes supplied by Hydranautics. The first stage consisted of two parallel pressure vessels, each consist of three membrane elements.

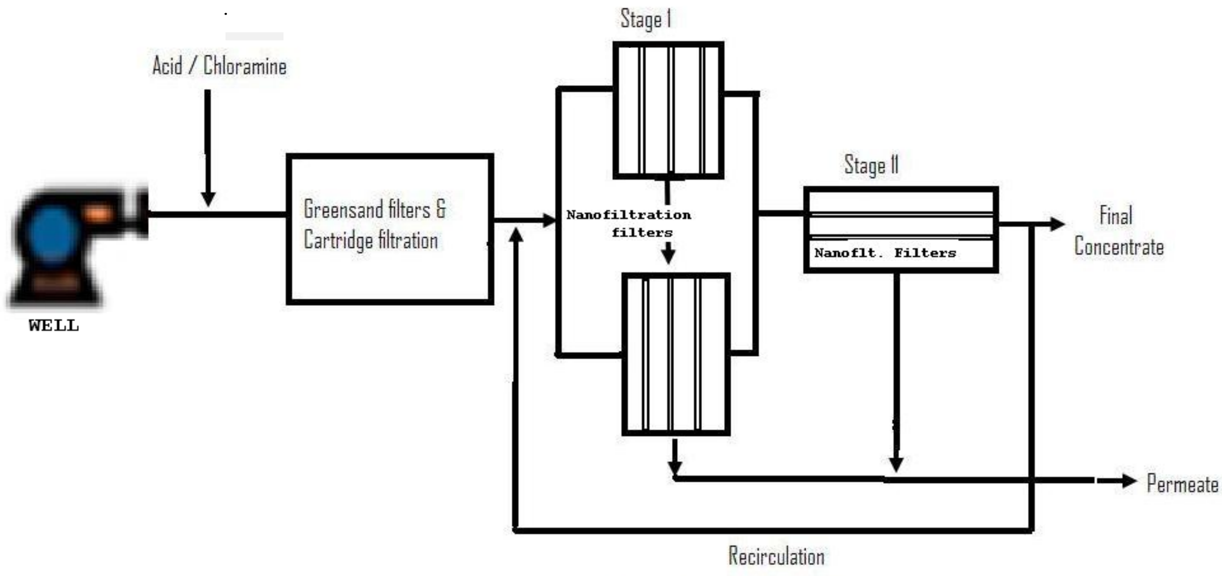

Figure 10. Nanofiltration Treatment Design Process

The second stage consisted of one pressure vessel containing three membrane elements (Hem, 2008). The system was assumed to run at 75 percent recovery. This means that 75 percent of the intake water enters the distribution system while 25 percent enters the wastewater system. Before the nanofiltration, the water would be filtered through a cartridge filter or greensand filters, to oxides manganese and to prevent the plugging of the membrane module with particles. Acid will be introduced into the nanofiltration feed line to keep the pH between 5.6 and 5.8 to enable solubility of carbonates to minimize inorganic scaling. Chloramines would be injected at a set rate and concentration to prevent biofouling (formation of a biological slime or biofilm that can be avoided by feeding chlorine into the feed water). Because the nanofilter membranes do not tolerate free chlorine, chloramines would be used. Chloramines are defined as chlorine that exists in a chemical combination with ammonia in water. Chloramines were made by mixing sodium hypochlorite with ammonium sulfate. Chloramines controlled such that no more than 0.1 mg/l of free chlorine applied to the membranes. The goal residual in the permeate stream will be one mg/l of chloramines.

Creating of pipeline distance and elevations using EPANET Software model

EPANET software was used to create a digital pipeline map for Kaw City. A modified version EPANET called EPANET-Z (Zonium Solutions) was used which has Google and Yahoo maps as the background. Parameters such as length of the pipeline, elevation of the nodes and equipment like pumps added into the model of the distribution system. The distribution system of the Kaw City receives its water from the existing city water tower (Figure 12).

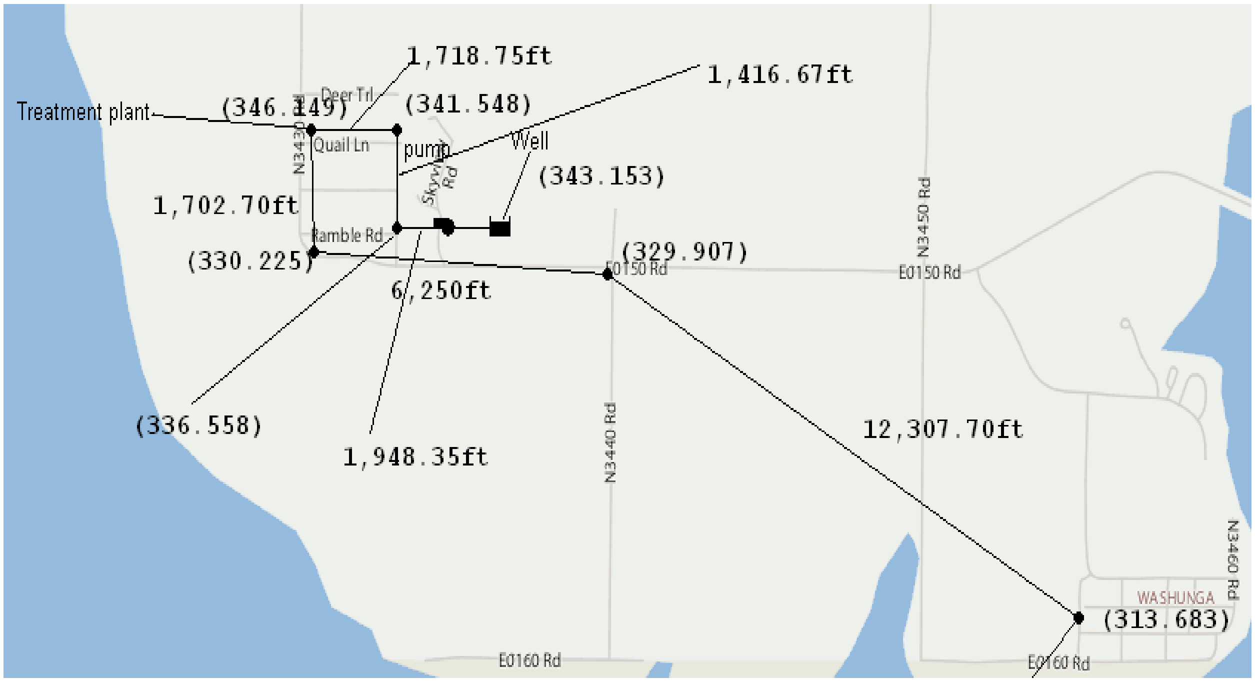

Figure 11: Kaw City Network System with Elevations

In EPANET–Z's toolbar, the pipe and link icons used to create link and endpoint (junction) of the pipe. Precisely, the node formed the endpoint of the pipeline and the link formed the pipeline (Figure 12).

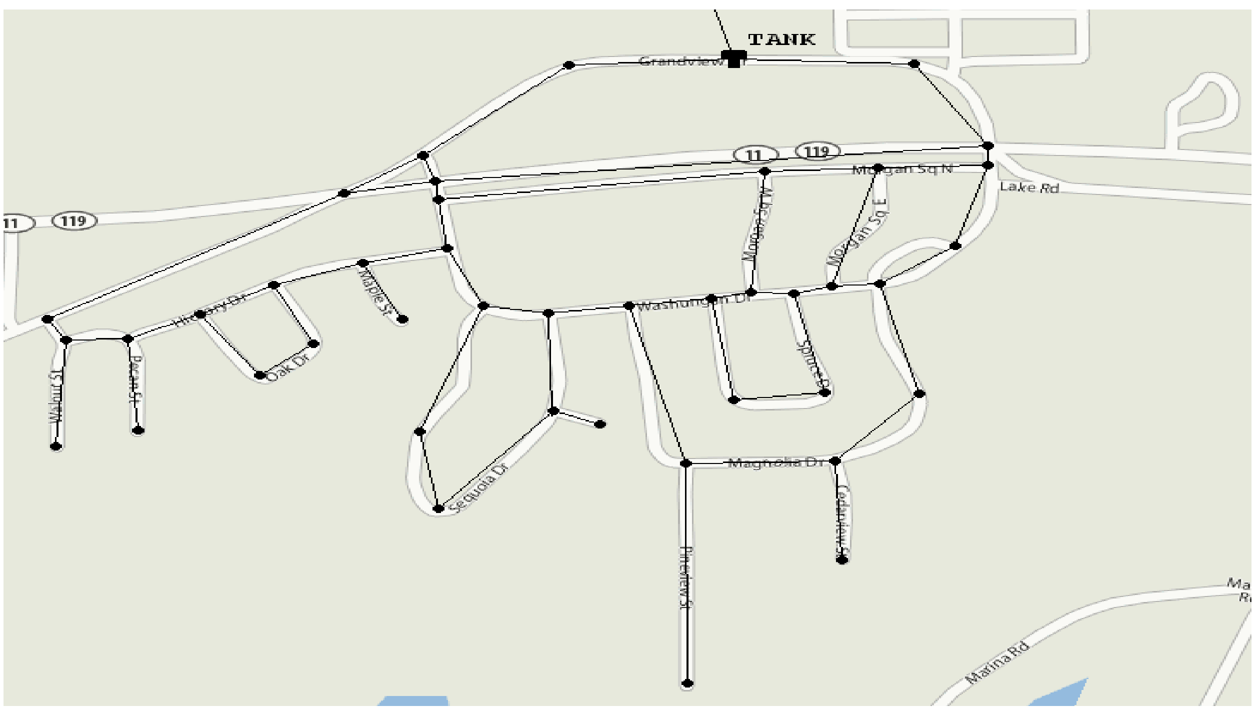

Figure 12: Kaw City's Pipeline Layout

The main tower (tank) and the pump are located in the model in addition to the pipelines and the nodes. Then EPANET-Z will save the data in an *.inp file by exporting the network (pipeline layout).

The elevation of each node was estimated by overlaying the pipeline file on a USGS 1/3 second elevation map in the GIS software program Global Mapper©. An xyz file is exported from Globalmapper. The elevations from this file are added to the node identification section of the EPANET input (.inp) file using a text editor such as WordPad or Notepad.

The elevations values were in meters but were converted to feet. In GlobalMapper, the measure icon can be used to calculate the distance (length) of the pipelines. This procedure was used repeatedly until all the measurements finished. In areas where there were large elevation changes between nodes, it was necessary to use Pythagoras's theorem to estimate the length of the pipeline between nodes. Alternatively, a tread measurement method can be applied. GlobalMapper was used to create a cross section from one node to another. The tread was used to measure the undulating cross section and multiple by the scale to get the exact distance.

For detailed estimates on the cost of constructing and operating various versions of the two systems (modified conventional and reverse osmosis), see Atta-Asiamah (2010). This includes estimates of smaller (Kaw City only - 60 gpm) and larger (Kaw City plus Shidler - 150 gpm) systems. Shown below in Table 7 is a comparison for the two systems for supplying Kaw City only (60 gpm). Because of the location of the treatment plant relative to the Kaw City, a large part of the cost is for the necessary pipe to connect well, treatment, and Kaw City. The cost of the modified conventional system is estimated to be about $3.00 per 1,000 gallons while the cost of the reverse osmosis system is about $4.00 per 1,000 gallons.

Table 7. Comparison of Capital and Annual Costs for Modified Conventional and Reverse Osmosis Plants to Supply 86,000 GPD to Kaw City (60 gpm)

Item	Unit	Modified Conventional				Reverse Osmosis			
Capital Cost		Years Life	Units	Unit Price	Initial Cost	Years Life	Units	Unit Price	Initial Cost
Well	depth	50	120	na	$ 4,020	50	120	na	$ 4,020
Pump and motor	gpm	3	60	na	$ 8,400	3	80		$ 9,080
Pipe, trench to WTP, 4" dia.	ft	50	5082	$17.5	$ 88,935	50	5082	$17.5	$ 89,087
Water Treatment Plant (wtp)	sqft	50	750	$125	$ 93,750	50	1000	$120	$ 120,000
Treatment equipment	tgpd	20	86		$ 81,000	20	86		$ 209,000
Pump: WTP to W.Tower	hp	10	6	na	$ 4,000	10	6	na	$ 4,000
Pipe,trench: WTP to W.Tower	ft	50	20262	$17.5	$ 354,585	50	20262	$17.5	$ 354,990
Engineering Cost	10% Cost	50	na	na	$ 4,000	50			$ 79,018
Total Initial Cost					$ 638,690				$ 869,195
Annual Operating Cost									
Pumping Well to WTP	Kwh		32.8	$10	$ 3,285		48	$10	$ 4,842
Energy within WTP	Kwh		46	$10	$ 4,599		100.3	$10	$ 10,356
Pumping: WTP to WT	Kwh		33.9	$10	$ 3,393			$10	$ 3,393
Chemical and supplies	cost/yr		-	-	$ 4,201		-	-	$ 4,621
Labor	hrs		730	$35	$ 25,550		913	$35	$ 31,938
Maintenance	1.5% Investment				$ 9,580				$ 13,038
Office expense	Cost/mo		12	$300	$ 3,600		12	$300	$ 3,600
Total Variable Cost					$ 54,208				$ 71,787
Annual Capital Cost					$ 39,971				$ 56,070
Total Annual Cost					$ 94,180				$ 127,857
Average Cost per 1000 gallons					$ 2.99				$ 4.05

Section 3: Principal Findings and Significance

The primary findings from the first three projects (Beggs, Oilton, and Braggs) suggest that it is possible to develop GIS-based water system simulations for small towns and rural communities at reasonable cost. This can be accomplished with a combination of public domain software, relatively low cost web-based systems such as Google Earth® , GIS software, and macro driven spreadsheets. Examples of the data requirements and steps necessary to run these simulations can be found in Lea (2009), Bhadbhade (2009), and Senyondo (2009).

The EPANET freeware program is capable of providing useful simulations of piping layouts, pumping demands, spatial analysis of water pressures and water ages in pipeline systems, and calculating operational costs (electricity for pumps) for small towns and rural areas. This software developed by EPA is free and reasonably sophisticated. Base systems can be developed and initially calibrated with minimal effort from the communities involved. The models can then be further refined and used to address specific water system planning needs such as excessive water ages, high pumping cost, low and high-pressure zones, and fire fighting capacities.

The most time consuming process is the development and validation of the current water supply system. The problems and their associated solutions differ between small towns and rural water districts. The findings or methods developed for rural water districts are reviewed first, followed by a discussion of small towns.

<u>Rural Water Districts</u>. In Oklahoma, the Oklahoma Water Resources Board (OWRB) has developed GIS files of pipelines for rural water districts. Supporting files provide information (generally from the year 1995) on the source of water, type of treatment, number of people served, number of meters, average use, and peak use. The GIS files contain estimates of pipeline location, length and diameter. The files do not contain elevation levels of system elements. The ORWB files show individual pipelines along with the location of their beginning and ending nodes. However, the pipes are not connected in a system that allows modeling using commercial software. Other problems include the presence of numerous duplicate pipes. These problems are solvable. Steps to fill these data gaps and allow modeling of the systems are outlined below.

1. The estimation of elevation at end nodes for individual pipes is accomplished by overlaying the pipelines on USGS elevation data sets. GIS software is used to overlay the pipeline map on a USGS 1/3 arc second elevation map and add the elevations to the nodes. Critical elevation points along the pipeline can be verified with GPS units when site visits are made.
2. Spreadsheet macros are developed to eliminate duplicate pipes and to join pipes at the appropriate nodes. The process of joining two pipes at a common node consists of replacing the node identification on one of the pipes with the identification of the joining pipe, so that both pipes have the same ending node. The process of joining pipes in the middle (creating a “T”) is accomplished by dividing the initial pipe into two shorter ones, and adding the identification of the ending node of the second pipe to the newly created nodes on the pipe which was just divided. This process creates one new pipe whose identification code (along with the identification of its nodes) must be added to the original list of pipes.
3. Initial estimates of rural water demands tied to specific spatial locations are accomplished by overlaying the pipeline maps on annual NRCS one-meter aerial photo files. Census blocks are generally too large geographically to be of use in locating the position of rural households. The initial estimates are used to develop an operating model that will be later revised through site visits, discussions with RWD personnel, and ground-truthing maps. Field GPS units can also be used in this step.
4. An initial analysis of the system under average and peak flow conditions for the current period is modeled, as well as an analysis, without additional major infrastructure additions, for the 2050-2060 time period.
5. Points of high and low pressure, points of constriction along pipelines, problems of pump and water tower cycling, water age in pipes (particularly dead ends) and unacceptable head losses are noted in both evaluations (current and year 2050).
6. From the problem list prepared in step 5, a priority list of problems is developed. Multiple (at least two) specific system changes (such as pipeline replacement, additional pumps, additional above-ground storage) are then modeled and cost data developed based on the required infrastructure changes.

7. The results of the modeling and priority list of infrastructure improvements are presented to the water district personnel.

Small Towns**.** Many small towns lack accurate water system maps and records. Between personnel limitations and non-availability of funding, the system managers cannot focus on long-term problems. Since the OWRB does not provide maps of small town systems, a different set of procedures is used to model and evaluate small towns.

1. Water managers or city engineers are contacted to determine the approximate locations and diameters of pipelines serving the city. Thus, the first step is to develop GIS-based pipeline maps. This is done using the freeware program EPANET-Z developed by Zonum Solutions®. This program allows the user to develop a pipeline map of a town using a street grid map obtained from Google Earth. The pipeline diameters must be provided by local officials. It is necessary to check the pipeline lengths using known measurements of square miles or measured highway miles to verify the distances assigned to the pipelines by the software.
2. Census block data from the 2000 census, along with the pipe line map developed in step 1, are used to determine the residential population served at each of the nodes on the pipe network.

The remaining steps follow the same procedure as for rural water districts, steps 4-7.

Section 3.1: Final Project Conclusions

The project was successful in constructing a methodology to evaluate rural water system infrastructure. The incorporation of different water sources, infrastructure issues, and modeling software indicates that several approaches can be taken to effectively help rural water systems plan and update their water supply infrastructure. The development of a cost estimating methodology was also an essential part of the project, since understanding the costs associated with different upgrades is important for the community to understand. Highlights of the project results include:

- Small systems have common problems of low demand and long, low-velocity lines, which result in high water age and low disinfectant residual.
- The common remedy for high water age, which is to loop the pipes, does not always work for small systems, due to very low demand. A loop will add even more length to an already excessively-long system.
- Elevation differences mean that some areas have high pressures while others have very low (sometimes unacceptable) pressures.
- Technical expertise and experience necessary to use either EPANET or WaterCAD are beyond the staffing capabilities of small systems. It took several months for engineering graduate students to become familiar with the software.
- Small communities need assistance in writing grants to get funding for system improvements. Just getting a grant written is beyond the capability of most system staff members.

To this last point, each of the communities participating in the project expressed anxiety about paying for the upgrades suggested by the simulations. Discussions with OWRB personnel indicate that significant effort has already taken place to educate rural water district personnel about requirements for applying for funding, including a multitude of fact sheets and even a yearly full-day conference sponsored by the Funding Agency Coordinating Team (advertised as "one stop shopping to find the financing you need for your project" (Oklahoma Rural Water Association, 2009)). Our experience suggests the promotion of this type of event is crucial, as is the technical help provided by "Circuit Riders" who travel to small water systems and provide educational sessions for system personnel. Finally, the need for professional engineering help indicates that an extension program (provided by any land-grant university) focused on this area would be in high demand, particularly for states with many rural water systems. Funding a full-time engineer to deal with projects such as those explored in this paper would provide significant benefit for the rural water systems assisted and would likely result in extremely positive publicity for the departments involved.

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Appendix A: Presentation to Southern Agricultural Economics Association February 2010

Decision Support System for Evaluating Rural Water Supply Infrastructure Scenarios

Primary Investigators:

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EXTENSION

Prepared for SAEA Meetings

February 9, 2010

Introduction / Statement of Need

- Rural water systems have difficulty making decisions about the future of their infrastructure
  - Should they consolidate with other systems?
  - What needs upgrading? Pipe, pump, tower,…
  - How much will it cost?
  - What funding sources are available?
- This project develops a process that assists towns in assessing and managing their water infrastructure needs
  - Including list of priority improvements

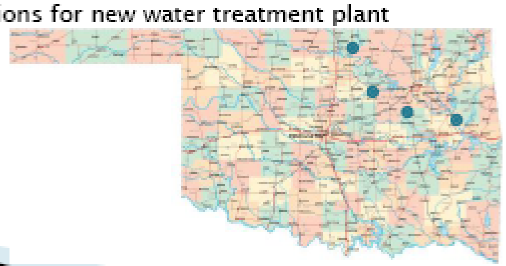
Project Description

- Steps in Decision Support Model
  1) Understand current system layout
  2) Obtain data for modeling infrastructure and possible upgrades (census projections, terrain maps)
  3) Analyze existing distribution system using free software (EPANET) or professional software
  4) Assess problem areas & explore solutions
  5) Estimate procurement and maintenance costs of various options
  6) Summarize available funding options and requirements

Test Systems

City	Population	Water Type
Beggs, OK	1,400	Surface
Braggs, OK	300	Ground
Oilton, OK	1,100	Ground
Kaw City, OK	400	Surface

- Kaw City: Also studying options for new water treatment plant



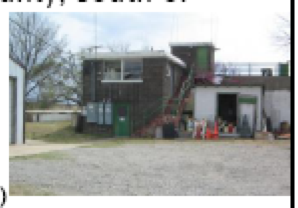
Common Problems

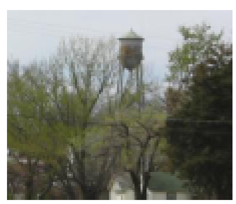
- Lack of funding and personnel
- Aging infrastructure
- Low pressure areas
- Water age in pipes
- Fire protection issues
- Old and/or incomplete pipe line and facilities maps

Case Study: Beggs, OK

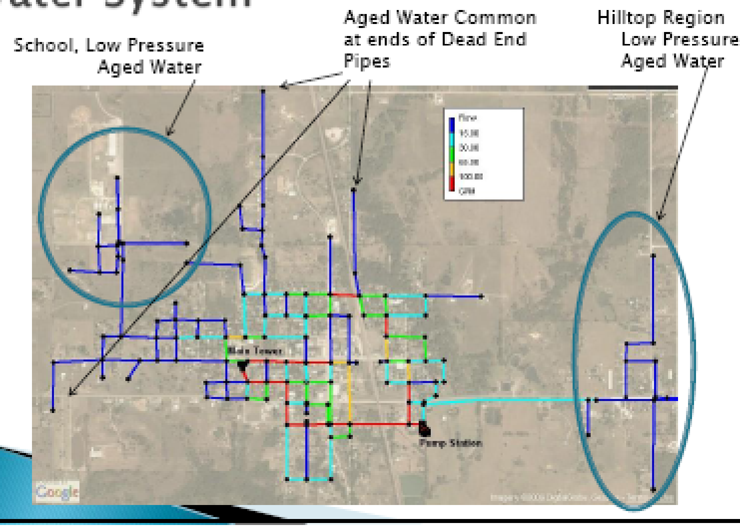
Location and Stats

- ~ 4.3 square miles, Okmulgee County, south of Tulsa
- Population of 1364
- Water supplied from Beggs Lake
- 1 Pump station (very low elevation)
- 2 Water storage tanks
  - 50,000 gal elevated (highest elevation)
  - 150,000 gal underground
- Pipelines
  - Majority are cast iron and 45+ years old
  - Minority are newer PVC
- 6” mains serve 2”-4” branches



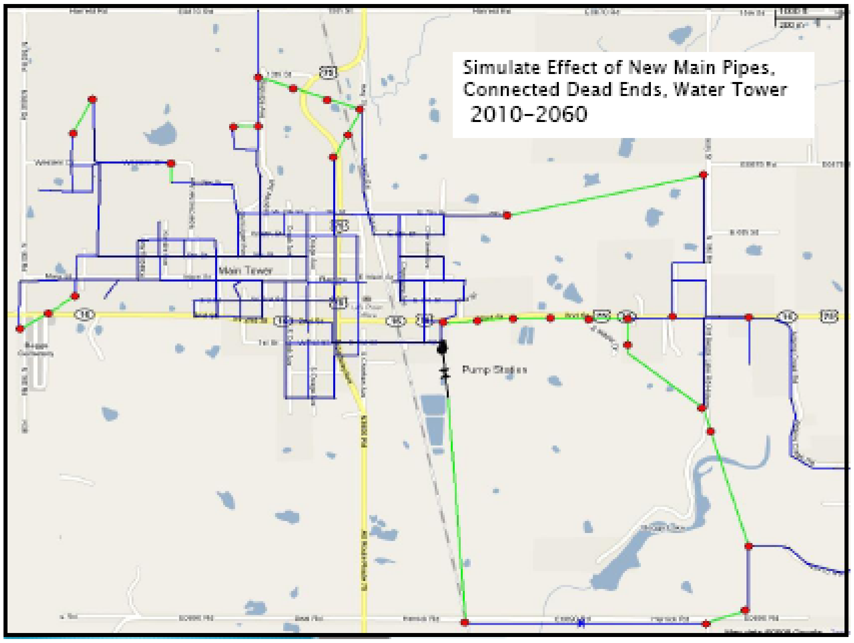


Digitized Pipeline Model of Beggs Water System



Use Model for Present and Future Scenarios

- Deficiencies within the current system
  - Hilltop region and dead ends fail fireflow tests badly
  - School area is also a concern because of its value and somewhat low pressures
- Possible Solutions
  - New or modified pumps
  - New or modified water tower
  - Replacing old pipes (that have inside deposits)
  - New pipes to eliminate dead ends or create new water paths

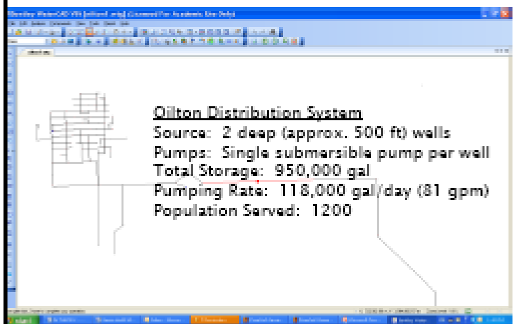


Tested different combinations and timing of improvements
Suggested order for improvements

1. Cost: $69,000; Two major pipes in rural East Beggs
2. Cost: $60,000; Three pipes in rural East Beggs to finish addressing the Hilltop pressure problems
3. Cost: $57,000; Then add all the remaining pipes to target selected dead ends
4. Cost: $60,500; Required fire hydrants & pipe fittings
5. (Cost: $167,000; New 50,000 gal water tower)

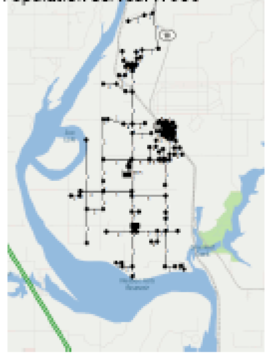
TOTAL = $415,000

Similar Analysis Performed in Braggs and Oilton



Oilton Distribution System
Source: 2 deep (approx. 500 ft) wells
Pumps: Single submersible pump per well
Total Storage: 950,000 gal
Pumping Rate: 118,000 gal/day (81 gpm)
Population Served: 1200

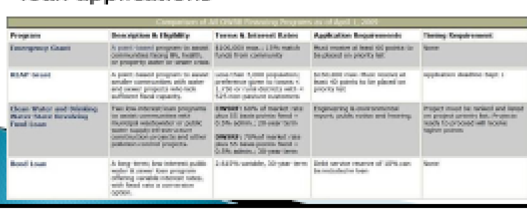
Braggs Distribution System
Source: Artesian Wells
Pumps: 3 in parallel
Total Storage: 200,000 gal
Pumping Rate: 75,600 gal/day (52.5 gpm)
Service Connections: 416
Population Served: 1030



Funding Options

- Once infrastructure improvements have been suggested, most communities ask, "How can we pay for this?"
- OWRB is already providing good resources
  - Financial Assistance Program
  - Loan and Grant Resource Guide
  - Full-day conference by Funding Agency Coordinating Team (FACT) ~250 attendees
- Our role: summarize information into easy-to-use document, provide workable examples of grant / loan applications

FUNDING AGENCY COORDINATING TEAM
FACT
2009 FACT Funding Conference



Conclusions

- Small systems have common problems of low demand and long, low-velocity lines, which result in high water age and low disinfectant residual.
- The common remedy for high water age, which is to loop the pipes, does not always work for small systems, due to very low demand. A loop will add even more length to an already excessively-long system.
- Elevation differences mean that some areas have high pressures while others have very low (sometimes unacceptable) pressures.

The Road Ahead

- Tool is not directly usable by most rural water systems (too complex)
- We believe Extension can play a role
  - Example: Rural hospital work
  - 1 full-time, dedicated employee handles 6-7 rural hospital assessments per year
  - Funded externally via HRSA grant
- Future funding?
- OSU Civil Engineering Class Project?