WATER SYSTEM MASTER PLAN
City of Enid, OK

Prepared for:

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EXECUTIVE SUMMARY

BACKGROUND

In August 2006 the City of Enid (City) retained C.H.Guernsey & Company (GUERNSEY) to perform an evaluation of the City's water system to identify shortcomings and capital improvements necessary for the City's continued growth. GUERNSEY evaluated all facets of the water supply system including well fields, collection system, water treatment, distribution pumping, distribution network, and water rights. The evaluations were carried out with the cooperation of City staff familiar with the water supply system. Recommendations were developed to meet the projected growth of the City through to 2050.

A draft report detailing the evaluations and recommendations for capital improvements was presented to the City in October 2007. Following a series of meeting to discuss the report, comments received from City staff were incorporated into a revised report dated February 2008.

In April 2008, the City retained GUERNSEY to undertake a fire flow analysis of the City's water system to identify shortcomings and capital improvements necessary for the adequate fire protection of properties in the City having the greatest Needed Fire Flow requirements. Following a series of meetings to discuss the findings of the analysis, comments received from City staff regarding the revised February 2008 report and the fire flow analysis have been incorporated into this final report.

STUDY ACTIVITIES

The following tasks were undertaken as part of the original study:

Task 1: Pre-Planning/Kick-off Meeting/Site Reconnaissance
Task 2: Review/Summarize Existing Data
Task 3: Water Supply Evaluation
Task 4: Water System Demand
Task 5: Collection System Evaluation
Task 6 Distribution Pumping System Evaluation
Task 7 Water Treatment Evaluation
Task 8 Distribution System Evaluation
Task 9 Elevated Storage Evaluation
Task 10 Wellhead Protection Plan Evaluation
Task 11 Public Meetings/Citizens Group
Task 12 Water Rights Analysis
Task 13 Water Rate Analysis and Cost of Service Study
Task 14 Final Report

The following tasks were undertaken as part of the fire flow analysis:

Task 1: Location of Properties
Task 2: Hydrant Selection
Task 3: Hydrant Modeling

Task 4: Fire Flow Modeling Report (incorporated into this report)

FINDINGS

The City population is projected to grow from 47,045 in 2000, to 58,803 by 2050. Based on providing water to the whole of Garfield County (projected 2050 population of 65,625) and certain assumptions of industrial growth, average day water demand is projected to increase from 9.59 million gallons per day (MGD) in 2006, to 18.45 MGD by 2050. Peak day demand is projected to increase from 17.06 MGD to 33.39 MGD over the same period. The derived demand figures are slightly conservative to ensure that planning infrastructure for future growth is not undersized.

Currently, water is being abstracted from the City wellfields in excess of the historic natural recharge rate of the aquifers i.e. water is being 'mined' from the aquifers. The result of this mining is that water levels in the aquifer will continue to drop over the long term, although short term fluctuations will occur in response to periods of wet weather. If all of the projected increase in water demand is met by increasing production in the existing wellfields, all of the wellfields are projected to be pumped dry prior to 2050 if the historic trend continues. If water abstraction from the existing wellfields was limited to the recharge rate, pumping from aquifers would be sustainable i.e. it can be maintained indefinitely. However, sustainable pumping would result in the City having a projected average day water deficit of 12 MGD by 2050.

To supply the additional 12 MGD, the City must develop an alternative water supply source to supplement the existing wellfields. Should the City decide to expand the wellfields by acquiring additional water rights, the City would require approximately 61,000 acres (or 95 square miles) of new water rights to enable both the existing and new wellfields to be pumped at a sustainable rate without mining occurring. Other alternative water supplies include pumping water from Kaw Lake, or constructing a dam close to the City on Turkey Creek or Skeleton Creek. It is recommended that further studies be undertaken to identify the water source best suited to meet the City's needs.

The percentage of current total water production supplied by the Cleo Springs and Ringwood wellfields is far greater than the percentage of total usable water stored in those wellfields. In contrast, the Drummond wellfield is under utilized when compared to the amount of usable water stored in the wellfield. It is recommended that production be switched from these two wellfields to Drummond such that the production in each wellfield is more in line with its percentage of usable water stored. This will ensure that the production life of the Cleo Springs and Ringwood wellfields are maximized.

The condition of the existing wellfield infrastructure varies, as the wellfields have been developed over a period of time. The Cleo Springs and Ringwood well houses and plants are in good condition, but the Ames, Drummond and Enid well houses are all due for replacement. The City has commenced replacing these well houses at a rate of three per year, but it is recommended that the rate of replacement be increased. Additional recommended infrastructure improvements include installing an emergency backup system to the Ames plant (to enable water to be pumped during a power outage), and replacing the Ames and

Drummond wellfields transmission line. This line is ageing and can only operate at reduced pressures and flows.

A review of the existing water rights and permits has revealed a discrepancy between the records held by the City and those held by the Oklahoma Water Resources Board (OWRB). It is recommended that water rights should be compared with county records to confirm their accuracy and to locate any additional records not held by the City. In addition, the City should confer with OWRB to correct any water permit errors.

Regarding the City's two water treatment plants, it is recommended that the chemical treatment building at Plant #1 be replaced such that the injection points on the raw water line can easily be accessed and not be contained within an underground vault. Due to the over pumping of the aquifers, nitrate concentrations are increasing in the raw water to an extent that nitrate removal treatment may be required at both plants in the future in order to meet drinking water standards. It is recommended at this stage to continue monitoring nitrate concentrations, and to install nitrate removal treatment at both plants when appropriate.

The distribution pumps at both plants will need to be replaced in the future to meet the increase in water demand. It is recommended that a phased replacement program be introduced to increase pumping capacity. The operating pressure in the Plant #2 pressure zone will need to be increased by 2050. This can be achieved by filling the existing elevated water storage tank to full depth.

The current storage capacity in the distribution system is adequate for fire protection requirements in 2050. Although not required for fire protection, the City has decided to install a 0.75 MG elevated storage tower in the north east part of the City as a means of addressing occasional pressure problems in the Plant #1 pressure zone. To improve fire flows and pressure in the southern part of the City, it was recommended that a standpipe be installed close to Vance AFB, and that the Gray Ridge Booster Station be upgraded to feed the standpipe. As an acceptable alternative to a standpipe, the City has decided to install a 0.75 MG elevated storage tower at the Gray Ridge Booster Station as a means of addressing occasional pressure problems in the Plant #1 pressure zone. To address low pressure problems in the distribution system on the suction side of the booster station (which would only be exacerbated by the station upgrade), it is recommended that the capacity of the main water line on Van Buren that feeds the booster station be increased by installing a parallel water line.

The distribution system will require additional, phased reinforcement to relieve pressure and capacity problems as the demand for water grows. Recommended improvements include:

- Install a 30-inch to 18-inch water line from Plant #1 north to US 81/Phillips intersection, to improve service in the northern part of the City.
- Phased installment of a 24-inch water line west from Van Buren to 30th Street, to meet the growth in industrial demand.
- Phased installment of a 36-inch to 12-inch water line from Plant #2 north towards the existing elevated water storage tower, to increase capacity to the tower.

As the City expands into areas currently not served by a water line, it is recommended that the City's practice of installing 16-inch water mains on section line roads be continued. Modeling indicates that 12-inch water mains on section line roads in identified growth areas would be adequate.

A fire flow analysis was undertaken to determine if the current water distribution system could adequately supply fire flows to 25 critical areas of the City that contain 30 properties with the largest fire flow needs. The analysis was based upon a fire occurring on a future peak day (with a demand of 31 MGD) to determine if the phased distribution improvements detailed above were adequate to meet the fire flow requirements. It was determined that approximately 60% of the areas required additional distribution improvements above those identified to meet future pressure and demand requirements. Most of the identified improvements are small scale projects requiring the installation of relatively short lengths of small diameter (6-inch and 8-inch) mains to improve flows to a particular hydrant. One exception is that to improve fire flows to the Autry Vo-Tech would require the installation of large diameter (12-inch and 18-inch) mains along Cleveland Street and Willow Road from the 24-inch main in Chestnut Avenue.

Based upon the findings of the financial analysis, the existing water system operations are underfunded from the existing rate system. The estimates for yearly increases in operational costs cannot be accommodated by the estimated growth in the system's water usage. To make up the shortfall, yearly increases in both the base and incremental water rates are required beyond the 2010 increment adjustment. Estimated increases vary from 2.25% per year to 3.50% per year. When the capital improvement projects are considered, the rate increases will be higher.

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1.0 INTRODUCTION

1.1 BACKGROUND

The City of Enid (City) currently supplies potable water to its citizens via groundwater pumped from the Cimarron Terrace, Enid Isolated Terrace and Cedar Hills aquifers. The Cimarron Terrace and Cedar Hills aquifers are located to the west and southwest of the City and the Enid Isolated Terrace aquifer is located within the City's boundaries and northwest of the City. Concerns over long-term capacity and sustainability of the current well fields, treatment processes, and distribution system have prompted the City to investigate the water supply system and develop a Water System Master Plan (Master Plan).

1.2 GOALS AND OBJECTIVES

The purpose of this Master Plan is to evaluate the City's water supply system to identify shortcomings and capital improvements necessary for the City's continued growth. GUERNSEY evaluated all facets of the water supply system including well fields, collection system, water treatment procedures, distribution pumping system, distribution network, and water rights. GUERNSEY worked closely with the City in the development of this study to ensure the City's needs were met and the public was well informed.

1.3 PROJECT SCOPE AND REQUIREMENTS

GUERNSEY in coordination with City personnel developed a Scope of Work for the Master Plan that involved 14 distinct tasks. The requirements involved in each of the tasks are described in greater detail in the following sections.

Task 1: Pre-Planning/Kick-off Meeting/Site Reconnaissance

The kick-off meeting was intended to bring together the City's staff and GUERNSEY's Team to solidify project goals and for use as a data transfer session. A communication network was identified and project milestones addressed. Historical information contained within the City's archives were copied and added to the project informational database. The Team also visited pertinent facilities and sites that were relevant to the City's water system including a comprehensive review and observation of the wellfields.

Task 2: Review/Summarize Existing Data

GUERNSEY reviewed background data to familiarize team members with historical reports (1982 Benham Study, the 2005 Comprehensive Plan, Various Envirotech reports, etc.) and physical site data (United States Geological Survey [USGS] maps, aerials, etc.). GUERNSEY used this time to investigate preliminary issues associated with the aquifers. The goal of this task was to educate team members regarding historical information to use as a basis to develop the new Master Plan.

Task 3: Water Supply Evaluation

The most important part of Enid's water system is the groundwater contained in the Cimarron Terrace Aquifer. The GUERNSEY Team reviewed historical archives maintained by the Oklahoma Water Resource Board (OWRB), USGS, and the City to evaluate the impact of the City's withdrawal on the long-term sustainability of the aquifer. Possible regulatory changes in the aquifers management could also impact long term groundwater usage. GUERNSEY met with the OWRB to gain insight into possible regulatory changes. GUERNSEY evaluated drawdown records, aquifer conductivity, and historical trends to assist Enid in projecting future water availability. The need for other options, such as additional groundwater development and/or surface water resources such as Kaw Lake, was also examined.

Task 4: Water System Demand

Dependable long-term water supply is essentially a balance between supply and demand. GUERNSEY, in collaboration with City staff, Enid citizens, and the latest comprehensive plan worked together to develop estimates of future water demands through 2050. Future water demands are heavily dependant upon both projected population growth, as well as industrial development (i.e., Advance Foods, Koch Fertilizer and ethanol plants). GUERNSEY worked closely with the Enid City Council and Metropolitan Area Planning Commission to develop reliable estimates of future industrial expansion. By evaluating supply on a holistic approach the GUERNSEY team was able to identify future system shortcomings and provide recommendations necessary for potential growth.

Task 5: Collection System Evaluation

To evaluate the City's current groundwater extraction infrastructure, the GUERNSEY Team evaluated each of the five groundwater wellfields including each of the 150+ existing wells, Supervisory Control and Data Acquisition (SCADA) system, collection piping, and sumps. The capacity and condition of the equipment was evaluated and recommendations for improvements were made as necessary. In addition to a physical inspection, GUERNSEY interviewed City staff to determine areas of operational difficulties both with physical apparatus and with electronic communication systems such as the existing SCADA system. The goal of this task was to evaluate the existing collection system and identify future requirements based upon current maintenance necessities as well as future demands.

Task 6 Distribution Pumping System Evaluation

In Enid's current water supply system, groundwater is collected from wellfields and transported to one of two distribution pumping stations where it is treated for public usage. A series of pumps lift treated water from large clear wells and distributes it throughout the City. Because the City currently only has one elevated water storage tank, the distribution pumps must run constantly for the system to function correctly. GUERNSEY evaluated the pumping capacity, storage capacity, SCADA communications, and associated equipment in the distribution pumping stations. GUERNSEY evaluated existing system parameters and made recommendations so the City is prepared to meet future demands.

Task 7 Water Treatment Evaluation

Currently the City's water treatment program consists of chlorination and fluorination of groundwater prior to storage in one of three clear wells. Groundwater quality historically has not been a major concern for the Enid water system; however, the impact of future regulatory requirements is unknown. Historically, drinking water regulations impose stricter treatment standards as time passes. Recent changes in microorganism and arsenic concentrations are an example of more stringent requirements.

GUERNSEY collected data on existing chlorine and fluorine usage and concentrations throughout the system to identify those areas requiring attention. Future demands developed from Task 4 were compared with existing treatment capacities and recommendations have been prepared accordingly.

Another issue is potential nitrate contamination. The City currently manages the nitrate levels by blending water from high nitrate wells with water from low nitrate wells. In the future this technique may not be adequate and potential treatment recommendations have been included.

Task 8 Distribution System Evaluation

As with many similar cities in Oklahoma, Enid's water distribution system was constructed decades ago under different demands than what currently exist. As the City has continued to grow, the distribution system expands to meet demands. Currently the City is experiencing significant industrial growth/water demand that must be met in the short term. The City must also maintain adequate water delivery capabilities to meet future growth.

The Master Plan includes a Distribution System Improvements Plan that will assist the City in identifying future water main extensions. GUERNSEY estimated future growth trends from sources such as the 2005 Comprehensive Plan, Enid Chamber of Commerce, Metropolitan Area Planning Commission, and input from local residents via public meetings. GUERNSEY identified areas that are growing or are anticipated to grow in the future that will require water system upgrades.

The GUERNSEY Team evaluated the water distribution system from a current demand perspective, but also from a future supply/demand opportunity as well. GUERNSEY evaluated the existing water distribution model and made updates as necessary. A distribution system contour map was prepared to illustrate average and maximum pressures for the system. The capacity of the existing network was compared to existing and future demands to identify areas of expansion and possible deficiencies. GUERNSEY understands the City has taken a proactive stance on maintenance of the distribution system and currently has budgeted funds for replacement or upgrades of system components on an annual basis. GUERNSEY engineers assisted the City in the identification of phased improvements that will optimize capital expenditures and prepare the City for future demands.

Task 9 Elevated Storage Evaluation

Currently pressure to the City's distribution system is maintained through pumps located at two distribution pumping stations. The City has one elevated water storage structure at this time. GUERNSEY engineers performed a cost-benefit analysis to determine the feasibility of creating additional elevated water storage. GUERNSEY used water modeling software to analyze existing and future demands to determine the amount and optimum location of the proposed storage structures.

Task 10 Wellhead Protection Plan Evaluation

The City currently has a wellhead protection plan for each of its five wellfields. The wellhead protection plan is a very important tool for City officials to protect the City's water supply and to reduce the risk of potential pollutants emanating from existing and future development. The existing wellhead protection plans were prepared approximately 10 years ago and are in need of evaluation and updating, as the potential exists for new contamination sources since the previous plans were developed.

Revised wellhead protection plans are currently being prepared by Oklahoma State University (OSU) for the City. Upon the completion of those plans, the City was to provide copies to GUERNSEY for review; however, at the time of this report, the revised wellhead protection plans have not been completed and are therefore not included within this report.

Task 11 Public Meetings/Citizens Group

To obtain public input on City growth trends and other relevant activities, and the potential impacts on water supply/system components, GUERNSEY participated and interacted with the Metropolitan Area Planning Commission. The Commission was used during the development of growth projections as they pertain to future water demands. GUERNSEY held two meetings with the Committee, one at the onset of the project to identify growth scenarios and a second approximately one-half way through the evaluation.

Task 12 Water Rights Analysis

The City currently has approximately 150 active or inactive water wells situated throughout seven wellfields. Over time, wells have been abandoned, re-drilled, or new wells have been added to the network. The City has indicated historical recordkeeping has been less than desirable and groundwater rights may not agree with current well locations.

GUERNSEY assisted the City in comparing well location data with current groundwater rights. GUERNSEY identified wells where additional groundwater rights are necessary as well as areas where water is available but is not being utilized.

Task 13 Water Rate Analysis and Cost of Service Study

GUERNSEY performed a water rate analysis and cost of service study consistent with the American Water Works Association (AWWA) manual on cost of service, rate designs, revenue requirements, and conservation techniques. The following four basic activities were performed in the development of a Rate Analysis and Cost of Service Study:

- System Revenue Requirements – defines the cost associated with operating the system;

- Cost of Service – Allocation of revenue requirement to customer classes;
- Rate Design - Allocation of class revenue requirements to individual customers; and,
- Access or Main Extension Policy – Identifies the access or main extensions supported by the rate design.

The Rate Analysis and Cost of Service Study provide a perspective that allows the City to realize the appropriate balance between maintaining a financially viable system and minimizing the rate change to the customer.

Task 14 Final Report

GUERNSEY prepared this Enid Water System Master Plan.

1.4 <u>FIRE FLOW SCOPE AND REQUIREMENTS</u>

With regards to the fire flow analysis study, GUERNSEY in coordination with City personnel developed a Scope of Work for the Master Plan that involved four distinct tasks. The requirements involved in each of the tasks are described in greater detail in the following sections.

Task 1 Location of Properties

The purpose of the analysis was to identify improvements to the water distribution system to enhance fire flow protection. To help aid GUERNSEY in this task, the City produced a list of all their commercial and public customers and identified their fire flow needs. GUERNSEY selected 25 hydrant pairs that served the greatest number of customers with the largest fire flow demands, in order to model the water distribution system in the vicinity of those hydrants. The purpose of the modeling was to compare actual hydrant flow measurements against predicted hydrant flows and Needed Fire Flows (NFF), to enable the identification of improvements to the water distribution system to enhance fire flow protection.

In order to select the most appropriate 25 hydrant pairs for modeling, it was necessary for GUERNSEY to identify all commercial and public customers in relationship to the hydrants. The City has a GIS system, maintained by Meshek Engineering, in which the locations of all fire hydrants can be graphically represented on aerial photographs of the city. GUERNSEY arranged for Meshek Engineering to create new layers in the GIS system that indicates the NFF of each property on the list. Each new layer represented a range of NFF (e.g. NFF > 5,000 gpm, NFF 4,000 – 5,000 gpm etc.) in order that the spatial distribution of properties with different NFF could be established.

Task 2 Hydrant Selection

On completion of Task 1, GUERNSEY assessed all of the information placed onto the GIS system and selected 25 hydrant pairs for modeling. The hydrants were selected on the basis of maximum NFF coverage i.e. those which can provide fire flow to the most properties with the highest combined NFF. A list of the selected hydrants was provided to the City Fire

Department. In order to implement the following task, the Fire Department carried out an ISO fire flow test at the 25 hydrant pairs, using one hydrant as the flow hydrant and one hydrant as the pressure gauge hydrant, thereby obtaining both flow and residual pressure.

Task 3 Hydrant Modeling

On receipt of the IFO fire flow test data from the City Fire Department, GUERNSEY used the fire flow test data to model the hydrants, to allow comparison between actual and predicted hydrant flows. If the flow from a hydrant was found to be less that that required for fire flows, further modeling was carried out to identify potential distribution improvements to increase hydrant flow.

Task 4 Fire Flow Modeling Report

Information from the modeling, detailing the selection of hydrants, the results of modeling and recommendations made to improve hydrant flows are incorporated the information required for the modeling report into this Enid Water System Master Plan. Section 8.6 of the Master Plan details the activities described above.

2.0 DESCRIPTION/CHARACTERIZATION OF THE STUDY AREA

2.1 CITY OF ENID

Enid is located approximately 67 miles north-northwest of Oklahoma City, Oklahoma near the west-central portion of Garfield County. The City has a population of approximately 47,050 and encompasses approximately 75.43 square miles (www.enidbuzz.com/enidhistory.html). The climate in Garfield County is classified as continental, temperate, sub-humid with an average annual temperature of 60.1°F. The average annual rainfall for the Enid area, as determined by the Oklahoma Climatological Survey, for the periods of 1895 through 2005 is 28.79 inches (Oklahoma Climatological Survey, www.climate.ocs.ou.edu/coop.php).

2.2 AQUIFER CHARACTERISTICS

The City is located in the Red Bed Plains Subregion, of the Osage Plains Section, of the Central Lowland Province, of the Interior Plains Physiographic Region of the United States (A Tapestry of Time and Terrain: Physiographic Regions of the Lower 48 United States, United States Geological Survey, tapestry.usgs.gov/physiogr/phsio.html, September 21, 2006). Geologically, Enid is located on the northeastern portion of the Northern Shelf of the Anadarko Basin (Geologic Provinces of Oklahoma, ok.geolosurvey1.gov/level2/geology/ok.geo.provinces. large. gif, September 21, 2006). The City is situated on the Enid Isolated Terrace Deposits which overlies the Permian-age El Reno Group’s Cedar Hills Sandstone and the Hennessy Group’s Bison, Salt Plains, and Kingman Siltstone Formations (Figures 2-1 and 2-2). Water for the City is supplied by five wellfields located in three aquifers, the Cimarron River Terrace Aquifer, the Enid Isolated Terrace Aquifer and the Permian-age Cedar Hills Sandstone Aquifer (Figures 2-1 and 2-2).

2.2.1 CIMARRON RIVER TERRACE AQUIFER

The Cimarron River Terrace Aquifer is located on the northeast side of the Cimarron River west, southwest, and south of the City, and consists of approximately 1,305 square miles. These deposits extend approximately 115 miles southeasterly from Freedom to Guthrie and unconformably overlie Permian-age formations which outcrop along the northeastern edge of the aquifer (Figure 2-3). The Cimarron River Terrace sediments were originally deposited as alluvium during the southward migration of the ancestral Cimarron River down the regional dip of the underlying Permian formations (Geohydrology of Alluvium and Terrace Deposits of the Cimarron River from Freedom to Guthrie, Oklahoma, Water Resources Investigations Report 95-4066, United States Geological Survey, Gregory P. Adams and DeRoy L. Bergman, 1995). These deposits have been reworked by water and wind to create sand dunes. The Cimarron River Terrace deposits are composed of interfingering lenses of clay, sandy clay and cross-bedded poorly sorted sand and gravel and vary in thickness form 0 to 120 feet. The large variation in thickness is attributed to the erosional surface of the underlying Permian formations.

2.2.2 ENID ISOLATED TERRACE AQUIFER

The Enid Isolated Terrace Aquifer is located in Garfield County and surrounds the general vicinity of the City. This aquifer has an aerial extent of approximately 81 square miles and extends over 52,000 acres (Evaluation of Aquifer Performance and Water Capabilities of the Enid Isolated Terrace Aquifer in Garfield County, Oklahoma, Oklahoma Water Resources Board (OWRB), Douglas C. Kent, Yvan J. Beausoleil and Fred E. Witz, May 1982) (Figure 2-4). The Enid Isolated Terrace Aquifer unconformably overlies Permian-age formations and is bound by outcrops of the Cedar Hills Sandstone to the west, and the Bison, Salt Plains, and the Kingman Siltstone Formations to the north and south, and the Salt Plains, and Kingman Siltstone Formations to the east. As with the Cimarron River Terrace sediments, the sediments of the Enid Isolated Terrace were deposited as alluvium associated with the migration of the ancestral Cimarron River (Evaluation of Aquifer Performance and Water Capabilities of the Enid Isolated Terrace Aquifer in Garfield County, Oklahoma, OWRB, Douglas C. Kent, Yvan J. Beausoleil and Fred E. Witz, May 1982). The Enid Isolated Terrace Aquifer is composed of discontinuous layers of clay, sandy clay, sand, and gravel and averages approximately 60 feet in thickness (Evaluation of Aquifer Performance and Water Capabilities of the Enid Isolated Terrace Aquifer in Garfield County, Oklahoma, OWRB, Douglas C. Kent, Yvan J. Beausoleil and Fred E. Witz, May 1982).

2.2.3 CEDAR HILLS SANDSTONE AQUIFER

The Permian-age Cedar Hills Sandstone Aquifer covers parts of Alfalfa, Garfield, Major and Woods counties. The Cedar Hills Sandstone overlies conformably the Bison Formation and unconformably underlies the Cimarron Terrace sediments. The OWRB has deemed the Cedar Hills Sandstone as part of the El Reno Minor Groundwater Basin (Hydrogeologic Report of the El Reno, Fairview, Isabella, and Loyal Minor Groundwater Basins in Central Oklahoma, Technical Report 2000-1, OWRB, Mark Belden, March 2000). The Cedar Hills Sandstone Aquifer consists of fine to medium grained sandstone interbedded with layers of siltstone and shale. The water bearing strata in the Cedar Hill Sandstone appears to be discontinuous and of a localized nature (Hydrogeologic Report of the El Reno, Fairview, Isabella and Loyal Minor Groundwater Basins in Central Oklahoma, Technical Report 2000-1, OWRB, Mark Belden, March 2000). Groundwater occurs and moves through the fractures and dissolution cavities which is an end result from the removal of soluble materials, such as calcite, within the discrete units. This process created a complex aquifer system exhibiting variable permeability and storage characteristics both laterally and vertically. This is confirmed by the numerous wells that have been drilled in close proximity to wells with significant production and yet, yield very little water (Hydrogeologic Report of the El Reno, Fairview, Isabella, and Loyal Minor Groundwater Basins in Central Oklahoma, Technical Report 2000-1, OWRB, Mark Belden, March 2000). The thickness of the Cedar Hills Sandstone ranges from less than 100 feet to 180 feet (Groundwater Oklahoma’s Buried Treasure, Oklahoma Water Resources Board, www.owrb.state.ok.us/news/publications).

3.0 STUDY METHODOLOGY

3.1 KICK-OFF MEETING

The kick-off meeting brought together the City's staff and GUERNSEY's Team to solidify project goals and served as a data transfer session. A communication network was identified and project milestones were addressed. The Kick-off Meeting was conducted at 1:30 PM on August 8, 2006 at the Enid City Offices and the following personnel were in attendance:

- James McClain – City of Enid;
- Chris Bauer – City of Enid;
- Jason Brinley – City of Enid;
- Robert Hitt – City of Enid;
- Bruce Boyd – City of Enid;
- Ken Senour- GUERNSEY;
- Karl Stickley- GUERNSEY;
- Erik Beiergrohslein –GUERNSEY;
- Carey Miller – GUERNSEY; and
- Michael Dewings – GUERNSEY.

3.2 SITE RECONNAISSANCE/ SITE VISITS

GUERNSEY personnel and City staff members visited pertinent facilities and sites that were relevant to the City's water system including a comprehensive review and observation of the wellfields. The initial site visits were conducted on August 8 and 9, 2006 with additional site visits conducted on September 6 and 7, 2006.

3.3 DATA COLLECTION

Historical information contained within the City's archives were copied and added to the project informational database. GUERNSEY personnel met with Oklahoma Water Resources Board (OWRB) staff on September 25, 2006 to obtain information and data concerning the City's water rights, well locations, well permits and groundwater allocations for the aquifers. Attendees at the meeting consisted of the following:

- Rick Wicker – OWRB;
- Robert S. Fabian – OWRB;
- Phyllis Robertson – OWRB;
- Carey Miller – GUERNSEY; and
- Nathan Madenwald – GUERNSEY.

Additional information was collected from data bases and documents contained within various state and federal agencies.

3.4 PUBLIC INVOLVEMENT

The Metropolitan Area Planning Commission (MAPC) was utilized to obtain public input on City growth trends and other relevant activities, and the potential impacts on water supply/system components. Two meetings were held with the MAPC to identify growth scenarios. Details on the date and content of those meetings are given below:

1. November 13, 2006 – The meeting discussed the project scope and current water rights.
2. February 12, 2007 – The meeting discussed population projection growths for the City and Garfield County. A summary of the meeting is given in Section 8.1.3.

Copies of the material presented to the MAPC are given in Appendix C.

4.0 COLLECTION SYSTEM EVALUATION

A site visit was made to the City on August 8-9, 2006 to observe the wellfields, plants etc. and to become familiar with the different components of the groundwater collection system. This was followed up by a second site visit on September 6-7, 2006 to carry out a detailed condition survey of the current collection system infrastructure. The following sections describe the individual components of the groundwater collection system and their current condition.

4.1 OVERVIEW OF COLLECTION SYSTEM

The City obtains its water from five wellfields in eight locations:

1. Ames Wellfield, located around the town of Ames, southwest of Enid.
2. Drummond Wellfield located between the towns of Ames and Drummond, southwest of Enid.
3. Cleo Springs Wellfield, located west of the town of Cleo Springs, west of Enid.
4. Ringwood Wellfield located southwest of the town of Ringwood, west of Enid.
5. Enid Wellfield, consisting of the following minor wellfields:
   a) Van Buren Wellfield, located in northwest Enid.
   b) Northwest Wellfield, located in northwest Enid.
   c) Carrier Wellfield, located between northwest Enid and the town of Carrier.
   d) Plant/Service Center Wellfield, located at Plant #1 in northwest Enid.

The Van Buren, Northwest, Carrier, and Plant/Service Center Wellfields are collectively known as the Enid Wellfield. These wellfields are identified and located on Figure 2-1.

More detailed information on the location of the wellfields, and the aquifers from which they draw their water, is provided in Section 5.0 "Water Supply Evaluation." An overview of the overall collection system is provided below.

- Raw groundwater from the wellfields is pumped to either Plant #1 water treatment plant (WTP) or Plant #2 WTP.
- Water from the Enid and Drummond Wellfields is pumped directly from the wells to Plant #1.
- The Ames, Cleo Springs and Ringwood Wells all pump into separate plants (one for each wellfield), from where the raw groundwater is either pumped to Plant #1 WTP (from the Ames Wellfield) or Plant #2 WTP (from the Cleo Springs and Ringwood Wellfields).
- Raw groundwater from the Ames Wellfield (via the plant) and the Drummond Wellfield (directly from the wells) is transmitted to Plant #1 WTP via a common transmission line.
- Similarly, raw groundwater from the Cleo Springs and Ringwood Wellfields (via their respective plants) is transmitted to Plant #2 via a common transmission line. A cross connection exists between both lines at the Imo Booster Station.

Historically, the City has developed a total of 166 water supply wells within the Ames, Cleo Springs, Drummond, Enid (Plant/Service Center, Van Buren, Northwest, and Carrier), and

Ringwood well fields. Of the total 166 wells developed, the total number of wells available to supply the City with water at the time of the survey was 147, with 19 wells having been abandoned. Of the available wells, the number of "active" wells was 116, with 31 "inactive" wells. The term "active" is defined as a well that is operable, and includes those that may temporarily be inoperable (e.g. it is awaiting a new pump; it is being re-built, etc.) At the time of the survey (See Section 4.11), 106 of the "active" wells were operable. The term "inactive" is defined as a well that no longer has the infrastructure to operate, or currently has no water in the well. The 19 wells that have been abandoned are not counted in the well numbers in the following sections, which provide an overview of the individual components of the collection system.

4.1.1 AMES WELLFIELD

The Ames Wellfield has a total of 33 wells, of which 26 are currently "active." Twenty-nine of the wells are completed in the Cimarron River Terrace Aquifer and four wells are completed in the underlying Permian Formations. The total depths of these wells vary from 38 feet to 170 feet below ground surface (bgs) with an average depth of approximately 74 feet bgs. The wells completed in the Cimarron River Terrace Aquifer are the deepest wells in the aquifer and provide the greatest yield from typically 200 to 250 gallons per minute (gpm) per well.

The well houses in this wellfield are constructed with a concrete base slab, lightweight concrete block walls with ventilation/light openings, steel door, and a concrete roof slab with a steel access hatch over the pump to allow for the removal of the pump with a crane. As the wells were constructed at different time periods (mid 1950s – 1980), there is some variation in the design of the access hatch. Most of them extend over the width of the well house, in line with the door and pump. The number and location of the ventilation openings (which also allow natural light into the well house) also vary, but all of them are the size of a wall block. Some of the well houses have larger (three blocks wide by three blocks high) windows, with metal frames. The outer walls of the well houses are painted.

In each well house, the discharge pipework is aboveground and contains a blow off line teed into it, a check valve, and a flow meter. Each well house also has an external valve vault that contains a main line valve that is used to isolate the well house from the trunk mains.

Water from the wells is transported from the wellfield to the Ames plant (See Section 4.4) via two trunk mains. The northern half of the wellfield is served by a 14-inch transite (asbestos cement) trunk main that increases in size to 20-inch towards the reservoir. The southern half of the wellfield is served by an eight-inch cast iron trunk main that gradually increases in size to 10-inch, 12-inch, and finally 14-inch towards the reservoir. The two trunk mains feed independently into the reservoir at the Ames plant.

4.1.2 DRUMMOND WELLFIELD

The Drummond Wellfield has a total of 30 wells, of which 20 are currently "active." All wells are completed in the underlying Cedar Hills Sandstone Aquifer. The depths of the wells vary from 61 feet bgs to 210 feet bgs and have an average total depth of 133 feet bgs.

The well houses are constructed with a concrete base slab, lightweight concrete block walls with ventilation/light openings, steel door, and a concrete roof slab with a steel access hatch over the pump to allow for the removal of the pump with a crane. Like the Ames Wellfield, the Drummond Wellfield was developed over a period of time (1950s) and so there is some slight variation in pump access cover and ventilation/light openings. The internal pipework/external valve vault arrangement that exists at the Ames well houses has also been used in this wellfield, although the well houses for recent re-drills have the internal valve vault arrangement, as per the Cleo Springs Wellfield.

The well houses pump directly into the transmission line that runs from the Ames plant (See Section 4.4) to Plant #1 WTP, via six-inch feeder lines.

4.1.3 CLEO SPRINGS WELLFIELD

The Cleo Springs Wellfield has a total of 31 wells, of which 28 are currently "active." All wells are completed in the Cimarron Terrace Aquifer. The depths of these wells are relatively shallow and vary from 21 to 82 feet bgs. Two of the wells are lateral wells approximately 20 feet bgs.

All wells were constructed at the same time (1984 – 1985), and therefore all well houses are identical. They are constructed with a concrete base slab, lightweight concrete block walls with ventilation/light openings, steel door, and a concrete roof slab with a full width steel access hatch over the pump to allow for the removal of the pump with a crane. The outer walls of each well house are painted.

The main difference in design between the newer well houses in the Cleo Springs Wellfield and those located in earlier developed wellfields, is that the well houses in the Cleo Springs Wellfield have an internal valve vault, which contains the check valve, flow meter and isolation valve. Immediately downstream of the pump, there is blow off line teed into the discharge line before it turns through 90° degrees downward to the valve vault.

Water from the wells is transported from the wellfield to the Cleo Springs plant (See Section 4.5) via a single 24-inch ductile iron trunk main. Water from the well houses is pumped into the trunk main via 13 six-inch feeder lines. The trunk main has three 21,000-gallon surge tanks, to minimize surge pressures that may develop in the line.

4.1.4 RINGWOOD WELLFIELD

The Ringwood Wellfield has a total of 28 wells, of which 21 are currently "active." All wells are completed in the Cimarron Terrace Aquifer. The depths of these wells vary from 45 bgs to 83 feet bgs.

The wellfield was developed at the same time as the Cleo Springs Wellfield (1984 – 1985), and so the construction of the well houses are identical to each other (See Section 4.1.3).

Water from the wells is transported from the wellfield to the Ringwood plant (See Section 4.6) via two 24-inch ductile iron trunk mains. Water from the well houses is pumped into the trunk mains via 10 six-inch feeder lines. The two trunk mains join upstream of the reservoir at the

Ringwood plant, and feed into it via a single 24-inch main. The system has two surge tanks; one 12,700-gallon tank on the western trunk main prior to the point where the two mains combine, and one 18,600-gallon tank downstream of the point of combination.

4.1.5 ENID (PLANT/SERVICE CENTER, VAN BUREN, NORTHWEST, AND CARRIER) WELLFIELD

The Enid Wellfield has a total of 25 wells, of which 21 are currently "active." All wells are completed in the Enid Isolated Terrace Aquifer. The depths of the wells vary from 35 feet bgs to 80 feet bgs.

The first wells were developed in the late 1890s in the Enid Isolated Terrace Aquifer. Between 1890 and 1950, 32 wells were installed in the Van Buren, Northwest, and Carrier Wellfields, and the original nine wells were abandoned. As of September 2006, eight well houses in the Carrier Wellfield had recently been demolished and three new well houses were under construction. Two new well houses were also under construction at Plant #1. There has also been some demolition of well houses in the past in the Van Buren and Northwest Wellfields, with recent construction of two new well houses in the Van Buren Wellfield.

The older well houses are similar in style to those found in the Ames and Drummond Wellfields, except that the access hatches for the pump do not extend the full width of the building. All well houses in the Northwest Wellfield appear to have been constructed from brick as opposed to lightweight concrete block. The two new well houses in the Van Buren Wellfield have pitched roofs constructed from profiled metal sheeting, with two lifting beams at either end of the house to lift off the roof for pump removal. The two well houses under construction at Plant #1 have the "traditional" concrete roof slab. The new well houses have the internal valve vault arrangement, as per the Cleo Springs wellfield.

Water from the wells is transported from the wellfields to Plant #1 WTP via a single 8-inch trunk main that gradually increases in size to 10-inch, 14-inch, and finally 16-inch ductile iron towards the WTP. Water from the well houses is pumped into the trunk main via a series of feeder lines.

4.2 WELL PUMPS

The original pumps installed in all the well houses were vertical turbine; however the City is now slowly replacing them with submersible pumps. In addition, submersible pumps are being installed in the new well houses being constructed (See Section 4.1.5).

The reason for the change in pump type is that the vertical turbine pumps are prone to damage by cavitation when the water level in an aquifer drops below normal levels (e.g., during a drought year when the head of water at the inlet to the pump is reduced). Cavitation occurs when the net water pressure at the pump inlet drops below the vapor pressure of water. In this situation, the net positive suction head available (NPSHA – the actual water energy at the pump inlet) on the pump becomes less than the net positive suction head required (NPSHR), the minimum fluid energy required at the pump inlet for satisfactory operation. Submersible pumps have the advantage over vertical turbine pumps as they are more tolerant of low NPSHA conditions.

4.3 WELL OPERATION

There are several factors that determine which wells the City operates at any given time to meet the water demand.

The Cleo Springs and Ringwood Wellfields are operated in alternation. For example, the Cleo Springs Wellfield will operate and the Ringwood Wellfield will be shut down, and will only be used when required to meet demand. After every couple of months, the operation of the wellfields is reversed.

The City pays a royalty on the water produced from the Ames Wellfield at a current cost of $0.20 per 1,000 gallons of water; therefore, to reduce expenses these wells are mainly used in the summer to help meet peak demand and are seldom utilized during the winter.

Both Plant #1 WTP and Plant #2 WTP have a run order, which lists the sequence in which wells should be started or stopped in response to the demand for water in each system. The run order shows the individual output of each well and the cumulative output of the wells as the next well on the run order is brought into operation. When there is a need to increase production, the next wells down the list are brought online until the cumulative output of the wells in operation matches the required production. Conversely, when there is a need to decrease production, wells are switched off from the bottom of the current operating wells on the list upwards until the cumulative output of the wells in operation is reduced to match the required production.

The list is updated regularly to account for maintenance, operational practices, and new water quality data obtained from sampling. The order of wells on the list is determined by royalty payments and water quality. Wells that require the City to pay a royalty are located at the bottom of the list, and so are only used in periods of peak demand. The run order shows the nitrate, chloride and total dissolved solids (TDS) concentrations for individual wells, and for blended water of the given wells in operation based upon the cumulative well outputs. Therefore the water quality can be predicted for the given combination of wells in operation. The run order can therefore be established from the wells operationally available to ensure that the best quality water is produced, and that drinking water regulatory limits are not exceeded.

4.4 AMES PLANT

The Ames Plant was constructed in 1950, and consists of a one-million gallon (MG) aboveground, circular, raw groundwater storage reservoir, and a pump house. All the wells in the Ames Wellfield pump into the storage reservoir. The plant pumps raw groundwater from the storage reservoir to Plant #1 WTP via a transmission line (See Section 4.7), which was installed at the same time as the plant and the development of the Ames and Drummond Wellfields.

The pump house contains three split-case centrifugal pumps (Table 4-1). The 200 hp variable frequency drive (VFD) pump is the lead pump, and its output is set manually via the Supervisory Control and Data Acquisition (SCADA) system. The fixed speed 200 hp pump is the main standby pump. The 75 hp pump can either be driven by an electric motor or a propane engine. The propane engine acts as an emergency back-up in the event of loss of power to the plant.

4.5 CLEO SPRINGS PLANT

The Cleo Springs Plant was constructed in 1984 to serve the development of the Cleo Springs Wellfield. It consists of a 1.5 MG below ground, rectangular, raw groundwater storage reservoir, and a pump house. All the wells in the Cleo Springs Wellfield pump into the storage reservoir. The plant pumps raw groundwater from the storage reservoir to Plant #2 WTP via the transmission line (See Section 4.8) that also transports water from the Ringwood Wellfield.

The pump house contains four vertical turbine pumps (Table 4-2). The 200 hp variable frequency drive (VFD) pump is the lead pump, and its output is set on the SCADA system, based upon achieving a given flow rate. The smaller 100 hp pumps are standby pumps. The diesel engine pump is an emergency back-up pump in the event of loss of power to the plant.

4.6 RINGWOOD PLANT

The Ringwood Plant was constructed in 1984 to serve the development of the Ringwood Wellfield. It consists of a 1.5 MG below ground, rectangular, raw groundwater storage reservoir, and a pump house. The plant is identical in construction to the Cleo Springs plant. All the wells in the Ringwood Wellfield pump into the storage reservoir. The plant pumps raw groundwater from the storage reservoir to Plant #2 WTP via the transmission line (See Section 4.8) that also transports water from the Cleo Springs Wellfield.

The pump house contains four vertical turbine pumps (Table 4-3). All three electric motor pumps are run using one VFD. The 150 hp pump is the lead pump, and its output is set on the SCADA system, based upon achieving a given flow rate. The smaller 75 hp pumps are standby pumps. The diesel engine pump is an emergency back-up pump in the event of loss of power to the plant.

4.7 TRANSMISSION LINE – AMES AND DRUMMOND WELLFIELDS

From the Ames Plant, raw groundwater is pumped to Plant #1 WTP via a 30-inch reinforced concrete transmission line, approximately 81,300 ft (15.25 miles) long. Wells in the Drummond Wellfield pump directly into the transmission line. The line passes through the Imo Booster Station (See Section 4.9).

4.8 TRANSMISSION LINE – CLEO SPRINGS AND RINGWOOD WELLFIELDS

The Cleo Springs Plant and the Ringwood Plant both pump into their own 30-inch ductile iron transmission line at the plant. These two transmission lines combine into one 42-inch ductile iron line close to the town of Ringwood. This main passes through the one-million-gallon

Ringwood Surge Tower, located at the highest point on the main. The surge tank has a 30-inch bypass pipeline, which is currently being used. The tower was bypassed in 1997 while it was re-painted, when the City discovered that the transmission system worked fine without it, and it has remained empty since then. The total length of transmission line from Cleo Springs to Plant #2 WTP is approximately 30 miles.

4.9 IMO BOOSTER STATION

The Imo Booster Station is located on Imo Road, on the southwest outskirts of Enid. The booster station was originally designed to pull a vacuum off the 30” concrete transmission line, and discharge water to the west side of Enid via a 24-inch main. With the construction of the Cleo Springs and Ringwood Wellfields, the 42-inch transmission line and Plant #2 WTP, the booster station was mothballed in 1984. In 2006, the booster station was re-activated and modified, to enable water to be pumped from the 30-inch transmission line into the 42-inch transmission line, and vice versa, utilizing the 24-inch main which ties into the 42-inch transmission line. The arrangement enables both Plant #1 WTP and Plant #2 WTP to receive raw groundwater from any of the four wellfields (Ames, Cleo Springs, Drummond, and Ringwood) located to the west or southwest of Enid.

The booster station originally had three pumps, but at the time of the survey only one 150 hp pump rated at approximately 2,000 gpm was in service.

4.10 SCADA SYSTEM

The City has a fairly comprehensive SCADA system that allows the water collection and distribution systems to be monitored and controlled. On the water collection system, the SCADA system is used to monitor and control the operation of the well pumps and the plant pumps.

The City has a licensed radio system, with a radio communication mast or antenna located at the following sites to facilitate the transfer of information within the system:

- Plant #1 WTP
- Plant #2 WTP
- Ames Plant
- Cleo Springs Plant
- Ringwood Plant
- Ringwood Surge Tower
- Imo Booster Station
- Elevated Water Tower (on the water distribution system)

Approximately 90 wells are connected to the telemetry system and can be remotely operated via a SCADA station. All wells in the Cleo Springs and Ringwood Wellfields are hard wired to their respective plants. The two plants then transmit and receive signals via the Ringwood Surge Tower communication tower. At the time of the survey, the modems were inoperable at 13 wells in the Ringwood Wellfield (Wells # 1, 2, 3, 4, 7, 9, 14, 17, 18, 21, 26, 27, and 28) due to a recent lightning strike, but were replaced.

The remaining wells on the SCADA system are in the Ames and Drummond wellfields. They communicate with the Ames Plant using a radio telemetry system. The wells in these wellfields that are not on the system are those which are used infrequently (i.e. high nitrate wells). None of the wells in the Enid Wellfield are on the SCADA system, due to their close proximity to Plant #1 WTP.

The City recently upgraded all of the programmable logic controllers (PLCs). At the time of the survey, Plant #2 WTP, and the Ames, Cleo Springs and Drummond wellfields and plants had Modicon PLCs. Plant #1 WTP and the Ringwood Wellfield and plant had already been upgraded to DirectLogic 205 PLCs, manufactured by Automation Direct. The reason for the upgrade is that the Modicon PLC is now old technology (introduced in 1992). The advantages of the DirectLogic 205 PLC over the Modicon PLC include:

- It is upgradeable. The City is looking to replace electronic equipment every five years. With the DirectLogic 205 PLC, the PLC can be up-graded as opposed to being changed.

- More control can be carried out. For example, the Ringwood Wellfield system (after it was upgraded) had the ability to reset well pump run times on the SCADA system, whereas the Cleo Springs Wellfield system prior to being upgraded did not have this facility.

The City has also recently upgraded the remote telemetry units (RTU's) used to transmit data. At the time of the survey, the Ames and Drummond wells on the SCADA system communicated with the Ames Plant using Zetron RTUs. With these units, it took approximately eight minutes for the SCADA system to poll all of the wells on the system. The City replaced these units with MDS 1710 Transceiver Series telemetry radio, manufactured by Microwave Data System. The reason for the upgrade is that while the Zetron RTUs are capable of being used as a telemetry radio to transmit data, they are primarily designed as a voice radio. A consequence of this was that the transmitted data was inconsistent, leading to errors in the received data. The upgrade therefore reduces data error. The MDS 1710 system also reduced the time to poll all of the wells on the system to approximately two minutes. As the MDS 1710 operates on a different frequency band (130 – 174 MHz, as opposed to 450 MHz) to the Zetron RTU's, the radio antennas within the system were replaced.

The City does not use proprietary SCADA system software. Instead, the City has written their SCADA system software using Microsoft-based software. This allows the City to upgrade the system as newer versions of the software become available at relatively little cost. Additionally, the SCADA system has been tailored to meet the City's needs.

The software used by the City in their system includes:

- KEPServer, HMI (human-machine interface) software that allows Windows DDE (Dynamic Data Exchange) compliant applications to exchange data with PLCs, so the software can be used with different PLCs.

- Microsoft Visual InterDev 6.0, web development software to produce web pages.

- Iocomp html control software, which allows the development of plant controls and plot graphs using web pages.

Currently, the software can produce multiple-parameter, real time charts, and at midnight the data moves over to being historic data, from which historic charts can be produced. The software allows historic data for a single parameter to be plotted on a user graph. Multiple parameter historic graphs cannot currently be plotted, but the program can be changed to do this. The software allows the system log to be viewed, and a record of alarms can either be picked up from the system log or an alarm log can be generated.

The main web page for each of the three plants (Ames, Cleo Springs, and Ringwood) shows the status of the reservoir level and the VFD speed. Except for the Ames Plant, the flow of water out of the plant is also shown. The main page also shows the status of the plant pumps and the run status of each well. Each plant also has a wellfield control page, which allows each pump to be started or stopped, and provides information on the status of the well (Running, Auto, Power, Overload, and CommFail).

4.11 CONDITION OF INFRASTRUCTURE

The detailed condition survey covered the following components of the collection system:

- All well houses in the Ames, Drummond, and Enid (Service Center, Van Buren, Northwest, and Carrier) wellfields.

- Approximately 1/3 of the well houses in the Cleo Springs and Ringwood Wellfields.

- The Ames, Cleo Springs, and Ringwood plants.

As all well houses in the Cleo Springs and Ringwood wellfields were constructed at the same time, it was decided to survey a sample of the total well houses, and that the condition of the sample well houses would be representative of the total well houses.

Further details on the individual well houses and plants are contained within Appendix A.

4.11.1 AMES, DRUMMOND, AND ENID WELL HOUSES

With the exception of re-drilled wells with new well houses, all wells in the Ames, Drummond, and Enid wellfields were constructed prior to 1980 over a period of time. There is therefore some variability as to the condition of the well house structure and the equipment inside. However, there are some maintenance issues that appeared to be common among many of the well houses.

The most commonly recurring maintenance issues for the well houses are identified below:

- Peeling exterior paintwork.
- Damage to wall blocks.
- Spalling of wall block surface due to freeze/thaw action.

- Rusting along the top or bottom edge of steel access door.
- Rusting of pump access cover.
- Openings made through the wall for pipework but not sealed after pipe installation.
- Cracks in interior and exterior mortar joints to wall blocks.
- Cracks running through wall block.
- Valve vaults in poor physical condition.
- Spalling of concrete on roof slab.

The issues identified indicate a slow, continuous and ongoing degradation of the well house structures since they were constructed, which if left unchecked, could lead to serious defects in the structural integrity of the well houses. In some well houses, signs that repairs have been made in the past are apparent. Regarding cracks running through wall blocks, the cracks do not necessarily run through the width of the block, which could indicate some form of structural settlement. In areas where spalling of the block surface has occurred, multiple, random cracks are visible on the surface of the block in the vicinity of the spalled area. However, in a few instances, there is only one vertical crack visible running through the middle of blocks, which does indicate some form of structural settlement.

Regarding the equipment housed within the well houses (pumps, pipework, valves, electrics etc.), again there is variability as to their condition. The City replaces pumps and repairs motors on an "as required" basis, therefore some pumps and motors are much older than others. When a pump is replaced, the City tends to upgrade the electronics within the existing control panel; therefore, many of the well houses have control panels that from the outside appear to be aged, but have been upgraded inside.

The City may also replace old pipework and valves when a pump is changed. Numerous pumps, pipework, and valves suffer from surface rusting, but this appears to be cosmetic. At the time of the survey, several pumps were leaking from packing, and some of the main line valves within the valve vaults were leaking, but the City routinely address these problems. There is therefore a gradual, continuous replacement of piping and equipment as required.

Heat within some of the well houses appeared to be an issue. Many of the wells that are shut down for the winter have insulation; however, all appeared to be in poor condition, possibly due to rodent activity. In some well houses, a plug-in electric space heater has been installed to maintain the temperature in the winter. Conversely, excess heat in the summer is problematic. In some instances (such as at Ames #6), an exhaust fan has been installed to remove excess heat in the summer to enable a new motor to operate. Well houses with windows merely have the frame but no glass. The windows are left in the open position in the summer, and then closed in the winter with the opening blocked up with plywood. Some of the well houses have had additional ventilation installed by knocking out individual wall blocks and installing a grill over the opening.

The Ames #7 well house has a problem in that every time it rains, water runs off the nearby road, down the well house vehicle parking area, and into the well house, causing the floor to flood. This has lead to the deposition of large quantities of sand and silt in front of and inside the well house.

At the time of the survey, some wells were not in operation for the given reasons (as stated by City staff) in Table 4-4. The current status of the wells ("active", "inactive" or abandoned) is also addressed.

Of the 49 inactive wells listed in Table 4-4, 9 are "active" wells that should be back in operation with either new pumps, or completion of construction. Of the remaining 40 wells, 19 have been abandoned and 21 are "inactive" wells; eight have lack of water and/or are silted in, three have pumps stuck in the well, 10 are out of operation for miscellaneous/unstated reasons.

4.11.2 CLEO SPRINGS AND RINGWOOD WELL HOUSES

From the sample survey carried out, it would appear that all wells in the Cleo Springs and Ringwood wellfields are in good condition. Very few of the problems encountered in the older wellfields appeared in the survey wells. The only things noted were some slight paintwork peeling, some slight wall block cracking, and some slight cracking of mortar. It is therefore reasonable to assume that the design and construction of the well houses is such that they can last at least 20 years before signs of deterioration begin to appear.

All internal equipment to the well houses is still in good condition, and some of the pumps have been replaced "as required." There were several pumps surveyed that had leaking packing.

One problem that has occurred in some of the Cleo Springs wells is the presence of iron fixing bacteria, which thrive on the soluble iron occurring in the groundwater. The bacteria cause the iron to come out of solution, causing iron deposits within the well and pipework. The City combats this by chlorinating the well three to four times a year, and then adding hydrogen peroxide to the well every few years.

At the time of the survey, some wells were not in operation for the given reasons (as stated by City staff) in Table 4-4. The current status of the wells ("active" or "inactive") is also addressed.

4.11.3 AMES PLANT

The exterior of the pump house appears to be in good condition. The outer surface to the reservoir is showing some signs of rusting reinforcement in a few areas, but there are no leaks from the reservoir. The roof of the reservoir is showing signs of wear and tear in a few locations.

There are a few equipment problems associated with the plant. The back-up propane engine does not work; therefore the plant will not be able to pump water should a power failure occur. The air valve to Pump #2 does not operate, and the 20-inch reservoir valves have never been exercised. The motor control center works satisfactorily, although it is outdated.

4.11.4 CLEO SPRINGS PLANT

The pump house and reservoir appear to be in good condition. The only minor signs of deterioration are patches of peeling paintwork to the edge of the roof and the interior walls of the pump house. There are signs of rusting on some of the pipework work and valves, but this is superficial. All equipment is in working order, and the motor to the 200 hp pump has been replaced.

4.11.5 RINGWOOD PLANT

All comments made in regards to the Cleo Springs Plant also apply to Ringwood Plant, except that all of the equipment is original, none has had to be replaced. One issue with the plant at the time of the survey was that the VFD was out of commission, due to the same lightning strike that had knocked out the modems in the wellfield (See Section 4.10).

4.11.6 IMO BOOSTER STATION

The booster station has recently received minimal upgrades to enable it to be re-commissioned. Of the three original pumps (150 hp, 200 hp, and 300 hp), the 150 hp pump has been retrofitted with a new motor, part of the motor control center has been upgraded, and valving modifications made to enable 2,000 gpm (2.88 MGD) to be pumped in either direction.

Physically, the foundations and concrete walls of the station are sound. The City has plans to implement a second phase of improvements, namely re-store the interior finish of the building (the roof trusses require attention), replace the pumps and upgrade the motor control center. The City has already replaced the roof to the station.

4.11.7 WELLFIELD COLLECTION LINES

The only issues offered by City staff concerning wellfield collection lines were in reference to the Ames Wellfield. There have been some recent bursts in both the transite and ductile iron trunk mains. From the previous "Water Supply Capacity Analysis" study carried out by Envirotech in 2000, there appears to be adequate capacity in the collection lines.

4.11.8 TRANSMISSION LINES

City staff reported no problems with the 42-inch ductile iron transmission line from the Cleo Springs and Ringwood wellfields. From the previous "Water Supply Capacity Analysis" study carried out by Envirotech in 2000, it is believed that there have been pipe bursts in the past to the 30-inch reinforced concrete transmission line from the Ames and Drummond wellfields, as it appears that the maximum pressure in this line is limited to 55 pounds per square inch gage (psig). At this pressure, the study states that the capacity of the transmission line is 10.2 MGD. The current production from the two well fields (Table 4-4) is 6660 gpm (9.59 MGD), but this is down from the original capacity of the well fields.

4.12 FUTURE INFRASTRUCTURE REQUIREMENTS

From the discussions in Section 4.11, the condition of the different components of the collection system can be summarized as follows-

- The Cleo Springs and Ringwood collection systems infrastructure (well houses, plants, collection lines and transmission lines) is in good condition and currently requires no major attention.
- The Ames, Drummond and Enid collection systems infrastructure is aging, and currently requires refurbishment in several areas.
- The City replaces pumps and motors on an "as required" basis. Replacement also usually involves electrical upgrades to the control panel, and replacement of pipes and valves where required.
- The City has recently upgraded the SCADA system to reduce errors in data transmission, and to provide additional control to the operation of the collection system.

From the above, it is clear that the City should focus future infrastructure requirements on the Ames, Drummond and Enid wellfields, to improve the condition of the collection systems to a reasonable standard and prevent further deterioration. Areas that require attention are discussed below.

4.12.1 AMES PLANT

The Ames Plant lacks an operable emergency standby pump or a standby generator, and the City is currently unable to pump water from the reservoir during a power outage. Loss of pumping capability would result in water shortages in City until such time that power is restored. Unless the City is able to hire in an emergency generator at short notice at any part of the day, the replacement of the emergency standby pump, or the installation of an emergency generator, should be considered a priority. Consideration should also be given to replace worn out valves to ensure full and proper operation of the plant.

4.12.2 WELL HOUSES

The majority of the well houses in the Ames, Drummond and Enid wellfields are showing signs of ongoing structural degradation. The City has estimated that the well houses have a life span of 50 years. On this basis, at the time of the survey, of the 88 'active' and 'inactive' wells present in the three well fields, a total of 49 wells (19 of the wells in the Enid well field, 20 wells in the Drummond well field, and 10 wells in the Ames well field) are due for well house replacement, as they are over 50 years old.

In 2005, the City began a program of building replacement wells, at a rate of three wells per year. At the time of the survey, 6 wells have already been replaced, all in the Enid Wellfield. By the time the above 49 wells have been replaced, further wells would be due for replacement. At the current rate of replacement, a total of 150 wells (i.e. all of the current wells) could be replaced in 50 years.

The above scenario would commit the City in the future to a continuous, never ending program of well house replacement. Alternatives to this arrangement include-

- Increasing the number of wells replaced per year. This would allow the reduction in time scale to replace a group of wells, and provide breaks in the replacement program before the next group of wells are due for replacement. This would also allow 'inactive' wells to be brought back into service quicker.
- The number of wells replaced per year could be increased by changing the method of well house construction. One example would be to purchase manufactured glass reinforced plastic (GRP) enclosures, which would then be brought to site and bolted down onto a concrete slab. Such a construction would be quicker than using lightweight concrete blocks.
- Design the replacement well houses with a longer life span e.g. 75 years. This would involve changing the materials of construction from lightweight concrete blocks to something more substantive, such as brick or concrete. A present worth analysis would be required to compare the initial increase in construction cost against the cost of replacing the current design well houses more frequently.
- Instigate a scheduled maintenance program to rehabilitate the existing well houses to extend their life span before replacement is due. This would involve attending to some of the issues raised in Section 4.11.1 in a systematic manner. A present worth analysis would be required to compare the cost of carrying out the maintenance program against the cost of replacing the well house earlier.

4.12.3 IMO BOOSTER STATION

The City already has begun to implement the second phase of improvements at the Imo Booster Station. The improvements performed at the time of the survey are minimal, just enough to re-commission the station. The second phase improvements should be sufficient to bring the station into a decent working order, which would involve replacement of the old pumps and motors and upgrading the control panel to modern standards (replacement of the roof has already been undertaken). The completion of the second phase improvements would provide the City with a secure method of transferring water from one transmission line to another, offering operational flexibility in the operation of the collection system. Given the condition of the 30-inch transmission line (see Sections 4.11.8 and 4.12.5), a hydraulic profile should be performed to ensure that the output of the refurbished booster station does not exceed the current pressure rating limit of the line.

4.12.4 AMES WELLFIELD COLLECTION LINES

The City has suffered from pipe line breaks in the well collection lines in the Ames Wellfield. Although not a priority, the City should consider replacement of the wellfield collection lines in the future, as the frequency of pipe bursts is likely to increase over time. Transite pipework, although durable and corrosion resistant, is an inherently brittle material with low impact resistance. Being a rigid pipe, it is not as flexible as semi-rigid pipes (ductile iron, PVC) and so is more susceptible to fracturing due to ground movements and traffic loading.

4.12.5 AMES AND DRUMMOND WELLFIELDS TRANSMISSION LINE

Due to the pressure in the main being limited in the transmission line to prevent line bursts, the capacity of the transmission line has been reduced, such that the calculated capacity of the transmission line at the reduced pressure is almost matched by the current output of the two well fields. There are wells in both well field that are 'inactive' not due to a lack of water, but due to infrastructure problems with the wells. The City could increase production in these well fields by re-drilling, but the increase in capacity would be hampered by the capacity of the transmission line. To pump more water, the pressure in the line would have to be raised, increasing the likelihood of pipe bursts. The City should therefore consider either replacing the existing transmission line with a larger diameter line, or installing a second transmission line to complement the existing transmission line, to increase the transmission capacity from the Ames and Drummond wellfields. Such a project should be scheduled in conjunction with any program of increased well replacement construction of 'inactive' wells to increase production from the two wellfields.

5.0 WATER SUPPLY EVALUATION

5.1 HISTORY OF ENID'S WATER SYSTEM

The first municipal water supply wells for the City were installed in the late 1890s in what is known as the Enid Isolated Terrace Aquifer, immediately west of the original town site. The first supply system included nine wells in the King Farm Wellfield and a swell gallery located at the original water plant. Between 1890 and 1950, 32 additional wells were installed in the Van Buren, Northwest, and Carrier wellfields. These wellfields are collectively known as the Enid Wellfield. By 1950, the original nine wells in the King Farm Wellfield had been abandoned. The remaining 32 wells in the Enid Wellfield had an average capacity of 2.3 million gallons per day (MGD) and a maximum capacity of 3.85 mgd.

In 1944, a study was conducted for an additional water supply to meet the demand of the City's growing population. The study concluded that sufficient water resources were present in the Cimarron River terrace deposits near Ames, Oklahoma to meet these growing demands. The City attempted to develop additional water supply wells in this aquifer; however, these attempts were blocked by the opposition of landowners and irrigation interests. The City was forced to look elsewhere. An evaluation of the Permian-age Cedar Hill Sandstone formation near the City of Drummond, Oklahoma was conducted and by 1955, 32 water supply wells had been installed in what is known as the Drummond Wellfield. The additional wells increased the average capacity of the City's water supply system to 5.0 MGD with a maximum capacity of 12 MGD.

The three-year drought between 1951 and 1954 coupled with the City's increasing demand for water required a search for additional water resources. The City was successful in acquiring water rights leases from the St. Louis and San Francisco Railroad and five wells were installed within the railroad right-of-way near Ames. These wells were completed in the Cimarron River Terrace Aquifer and represented the first development of the Ames Wellfield.

In 1969 the City was again faced with the need to increase its water supply and in early 1970 successfully leased additional water rights in the vicinity of Ames. These leases included significant areas underlain by the Cimarron River Terrace Aquifer. Between 1970 and 1980, 33 additional wells were completed in Ames Wellfield. Thirty of these wells were completed in the Cimarron River Terrace Aquifer and three wells were completed in the Cedar Hills Sandstone Aquifer. In 1980, the average water demand was approximately 14 MGD and the maximum demand was 21 MGD.

Due to the rapid growth in population and new construction in the early 1980s, the City experienced potential water shortages and problems in supplying adequate volumes and pressures in the water distribution system. A Water Source Study and Distribution System Analysis were completed in 1982. Based on these studies, 59 additional wells were completed in the Cimarron River Terrace Aquifer between 1983 and 1985. These wells established two new wellfields, the Cleo Springs Wellfield and the Ringwood Wellfield. Thirty-one wells were installed in the Cleo Springs Wellfield and 28 wells were installed in the Ringwood Wellfield. These new wells provided an additional 10.5 mgd capacity to the City's water supply system.

The City's water demand has decreased from its peak in the spring of 1983 of an average demand of 15 MGD and a maximum demand of 22 MGD to its current daily average of 9.6 MGD with a peak demand of 17.1 MGD. This decrease in demand was due to a decrease in population from 57,800 to 47,045 (United States Census Bureau, www.census.gov) and a depressed economy associated with a downturn in the oil, natural gas and agricultural industries and the closing of several major industrial facilities.

Historically, the City has developed a total of 166 wells within the Ames, Cleo Springs, Drummond, Enid (Plant/Service Center, Van Buren, Northwest, and Carrier) and Ringwood wellfields (Figure 2-1, Table 4-4). Of the total 166 wells, 116 are currently active and supply water to meet the City's water requirements. Thirty-one wells are inactive and the remaining 19 wells have been abandoned. Production from the City's wellfields from 2000 through 2005 has averaged 3,386 million gallons per year (MGY) or 9.27 mgd. A review of the production figures for the City's wellfields from 1995 through 2005 revealed the City's maximum production of 3,723 MGY or 10.2 MGD occurred in 2000 (Table 5-1, Figure 5-1). The maximum production capacity of the City's water supply system is approximately 17.1 MGD based on the current reported well pumping rates.

5.2 AQUIFER CHARACTERISTICS

A description of the characteristics of each aquifer used by the City to supply water is given in Section 2.2.

5.3 AVAILABILITY OF GROUNDWATER

5.3.1 CIMARRON RIVER TERRACE AQUIFER

The saturated thicknesses, water elevations and approximate porosities were determined from information obtained from the City for the wells located within the Cimarron Terrace Aquifer, and are summarized in Table 4-4. The information contained within this table was used to determine the current thickness of the saturated zone and to construct Isopach Maps depicting the approximate thickness of the saturated zones for the Ames, Cleo Springs, and Ringwood wellfields located within the Cimarron Terrace Aquifer (Figures 5-2 through 5-4).

Calculations made from the information derived from the City and the Ames Wellfield Saturated Zone Isopach Map (Figure 5-2) show that the average saturated thickness of the aquifer in the Ames Wellfield is 39.3 feet. This Isopach Map also depicts the approximate area of the Ames Wellfield. This area encompasses approximately 12,900 acres. This would indicate that there is approximately 505,000 acre-feet of saturated zone contained within the boundaries of the wellfield. Information obtained from the City indicates that an average porosity of 18% exists in the Cimarron Terrace Aquifer. This would indicate that there are approximately 90,900 acre-feet of water in storage for the Ames Wellfield.

Calculations made from the information derived from the City and the Cleo Springs Wellfield Saturated Zone Isopach Map (Figure 5-3) show that the average saturated thickness of the aquifer in the Cleo Springs Wellfield is 26.8 feet. This Isopach Map also depicts the approximate area of the Cleo Springs Wellfield. This area encompasses approximately 7,000 acres. This would indicate that there is approximately 187,500 acre-feet of saturated zone contained within the boundaries of the wellfield. Information obtained from the City indicates that an average porosity of 18% exists in the Cimarron Terrace Aquifer. This would indicate that there are approximately 33,800 acre-feet of water in storage for the Cleo Springs Wellfield.

Calculations made from the information derived from the City and the Ringwood Wellfield Saturated Zone Isopach Map (Figure 5-4) show that the average saturated thickness of the aquifer in the Ringwood Wellfield is 23.8 feet. This Isopach Map also depicts the approximate area of the Ringwood Wellfield. This area encompasses approximately 9,900 acres. This would indicate that there is approximately 235,000 acre-feet of saturated zone contained within the boundaries of the wellfield. Information obtained from the City indicates that an average porosity of 18% exists in the Cimarron Terrace Aquifer. This would indicate that there are approximately 42,300 acre-feet of water in storage for the Ringwood Wellfield.

The aquifer recharge rate for the Cimarron Terrace Aquifer has previously been determined to be approximately 8.5% of the mean average annual precipitation for the area (Geohydrology of Alluvium and Terrace Deposits of the Cimarron River from Freedom to Guthrie, Oklahoma, Water Resources Investigations Report 95-4066, United States Geological Survey, Gregory P. Adams and DeRoy L. Bergman, 1995). The average annual rainfall for the North Central Climate Division (which includes Garfield, Major and Woods Counties) as determined by the Oklahoma Climatological Survey for the periods of 1895 through 2005 is 28.79 inches (Oklahoma Climatological Survey, www.climate.ocs.ou.edu/coop.php). By taking 8.5% of this average, an annual recharge rate of approximately 2.45 inches (0.204 feet) can be established for the area encompassing the City's wellfields in the Cimarron Terrace Aquifer. Recharge from agricultural wells, streams, and the Cimarron River is relatively small in the area and is not included within these calculations. Based on the aereal extent of the wellfields, the yearly recharge to the aquifer for the Ames, Cleo Springs, and Ringwood wellfields would be equivalent to approximately 2,600; 1,400 and 2,000 acre-feet of water respectively.

5.3.2 ENID ISOLATED TERRACE AQUIFER

The saturated thicknesses, water elevations and approximate porosities were determined from information obtained from the City for the wells located within the Enid Isolated Terrace Aquifer, and are summarized in Table 4-4. Calculations made from the information derived from the City and the Enid Wellfield Saturated Zone Isopach Map (Figure 5-5) show that the average saturated thickness of the aquifer in the Enid Wellfield is 26.5 feet. This Isopach Map also depicts the approximate area of the Enid Wellfield. This area encompasses approximately 6,700 acres. This would indicate that there is approximately 177,700 acre-feet of saturated zone contained within the boundaries of the wellfield. Information obtained from the City shows that an average porosity of 25% exists in the Cimarron Terrace Aquifer. This would indicate that there are approximately 44,400 acre-feet of water in storage for the Enid Wellfield.

The aquifer recharge rate for the Enid Isolated Terrace Aquifer has previously been determined to be approximately 7.4% of the mean average annual precipitation for the area (Hydrogeologic Report of the El Reno, Fairview, Isabella, and Loyal Minor Groundwater Basins in Central Oklahoma, Technical Report 2000-1, OWRB, Mark Belden, March 2000). By taking 7.4% of the annual average rainfall (28.79 inches), an annual recharge rate of approximately 2.13 inches

(0.178 feet) can be established for the Enid Isolated Terrace Aquifer. Recharge from agricultural wells, streams, and the Cimarron River is relatively small in the area and is not included within these calculations. Based on the aereal extent of the wellfield, the yearly recharge to the aquifer for the Enid Wellfield would be equivalent to approximately 1,200 acre-feet of water.

5.3.3 CEDAR HILLS SANDSTONE AQUIFER

The saturated thicknesses, water elevations and approximate porosities were determined from information obtained from the City for the wells located within the Cedar Hills Sandstone Aquifer, and are summarized in Table 4-4. The information contained within Table 4-4 was utilized to determine the current thickness of the saturated zone and construct an Isopach Map (Figure 5-6) depicting the approximate thickness of the saturated zone for the Drummond Wellfield located within the Cedar Hills Sandstone Aquifer.

Calculations made from the information derived from the City and the Drummond Wellfield Saturated Zone Isopach Map (Figure 5-6) show that the average saturated thickness of the aquifer in the Drummond Wellfield is 80.4 feet. This Isopach Map also depicts the approximate area of the Drummond Wellfield. This area encompasses approximately 16,100 acres. This would indicate that there is approximately 1,294,100 acre-feet of saturated zone contained within the boundaries of the wellfield. Information obtained from the City indicates that an average porosity of 18% exists in the Cimarron Terrace Aquifer. This would indicate that there are approximately 233,000 acre-feet of water in storage for the Drummond Wellfield.

The aquifer recharge rate for the Cedar Hills Sandstone Aquifer has previously been determined to be approximately 2.5% of the mean average annual precipitation for the area encompassing the aquifer (Evaluation of Aquifer Performance and Water Capabilities of the Enid Isolated Terrace Aquifer in Garfield County, Oklahoma, OWRB, Douglas C. Kent, Yvan J. Beausoleil and Fred E. Witz, May 1982). By taking 2.5% of the annual average rainfall (28.79 inches), an annual recharge rate of approximately 0.72 inches (0.06 feet) can be established for the Enid wellfields producing from the Cedar Hills Sandstone Aquifer. Recharge from agricultural wells, streams, and the Cimarron River is relatively small in the area and is not included within these calculations. Based on the aereal extent of the water rights, the yearly recharge to the aquifer for the Drummond Wellfield would be equivalent to approximately 1,000 acre-feet of water.

5.4 HYDRAULIC PROPERTIES

The hydraulic properties of an aquifer describe its ability to transmit and store water. These properties include hydraulic conductivity, transmissivity, and specific capacity. The ability of an aquifer to transmit water under a pressure gradient is its hydraulic conductivity. Transmissivity is the rate of flow that water is transmitted through a vertical section of the aquifer that is one foot wide and extending the full saturated thickness of the aquifer under a hydraulic gradient of one (Groundwater and Wells Second Edition, Fletcher G. Driscoll, Ph. D., 1989). Transmissivity is the product of hydraulic conductivity and saturated thickness. The specific capacity of an aquifer is the measure of the aquifer’s yield expressed as a sustainable pumping rate divided by drawdown.

5.4.1 CIMARRON RIVER TERRACE AQUIFER

The hydraulic conductivity and specific yield of the Cimarron Terrace Aquifer has been determined and/or estimated from 23 previously conducted aquifer tests (Geohydrology of Alluvium and Terrace Deposits of the Cimarron River from Freedom to Guthrie, Oklahoma, Water Resources Investigations Report 95-4066, United States Geological Survey, Gregory P. Adams and DeRoy L. Bergman, 1995). The hydraulic conductivity ranges from 1.74×10^{-4} feet per second (ft/s) to 6.3×10^{-3} ft/s with a median hydraulic conductivity value for the alluvium of 2.6×10^{-3} ft/s and for the terrace deposits 1.13×10^{-3} ft/s.

The saturated thickness of the Cimarron Terrace Aquifer ranges from 0 feet at its northeast contact with the Permian-age formations to in excess of 80 feet (Water Supply Capacity Analysis, City of Enid, Envirotech Companies, December 2000). The wide variation of the saturated thickness is directly related to the erosional surface of the underlying Permian-age formations. The average saturated thickness of the aquifer was estimated to be 28 feet (Geohydrology of Alluvium and Terrace Deposits of the Cimarron River from Freedom to Guthrie, Oklahoma, Water Resources Investigations Report 95-4066, United States Geological Survey, Gregory P. Adams and DeRoy L. Bergman, 1995).

The transmissivities derived from these aquifer tests vary from 603 ft^2/day to 10,184 ft^2/day (Geohydrology of Alluvium and Terrace Deposits of the Cimarron River from Freedom to Guthrie, Oklahoma, Water Resources Investigations Report 95-4066, United States Geological Survey, Gregory P. Adams and DeRoy L. Bergman, 1995). The specific capacity/specific yield determined during the aquifer test ranges from 0.0016 to 0.39 with and median specific yield of 0.067.

The hydraulic properties described above for the Cimarron Terrace Aquifer are consistent with values determined in previous studies conducted on the aquifer in 1952 (Bulletin No. 9, Ground Water Resources of the Cimarron Terrace, Division of Water Resources Oklahoma Planning and Resources Board, Edwin W. Reed, Joe L. Hogg, Joseph E. Barclay and George H. Peder, 1952).

The current temporary equal proportionate share established by the OWRB for the Cimarron Terrace Aquifer is 2.0 acre-feet per acre per year. On August 8, 2000 the OWRB issued a tentative order recommending that the equal proportionate share for this aquifer be decreased to 1.0 acre-foot per acre per year. This allocation has not been finalized by the OWRB but is anticipated to become effective within the next few years.

5.4.2 ENID ISOLATED TERRACE AQUIFER

The hydraulic conductivity and transmissivity for the Enid Isolated Terrace Aquifer were generated utilizing indirect methods due to the unavailability of aquifer test data. Information obtained from drillers logs, related to thickness and lithology, and weighting factors for hydraulic conductivity based on grain size were used to calculate a weighted average of the transmissivity and hydraulic conductivity (Evaluation of Aquifer Performance and Water Capabilities of the Enid Isolated Terrace Aquifer in Garfield County, Oklahoma, OWRB, Douglas C. Kent, Yvan J. Beausoleil and Fred E. Witz, May 1982). This same method of approximation was used on four wells completed in the Cimarron Terrace Aquifer with available lithology and aquifer test data. A comparison of the two methods revealed the results to be extremely close. The hydraulic conductivity and transmissivity for the Enid Isolated Terrace Aquifer were computed to be 1.43 x 10^{-3} ft/s and 4,455 ft^2/day, respectively.

The saturated thickness of the Enid Isolated Terrace Aquifer varies from 0 feet at the boundaries of the deposits to a maximum of approximately 55 feet (Water Supply Capacity Analysis, City of Enid, Envirotech Companies, December 2000). The average saturated thickness of the aquifer has been estimated as 36 feet (Evaluation of Aquifer Performance and Water Capabilities of the Enid Isolated Terrace Aquifer in Garfield County, Oklahoma, OWRB, Douglas C. Kent, Yvan J. Beausoleil and Fred E. Witz, May 1982). The specific capacity/specific yield determined for the Enid Isolated Terrace Aquifer is 0.30.

On August 10, 1982, the OWRB established an equal proportionate share from the Enid Isolated Terrace Aquifer of 0.5 acre-feet per acre of land per year. The final order was issued on November 9, 1982.

5.4.3 CEDAR HILLS SANDSTONE AQUIFER

The hydraulic conductivity for the Cedar Hills Sandstone Aquifer varies significantly between the various types of strata within the aquifer. The hydraulic conductivity ranges from 5.79 x 10^{-4} ft/s to 1.16 10^{-9} ft/s with an average hydraulic conductivity of 1.16 x 10^{-5} ft/s (Hydrogeologic Report of the El Reno, Fairview, Isabella, and Loyal Minor Groundwater Basins in Central Oklahoma, Technical Report 2000-1, OWRB, Mark Belden, March 2000).

The saturated thickness of the Cedar Hills Sandstone Aquifer in the area has been estimated to be approximately 52 feet. The specific capacity/specific yield determined for the fine grained sandstone and siltstone strata in the aquifer is 0.05 (Hydrogeologic Report of the El Reno, Fairview, Isabella and Loyal Minor Groundwater Basins in Central Oklahoma, Technical Report 2000-1, OWRB, Mark Belden, March 2000).

The transmissivity for the Cedar Hills Sandstone Aquifer fine grained sandstone and siltstone strata is 52 ft^2/day.

The current temporary equal proportionate share established by the OWRB for the Cedar Hills Sandstone Aquifer is 2.0 acre-feet per acre per year. On May 8, 2001, the OWRB issued a tentative order that the equal proportionate share for the El Reno Minor Groundwater Basin (which includes the Cedar Hills Sandstone Aquifer) be decreased to 1.0 acre-foot per acre per year. This allocation has not been finalized by the OWRB but is anticipated to become effective within the next few years.

5.5 PRODUCTION OF GROUNDWATER

Historically, the City has developed a total of 166 wells within the Ames, Cleo Springs, Drummond, Enid (Plant/Service Center, Van Buren, Northwest, and Carrier) and Ringwood wellfields. Of the total 166 wells, 116 are currently active and supply water to meet the City's water requirements. Thirty-one wells are inactive and the remaining 19 wells have been abandoned (Table 4-4). Production from the City's wellfields from 2000 through 2005 has averaged 3,386 MGY or 9.28 MGD. A review of the production figures for the City's wellfields from 1995 through 2005 revealed the City's maximum production of 3,723 MGY or 10.2 MGD occurred in 2000 (Table 5-1, Figure 5-1). The maximum production capacity of the City's water supply system is approximately 17.1 mgd based on the current reported well pumping rates.

5.5.1 AMES WELLFIELD

The Ames Wellfield is located in the vicinity of Ames approximately 20 miles southeast of Enid. The Ames Wellfield is comprised of 33 wells; however, currently seven of the wells are inactive (Figures 2-1 and 5-2, and Table 4-4). Twenty-nine wells are completed in the Cimarron River Terrace Aquifer and four wells are completed in the underlying Permian Formations. The total depths of these wells vary from 38 feet to 170 feet below ground surface (BGS) with an average depth of approximately 74 feet bgs. The wells completed in the Cedar Hills Aquifer are the deepest wells in the aquifer (143 to 170 feet bgs) and provide the greatest yield, typically 200 gallons per minute (gpm) per well or more.

The City pays a royalty on the water produced from the Ames Wellfield at a current cost of $0.20 per 1,000 gallons of water; therefore, the wells are mainly used in the summer to help meet peak demand and are seldom utilized during the winter.

Drought conditions experienced during the last few years have forced the City to reduce production from three wells (5, 17, and 26) in the Ames Wellfield. The production draw down of the groundwater levels in these wells has exceeded the recharge capabilities causing the wells to cavitate during periods of maximum pumping.

The Water Supply Capacity Analysis conducted in 2000 for the City indicates the total production capacity of the Ames Wellfield as being 2.9 MGD, with individual well capacities ranging from 42 gpm to 412 gpm.

Production figures from 2000 through 2005 indicate the Ames Wellfield has produced an average of 559 MGY (1,716 acre-feet per year) over the five-year period. This is equivalent to an average daily production of 1.53 MGD or an average of 40.91 gpm for each of the 26 active wells.

By comparing the recharge rate established for the Ames Wellfield of 2,623 acre-feet with the average yearly production rate of 1,716 acre-feet per year for the Ames Wellfield, the rate of production did not exceed the annual recharge rate. The average water levels for the Ames Wellfield from 1988 through 2005 are depicted in Table 5-2 and presented graphically on Figure 5-7. There has been a wide fluctuation in the water level for the Ames Wellfield during this timeframe; however, the trend shows an overall decline in the water level. From 2000 to 2005, the water level in the Ames Wellfield declined from 17.3 feet to 20.8 feet. The decline in water levels in excess of the recharge rate may be an indication of production from industrial and agricultural wells in the area of the Ames Wellfied.

5.5.2 CLEO SPRINGS WELLFIELD

The Cleo Springs Wellfield is located approximately 30 miles west of the City near the Town of Cleo Springs. The Cleo Springs Wellfield currently consists of 31 wells of which 28 are active (Figures 2-1 and 5-3, and Table 4-4). All 31 wells in the Cleo Springs Wellfield are completed in the Cimarron Terrace Aquifer. The depths of the wells are relatively shallow and vary from 21 to 82 feet bgs with 7.6 to 45.8 feet of saturated zone. Two of the wells are lateral wells approximately 20 feet bgs.

The Cleo Springs Wellfield is drought sensitive. Drought conditions experienced during the last few years have forced the City to reduce production from four wells (1, 5, 9, and 24) in the Cleo Springs Wellfield. The production draw down of the groundwater levels in these wells has exceeded the recharge capabilities causing the wells to cavitate during periods of maximum pumping.

The Water Supply Capacity Analysis conducted in 2000 for the City indicates the total production capacity of the Cleo Springs Wellfield as being 4.0 MGD, with individual well capacities ranging from 50 gpm to 230 gpm.

Production figures from 2000 through 2005 indicate the Cleo Springs Wellfield has produced an average of 1,010 MGY (3,100 acre-feet per year) over the five-year period. This is equivalent to an average daily production of 2.77 MGD or an average of 68.63 gpm for each of the 28 active wells.

By comparing the recharge rate established for the Cleo Spring Wellfield of 1,427 acre-feet per year with the average yearly production rate of 3,100 acre-feet per year for the Cleo Springs Wellfield, the rate of production exceeded the annual recharge rate. The average water levels for the Cleo Springs Wellfield from 1988 through 2005 are depicted in Table 5-2 and presented graphically on Figure 5-7. There has been a wide fluctuation in the water level for the Cleo Springs Wellfield during this time frame; however, the overall trend shows a decline in the water level. In 1984, when the wells were drilled, the average water level was 16.1 feet. From 2000 to 2005, the water level in the Cleo Springs Wellfield declined from 23.3 feet to 29.9 feet.

5.5.3 DRUMMOND WELLFIELD

The Drummond Wellfield is located approximately 12 miles southwest of the City in the vicinity of Drummond and currently consists of 33 wells of which 20 are active, 10 are inactive, and three are abandoned (Figures 2-1 and 5-6, and Table 4-4). The wells in the Drummond Wellfield are completed in the underlying Cedar Hills Sandstone Aquifer. The depths of the wells vary from 61 feet bgs to 210 feet bgs and have an average total depth of 131 feet bgs.

The City was forced to reduce production from one well (Oklahoma Climatological Survey, www.climate.ocs.ou.edu/coop.php) in the Drummond Wellfield. The production drawdown of the groundwater level in this well has exceeded the recharge capabilities causing the well to cavitate during periods of maximum pumping.

The Water Supply Capacity Analysis conducted in 2000 for Enid indicates the total production capacity of the Drummond Wellfield as being 3.3 MGD, with individual well capacities ranging from 40 gpm to 280 gpm.

Production figures from 2000 through 2005 indicate the Drummond Wellfield has produced an average of 820 MGY (2,518 acre-feet per year) over the five-year period. This is equivalent to an average daily production of 2.25 MGD or an average of 78.01 gpm for each of the 20 active wells.

By comparing the recharge rate established for the Cedar Hills Aquifer of 966 acre-feet per year with the average yearly production rate of 2,518 acre-feet per year for the Drummond Wellfield, the rate of production exceeded the annual recharge rate. The average water levels for the Drummond Wellfield from 1988 through 2005 are depicted in Table 5-2 and presented graphically on figure 5-7. There has been a wide fluctuation in the water level for the Drummond Wellfield during this timeframe; however, the overall trend shows a decline in the water level. From 2000 to 2005, the water level in the Drummond Wellfield declined from 29.2 feet to 43.8 feet.

5.5.4 ENID (PLANT/SERVICE CENTER, VAN BUREN, NORTHWEST, AND CARRIER) WELLFIELD

The Enid (Plant/Service Center, Van Buren, Northwest, and Carrier) Wellfield is located in and northwest of the City. The Enid Wellfield currently consists of 41 wells which are completed in the Enid Isolated Terrace Aquifer (Figures 2-1 and 5-5, and Table 4-4). Twenty-one of the 41 wells are currently active, four are inactive, and 16 are abandoned. The total depths of the wells in the Enid Wellfield vary from 35 feet bgs to 80 feet bgs.

The Water Supply Capacity Analysis conducted in 2000 for the City indicates the total production capacity of the Enid Wellfield as being 1.2 MGD, with individual well capacities ranging from 40 gpm to 120 gpm.

Production figures from 2000 through 2005 indicate the Enid Wellfield has produced an average of 348 MGY (1,066 acre-feet per year) over the five-year period. This is equivalent to an average daily production of 0.953 MGD or an average of 31.53 gpm for each of the 21 active wells.

By comparing the recharge rate established for the Enid Isolated Terrace Aquifer of 1,191 acre-feet per year with the average yearly production rate of 1,066 acre-feet per year for the Enid Wellfield, the rate of production was slightly less than the annual recharge rate. The average water levels for the Enid Wellfield from 1988 through 2005 are depicted in Table 5-2 and presented graphically on Figure 5-7. There has been a wide fluctuation in the water level for the Enid Wellfield during this period; however, the overall trend shows a decline in the water level. From 2000 to 2005, the water level in the Enid Wellfield declined from 25.8 feet to 29.9 feet. The decline in water levels in excess of the recharge rate may be an indication of production from industrial and agricultural wells in the area of the Enid Wellfield.

5.5.5 RINGWOOD WELLFIELD

The Ringwood Wellfield is located approximately 25 miles southwest of the City and southwest of Ringwood. The Ringwood Wellfield is currently comprised of 28 wells of which 21 are active and seven are inactive (Figures 2-1, 5-4, and Table 4-4). All wells in the Ringwood Wellfield are completed in the Cimarron Terrace Aquifer. The depths of the wells in the Ringwood Wellfield vary from 45 feet bgs to 83 feet bgs.

The Ringwood Wellfield is drought sensitive. Drought conditions experienced during the last few years have forced the City to reduce production from seven wells (6, 15, 16, and 22 through 25) in the Ringwood Wellfield. The production draw down of the groundwater level in these wells has exceeded the recharge capabilities causing the wells to cavitate during periods of maximum pumping.

The Water Supply Capacity Analysis conducted in 2000 for the City indicates the total production capacity of the Ringwood Wellfield as being 2.8 MGD, with individual well capacities ranging from 43 gpm to 167gpm.

Production figures from 2000 through 2005 indicate the Ringwood Wellfield has produced an average of 649 MGY (1,991 acre-feet per year) over the five-year period. This is equivalent to an average daily production of 1.235 MGD or an average of 58.8 gpm for each of the 21 active wells.

By comparing the recharge rate established for the Ringwood Wellfield of 2,015 acre-feet per year with the average yearly production rate of 1,991 acre-feet per year for the Ringwood Wellfield, the rate of production did not exceed the annual recharge rate. The average water levels for the Ringwood Wellfield from 1988 through 2005 are depicted in Table 5-2 and presented graphically on Figure 5-7. There has been a wide fluctuation in the water level for the Ringwood Wellfield during this time frame; however, the overall trend shows a decline in the water level. In 1984, when the wells were drilled, the average water level was 27.5 feet. From 2000 to 2005, the water level in the Ringwood Wellfield declined from 33.6 feet to 38.6 feet. The decline in water levels in excess of the recharge rate may be an indication of production from industrial and agricultural wells in the area of the Ringwood Wellfield.

A comparison of the average yearly production rates and recharge rate stated above for each wellfield is given in Table 5-3.

5.6 FUTURE AVAILABILITY OF GROUNDWATER

Based upon the data presented in the previous section, the total average yearly production of groundwater during the period 2000 to 2005 (10,391 acre-feet per year, or 3,386 MGY) exceeded the total average annual recharge rate (8,222 acre-feet per year, or 2,679 MGY). As the production rate exceeded the recharge rate, the wellfields are currently being 'mined' of water. If this 'mining' continues, the water levels in the aquifers will continue to decline to such an extent that the production yield from the aquifers will decrease, as wells will be progressively pumped 'dry'. This will be due to the cone of depression ('drawdown') in water level local to each pump reaching the pump intakes, with shallower wells being pumped 'dry' first. At this point, production would have to be severely curtailed to allow the aquifers to recover through recharge. The time frame in which all of the wellfields would be pumped 'dry' can be estimated from the volume of usable water stored in the wellfields, the projected future production rates and the recharge rate.

From the data presented in Section 5.3 (which is derived from Table 4-4), the total estimated volume of water stored in the wellfields at the time of water table readings is 444,299 acre-feet, or 144,775 MG. It should be noted that the wellfield acreages as stated in Section 5.3 are greater than the City's total water rights acreage (See Section 6.2.1). This is because the wellfield areas are based upon the area encompassing the wells within a wellfield, which represents the zone of influence on the aquifer for each wellfield, but the City does not own all of the water rights within the wellfield areas.

As the water table readings for each wellfield were taken at various times throughout 2004 and 2005, the estimated volume of water was normalized to the beginning of 2006 by taking into the account the production of each wellfield from the time of the readings, and by pro-rating the recharge rate. This normalization reduced the total estimated volume of water in the wellfields to 439,982 acre-feet, or 143,369 MG. Table 5-4 provides a summary on the estimation of the volume of total water stored in the wellfields at the beginning of 2006.

Not all of the total water stored in the wellfields will be able to be pumped from the wellfields. According to City staff, the intakes to the well pumps are located approximately 10 feet off the bottom of the aquifers. Therefore the wells will 'run dry' before the water table reaches the bottom of the aquifer. The usable water stored in each wellfield is therefore taken as its total aquifer saturated thickness minus the bottom 10 feet. In making this adjustment, the estimated usable water stored in the wellfields at the beginning of 2006 is 340,754 acre-feet, or 111,035 MG. This represents approximately 77% of the total stored water. Table 5-5 provides a summary on the estimation of the volume of usable water stored in the wellfields at the beginning of 2006.

Future production rates were calculated for each year from 2006 to 2050 using the projected water demands given in Table 8-2. For each year within this period, the projected yearly demand was subtracted from the annual recharge rate to produce the net annual water deficit (i.e. the net loss in usable water stored in the wellfields). Each annual deficit was then subtracted from the usable water stored at the start of each year to produce the usable water stored at the start of the following year. To ensure that each wellfield would 'run dry' in the same year, the net annual water deficit was assigned to each wellfield in proportion to its percentage of the total usable water stored (See Table 5-5).

A plot of the yearly water demand, recharge rate and usable water stored in the wellfields for the period 2006 to 2050 is shown in Figure 5-8. This plot shows that at the projected future water demands, the wellfields are projected to 'run dry' of usable water in 2042. By 2050, the water deficit is projected to be 12,438 acre-feet per year, or 4,054 MGY. This represents the volume of water required to be supplied from alternative sources to allow water to be abstracted from the wellfields at a sustainable rate (i.e. to prevent 'mining').

The above projection is based upon the recharge rate given for each aquifer in the publications referenced in Section 5.3. A more accurate representation of recharge rates can be estimated from comparing past production data with historic water table levels. A change in water table over a year represents the difference between wellfield production and recharge rate. As both the saturated thickness and total water stored in each wellfield is known, the amount of water stored per foot of saturated thickness can be calculated for each wellfield (See Table 5-5). Therefore a given change in water table level in feet can be related to the net change in the total water stored for that wellfield. As the annual wellfield production is known, the recharge rate can be calculated by comparing the difference between production and the net change in total water stored, via the change in water table level over the year.

Table 5-1 provides annual production data for the wellfields during the period 1995 to 2005. The corresponding annual changes in water table levels are provided in Table 5-2. For this period, the total production was compared to the net change in water table level for each wellfield, and the recharge rate was calculated so that the difference between production and recharge matched the calculated change in total water stored from the water level changes. A comparison of the published and estimated annual recharge rates are given in Table 5-6. This shows that:

- The estimated recharge rates for the Ames and Enid wellfields were lower than the published recharge rates.
- The estimated recharge rate for the Ringwood Wellfield was almost identical to the published recharge rate.
- The estimated recharge rate for the Cleo Springs Wellfield was higher than the published recharge rate.
- The Drummond Wellfield had a 'negative' recharge rate i.e. assuming no recharge took place, the wellfield 'lost' more water than was produced.

The Drummond Wellfield was subject to some large water level fluctuations (over 7 ft per year) when compared to the other wellfields. The water level data used for the net change over the period is the annual average water table, and is determined from all of the readings taken from individual wells. In some instances, a well water level reading was taken only once per year, and not every well was recorded every year. As the Drummond Wellfield has the greatest range of well depths of the wellfields, it is possible that the lack of data from some wells in certain years has skewed the data. In addition, the water level would be different if a once-yearly reading was taken when the well was in production as opposed to when the well had not produced for some time.

Due to these factors, the estimated recharge rate for the Drummond Wellfield was taken to be the published recharge rate (966 acre-feet per year, or 315 MGY). From this assumption and the estimated recharge rates given in Table 5-6, the total annual average recharge rate over this period was calculated to be 8,719 acre-feet per year, or 2,360 MGY. This represents a 12% decrease in the total annual average recharge rate when compared with the published data.

Using this revised recharge rate, and using the same procedure as described earlier, a plot of the yearly water demand, recharge rate and usable water stored in the aquifers for the period 2006 to 2050 was produced (Figure 5-9). This plot shows that at the projected future water demands, the wellfields are projected to 'run dry' of usable water in 2040, 2 years earlier when compared with the first set of calculations. By 2050, the water deficit is projected to be 13,419 acre-feet per year, or 4,373 MGY.

Both projections shown in Figures 5-8 and 5-9 assume that future demand will be apportioned out to each wellfield in proportion to the amount of usable water stored in each wellfield, so that all the wellfields will 'run dry' simultaneously. However, the current percentage of production met by each wellfield does not match the percentage of usable water stored in each wellfield. Table 5-7 presents the production data for each wellfield given in Table 5-1 as a percentage of the total production, and compares it to the percentage of total usable water stored in each wellfield, as given in Table 5-5. This shows that the production from the Cleo Springs and Ringwood wellfields (28% and 19% respectively) is far greater than the usable water stored in them (5% and 7% respectively). Conversely, the Drummond Wellfield contains 60% of the total usable water, but accounts for only 25% of the total production. The Ames and Enid wellfields have similar percentages of production and total usable water. If future demand is apportioned out to each wellfield in proportion to percentage production of each wellfield, then it is clear that some wellfields (Cleo Springs and Ringwood) will 'run dry' before others (Drummond).

Based upon the above percentage production rates, the time frame in which each wellfield would 'run dry' was calculated using the projected future demands with both the published and estimated recharge rates, the results of which are given in Table 5-8. This shows that the Cleo Springs Wellfield would be the first wellfield to 'run dry' (2015), followed by the Ringwood Wellfield (2021-2022), the Enid Wellfield (2034-2036) and then the Ames Wellfield (2042-2045). In both scenarios, the Drummond Wellfield is projected not to 'run dry' prior to 2050. In these calculations, it was assumed that the percentage of total production remained constant, so that when a wellfield 'ran dry', the lost production from that wellfield was made up from an alternative water supply source, and not made up by increasing production in the remaining wellfields. If lost production was made up in this manner, then the time frame in which the remaining wellfields would 'run dry' would be shortened.

To summarize all of the above discussion, the calculations indicate that to extend the life of the wellfields, the City must switch production from the Cleo Springs and Ringwood wellfields to the Drummond Wellfield, such the percentage production from each wellfield is more in line with its percentage of usable water stored. Even with these changes however, the calculations indicate that all of the City's wellfields could 'run dry' within the next 35 years if the projected growth in demand is realized. They indicate that demand will continue to outstrip recharge and that the City will have to find additional water. This could be achieved by expanding the current wellfields through the purchase of additional water rights, or through alternative water supply sources (See Section 5.8)

5.7 WATER QUALITY

The chemical quality of groundwater, in general, is a reflection of the chemical composition of the rocks with which it comes in contact. As water passes through the soil and rocks, it takes minerals into solution. The concentrations of the minerals in the groundwater are dependent on their solubility and duration of contact. Some other factors that could impact the groundwater quality include problems associated with oil and gas production activities, industrial activities, poorly constructed sanitary landfills and agricultural chemicals.

The City monitors their wellfields on a regular basis as required by the Oklahoma Department of Environmental Quality (ODEQ). Three chemicals of concern (COC) have been encountered in the groundwater of the City's wellfields. These COCs include total dissolved solids (TDS), nitrates, and chlorides. The concentrations of each of the COCs are depicted by each well location on Figures 5-2 through 5-6 and in Tables 5-9 through 5-23.

TDS is basically the total concentration of dissolved minerals in water and is determined from the weight of dry residue remaining after a sample of water has evaporated. Water with high levels of TDS will have a distinctive taste and can be corrosive and/or cause deposits within the water supply system. TDS is listed as a secondary drinking water contaminant by the Environmental Protection Agency (EPA) and has a Recommended Maximum Limit (RML) of 500 milligrams per liter (mg/L). Secondary contaminations are those that cause cosmetic effects such as skin or tooth discoloration or aesthetic effects such as taste, odor, or color in drinking water.

Nitrate, unlike other elements in groundwater, is not derived primarily from the minerals in the rocks. Sources of nitrogen in the soil include certain plants (legumes), decomposing plant debris, animal waste, nitrate fertilizers, and sewage discharges and lagoons. High nitrate concentrations in water supplies can cause a toxic effect on young infants. Methemoglobinemia (blue baby syndrome) is a disease caused by nitrates which converts to nitrites in the intestines, resulting in an overabundance of methemoglobin molecules. Symptoms of the disease include listlessness and a bluish tinge to the skin. The National Primary Drinking Water Standards established by the EPA depict nitrate as a primary contamination and has set a Maximum Contaminate Level (MCL) of 10 mg/L.

Chloride is derived from minerals such as halite, ancient sea water trapped in sedimentary rocks when they were formed, evaporites, human waste, animal waste, oil and gas production activities, and industrial waste. The National Secondary Drinking Water Standards established by the EPA depict chloride as a secondary contamination and has set an RML of 250 mg/L.

5.7.1 AMES WELLFIED

Table 5-9 is a summary of the laboratory analysis conducted for TDS from 1999 through 2005 for the Ames Wellfield. The annual averages for TDS concentrations are also presented graphically for each wellfield on Figure 5-10. The information contained in Table 5-9 and Figure 5-10 indicates that the average TDS concentration contained in the water produced from the Ames Wellfield has shown an overall increase over the seven-year period and from 437 mg/L in 2003 to 526 mg/L in 2005, a 20.4% increase. The 2005 level exceeds the EPA RML of 500 mg/L. Table 5-9 indicates that the most current laboratory analysis for each of the wells in the Ames Wellfield showed that nine of the wells (1, 2, 4, 6, 8, 9, 10, 12, 14, and 19) contained concentrations of TDS in excess of the EPA RML.

Table 5-14 is a summary of the laboratory analysis for nitrate from 1996 through 2005 for the Ames Wellfield. The annual averages for nitrate concentrations are also presented graphically for each wellfield on Figure 5-11. The information contained in Table 5-14 and Figure 5-11 indicates that the average nitrate concentration contained in the water produced from the Ames Wellfield has shown an overall increase over the 10-year period; however, there has been a decrease in nitrate concentration from 9.3 mg/L in 2002 to 8.6 mg/L in 2005, a 7.5% decrease. Table 5-14 indicates that the most current laboratory analysis for each of the wells in the Ames Wellfield showed that nine of the wells (1, 11, 13, 14, and 16, through 19, 21, and 25) contained concentrations of nitrate in excess of the EPA MCL.

Table 5-19 is a summary of the laboratory analysis conducted for chloride from 1996 through 2005 for the Ames Wellfield. The annual averages for chloride concentrations are also presented graphically for each wellfield on Figure 5-12. The information contained in Table 5-19 and Figure 5-12 indicates that the average chloride concentration contained in the water produced from the Ames Wellfield has shown an overall increase over the 10-year period; however, there has been a decrease in chloride concentration from 136 mg/L in 2002 to 104 mg/L in 2005, a 23.5% decrease. Table 5-19 indicates that the most current laboratory analysis for each of the wells in the Ames Wellfield showed that one of the wells (6) contained concentrations of chloride in excess of the EPA RML.

5.7.2 CLEO SPRINGS WELLFIELD

Table 5-10 is a summary of the laboratory analysis conducted for TDS from 1999 through 2005 for the Cleo Springs Wellfield. The information contained in Table 5-10 and Figure 5-10 indicates that the average TDS concentration contained in the water produced from the Cleo Springs Wellfield has shown an overall increase over the seven-year period and increased from 326 mg/L in 2003 to 434 mg/L in 2005, a 33.1% increase. Table 5-10 indicates that the most current laboratory analysis for each of the wells in the Cleo Springs Wellfield showed that 15 of the wells (1, 2, 3, 5, 6, 8, 18, and 23 through 30) contained concentrations of TDS in excess of the EPA RML.

Table 5-15 is a summary of the laboratory analysis conducted for nitrate from 1996 through 2005 for the Cleo Springs Wellfield. The information contained in Table 5-15 and Figure 5-11 indicates that the average nitrate concentration contained in the water produced from the Cleo Springs Wellfield has shown an overall increase over the 10-year period; however, there has been a decrease in nitrate concentration from 5.6 mg/L in 2002 to 5.3 mg/L in 2005, a 5.4% decrease. Table 5-15 indicates that the most current laboratory analysis for each of the wells in the Cleo Springs Wellfield showed that three of the wells (18, 20, and 21) contained concentrations of nitrate in excess of the EPA MCL.

Table 5-20 is a summary of the laboratory analysis conducted for chloride from 1996 through 2005 for the Cleo Springs Wellfield. The information contained in Table 5-20 and Figure 5-12 indicates that the average chloride concentration contained in the water produced from the Cleo Springs Wellfield has shown an overall decrease over the 10-year period and a decrease in chloride concentration from 99 mg/L in 2001 to 56 mg/L in 2005, a 43.4% decrease.

5.7.3 DRUMMOND WELLFIELD

Table 5-11 is a summary of the laboratory analysis conducted for TDS from 1999 through 2005 for the Drummond Wellfield. The information contained in Table 5-11 and Figure 5-10 indicates that the average TDS concentration contained in the water produced from the Drummond Wellfield has shown an overall increase over the seven-year period and increased from 436 mg/L in 2003 to 541 mg/L in 2005, a 24% increase. The 2005 level exceeds the EPA RML of 500 mg/L. Table 5-11 indicates that the most current laboratory analysis for each of the wells in the Drummond Wellfield showed that 11 of the wells (1, 5, 6, 9, 10, 19, 20, 21, 28, 29, and 33) contained concentrations of TDS in excess of the EPA RML.

Table 5-16 is a summary of the laboratory analysis conducted for nitrate from 1996 through 2005 for the Drummond Wellfield. The information contained in Table 5-16 and Figure 5-11 indicates that the average nitrate concentration contained in the water produced from the Drummond Wellfield has shown an overall increase over the 10-year period; however, there has been a decrease in nitrate concentration from 8.0 mg/L in 2002 to 7.5 mg/L in 2005, a 6.3% decrease. Table 5-16 indicates that the most current laboratory analysis for each of the wells in the Drummond Wellfield showed that four of the wells (5, 7, 8, and 23) contained concentrations of nitrate in excess of the EPA MCL.

Table 5-21 is a summary of the laboratory analysis conducted for chloride from 1996 through 2005 for the Drummond Wellfield. The information contained in Table 5-21 and Figure 5-12 indicates that the average chloride concentration contained in the water produced from the Drummond Wellfield has shown an overall increase over the 10-year period; however, there has been a decrease in chloride concentration from 153.0 mg/L in 2001 to 109 mg/L in 2005, a 28.8% decrease. Table 5-21 indicates that the most current laboratory analysis for each of the wells in the Drummond Wellfield showed that one of the wells (10) contained concentrations of chloride in excess of the EPA RML.

5.7.4 ENID (PLANT/SERVICE CENTER, VAN BUREN, NORTHWEST, AND CARRIER) WELLFIELD

Table 5-12 is a summary of the laboratory analysis conducted for TDS from 1999 through 2005 for the Enid Wellfield. The information contained in Table 5-12 and Figure 5-10 indicates that the average TDS concentration contained in the water produced from the Enid Wellfield has shown an overall increase over the seven-year period and increased from 281 mg/L in 2003 to 333 mg/L in 2005, an 18.5% increase.

Table 5-17 is a summary of the laboratory analysis conducted for nitrate from 1996 through 2005 for the Enid Wellfield. The information contained in Table 5-17 and Figure 5-11 indicates that the average nitrate concentration contained in the water produced from the Enid Wellfield has shown an overall increase over the 10-year period; however, the nitrate concentration has remained relatively consistent from 9.8 mg/L in 2002 to 9.7 mg/L in 2005. Table 5-17 indicates that the most current laboratory analysis for each of the wells in the Enid Wellfield showed that seven of the wells (Van Buren 3; Northwest 2, 3, and 6 through 8; and Carrier 12) contained concentrations of nitrate in excess of the EPA MCL.

Table 5-22 is a summary of the laboratory analysis conducted for chloride from 1996 through 2005 for the Enid Wellfield. The information contained in Table 5-22 and Figure 5-12 indicates that the average chloride concentration contained in the water produced from the Enid Wellfield has shown an overall increase over the 10-year period; however, there has been a decrease in chloride concentration from 90 mg/L in 2002 to 54 mg/L in 2005, a 40% decrease.

5.7.5 RINGWOOD WELLFIELD

Table 5-13 is a summary of the laboratory analysis conducted for TDS from 1999 through 2005 for the Ringwood Wellfield. The information contained in Table 5-13 and Figure 5-10 indicates that the average TDS concentration contained in the water produced from the Ringwood Wellfield has shown an overall increase over the seven-year period and increased from 326 mg/L in 2003 to 434 mg/L in 2005, a 33.1% increase. Table 5-13 indicates that the most current laboratory analysis for each of the wells in the Ringwood Wellfield showed that six of the wells (14, 21, and 25 through 28) contained concentrations of TDS in excess of the EPA RML.

Table 5-18 is a summary of the laboratory analysis conducted for nitrate from 1996 through 2005 for the Ringwood Wellfield. The information contained in Table 5-18 and Figure 5-11 indicates that the average nitrate concentration contained in the water produced from the Ringwood Wellfield has shown an overall increase over the 10-year period and increased from 6.9 mg/L in 2000 to 8.2 mg/L in 2005, a 13% increase. Table 5-18 indicates that the most current laboratory analysis for each of the wells in the Ringwood Wellfield showed that five of the wells (14, 20, 21, 26, and 28) contained concentrations of nitrate in excess of the EPA MCL.

Table 5-23 is a summary of the laboratory analysis conducted for chloride from 1996 through 2005 for the Ringwood Wellfield. The information contained in Table 5-23 and Figure 5-12 indicates that the average chloride concentration contained in the water produced from the Ringwood Wellfield has shown an overall increase over the 10-year period; however, there has been a decrease in chloride concentration from 96 mg/L in 2001 to 60 mg/L in 2005, a 37.5% decrease.

5.8 <u>ALTERNATIVE WATER SUPPLY SOURCES</u>

5.8.1 ALTERNATIVE WATER SUPPLY SOURCE STUDY (OSU, 2003)

A study was conducted in 2003 by A.K. Tyagi, PhD, PE, to research the quantity and quality of alternative sources of surface water and ground water for the City (Alternative Water Sources for Enid, City of Enid, Oklahoma Infrastructure Consortium School of Civil and Environmental Engineering Oklahoma State University, March 2003). This study evaluated the existing Ames, Cleo Springs, Drummond, Enid, and Ringwood wellfields and existing surface water reservoirs including Canton, Great Salt Plains, Kaw, and Pawnee lakes. The study concluded that the City's existing five wellfields provides an adequate source of water to meet the City's average water demands. The study further concluded that the City should continue holding the water supply allocation of surface water from Kaw Lake as a contingency plan.

Based on the information included within the study and the considerable amount of money that has been invested in building and maintaining the groundwater infrastructure, the OSU report concluded that groundwater was the best option to serve as the continuing main water resource.

5.8.2 KAW LAKE

Kaw Lake is the most readily available alternative source of water supply in this region for the City. A review of information obtained from the OWRB revealed that the City previously held Stream Water Permit # 1981-180 in the amount of 25,000 acre-feet per year until it authorized the OWRB to assign and transfer the same water right to the Kaw Reservoir Authority (KRA) on July 3, 2001. As a result of that request, a review of the historical water use under the permit was triggered and the OWRB determined that some forfeiture of water under the permit had occurred due to non-use. As a result, the permit was officially transferred to the KRA on August 27, 2002 in the reduced amount of 14,159 acre-feet per year, a reduction of 56.6%.

Kaw Lake has a dependable water supply pool yield of 187,040 acre-feet per year. Currently, there have been nine water rights issued in the water supply pool for a total of 141,403 acre-feet per year. This leaves 45,637 acre-feet per year currently available for appropriation. There are three pending applications with the OWRB totaling 64,050 acre feet per year. One of the pending applications is from the KRA for 35,600 acre-feet per year. They have not yet been able to show a present and future need for this amount of water as required by the Oklahoma Stream Water Law.

Options for the City to secure water from Kaw Lake include:

- Join/contract with KRA to obtain water under their existing 1981-0180 (the old Enid) permit;
- Apply for a new water right from Kaw which will require showing a present or future need (including a schedule of use) – there is a process to move up past existing pending applications; and
- Approach other existing permit holders to determine if they would contract or assign water under their existing permit to the City.

Either directly or indirectly, these options would also require payment to the US Army Corps of Engineers for storage costs allocated to the water supply pool.

Should the City choose to pursue evaluating the Kaw Lake option, the study should consider opportunities for regionalization partnerships. Currently, public water supply entities in Woods, Alfalfa and Grant Counties are partnering with six southern Kansas counties in exploring regional water supply opportunities under the Sunflower H2O Initiative. Clearly, a Kaw Lake transmission line to Enid is in close proximity to the Sunflower project area and may be worth exploring as a partnering opportunity. Additionally, the OWRB is encouraging regional water planning studies as part of the Update of the Oklahoma Comprehensive Water Plan and might be a potential funding source for exploring this option.

5.8.3 NATURAL RESOURCES CONSERVATION SERVICE (NRCS) WATERSHED DAMS

As part of its upstream watershed program, the Natural Resources Conservation Service has constructed more than 2,100 flood control/water supply structures in Oklahoma under four different Congressional authorizations. These projects represent a $2 Billion investment that provides over $72 Million in annual benefits. Recently, Congress passed the Small Watershed

Rehabilitation Act of 2000 which authorized the NRCS to provide technical and financial assistance to watershed project sponsors in rehabilitating their aging dams to extend their service life.

In recent years increased emphasis has been placed on the development of multipurpose lakes constructed for floodwater detention. In addition to widespread recreational use of sediment pools of watershed structures, local sponsors have added storage for municipal, irrigation, recreation and fish and wildlife purposes. These multipurpose lakes foster economic growth in cities, towns and rural areas by providing dependable water supplies and recreational areas attractive to residents and tourists.

In the Enid area, two watersheds that have been the subject of NRCS planning and construction are Upper Red Rock Creek to the east and Turkey Creek to the west. Upper Red Rock Creek Watershed has had 43 of its 56 currently identified sites constructed while Turkey Creek Watershed has had only 1 of its 11 currently identified sites completed.

5.8.3.1 Upper Red Rock Creek Watershed

Opportunities to develop a multipurpose water supply site in the Upper Red Rock Creek Watershed will be challenging as a majority of the dams have been constructed (43 out of 56) and configured for flood control only. While there may be some suitable sites to evaluate for multipurpose water supply, implementation may require removal or operation of existing sites in a dry detention mode to allow inflows into the new development site. Watershed work plan sites remaining for construction typically have relatively small drainage areas of less than 10 square miles, with the largest being just under 14 square miles. An appraisal level dependable yield analysis of any prospective site should clearly be one of the early work elements for project screening in this watershed. Close coordination with NRCS planning staff in evaluating potential multipurpose sites in the Upper Red Rock Watershed would be needed particularly in light of options that might impact existing watershed structures.

5.8.3.2 Turkey Creek Watershed

Only one Turkey Creek Watershed dam (Site # 10) has been completed which provides more flexibility in evaluating opportunities for multipurpose site development. While the remaining sites in the watershed work plan are not multipurpose design sites, there could be some opportunity for site repositioning for water supply development. Three of the prospective sites have more substantial drainage areas of 11, 15 and 27 square miles, as currently configured in the watershed work plan. Potential sites may be located just to the southwest of Goltry and also south of Enid between Waukomis and Hennessey. An appraisal level dependable yield analysis of any prospective site should clearly be one of the early work elements for project screening in this watershed. Close coordination with NRCS planning staff in evaluating potential multipurpose sites in the Turkey Creek Watershed appears to be in order.

5.8.4 ADDITIONAL GROUNDWATER DEVELOPMENT

While developing additional groundwater well fields is an option that could be explored, it should be considered in light of basin studies underway and public policy discussions that are ongoing. As discussed in Section 5.4.1, the OWRB staff recommended a tentative order that the equal proportionate share allocation for the Cimarron Terrace be decreased to 1.0 acre-foot per acre per year. If adopted, all existing temporary permits in this basin would be reduced from 2.0 acre-feet per acre per year to the regular permit allocation of 1.0 acre-foot per acre per year. In the case of the Enid Isolated Terrace, the final equal proportionate share has been set at a much reduced rate of 0.5 acre-feet per acre per year. In the case of the Cedar Hills Sandstone (El Reno Minor Basin), the OWRB has approved a tentative order recommending 1.0 acre-foot per acre per year allocation.

Additionally, consideration should be given to the discussion of the future of groundwater management ongoing in the Update of the Oklahoma Comprehensive Water Plan. Through the OWRB's public participation process, recommendations are being voiced to transition the Oklahoma Groundwater Law from the current "mining" concept to a more "sustainable" management scheme. If one considers it on its face, such a sustainable concept might be based on an allocation that would be something close to or less than recharge. If such a hypothesis is applied to the subject groundwater basins, new "sustainable" equal proportionate shares could be established. Based on the recharge rates estimated in Section 5.3, the "sustainable" equal proportionate shares could be in the range of 0.20 acre-feet per acre per year in the Cimarron Terrace Aquifer, 0.18 acre-feet per acre per year in the Enid Isolated Terrace Aquifer and 0.06 acre-feet per acre per year in the Cedar Hills Sandstone Aquifer. Such a transitioning of the current Groundwater Law to a more sustainable basis clearly has a significant impact on future well field development. It should also be noted that the Oklahoma Legislature has been debating introduced legislation relating to changes in the Groundwater Law and has Interim Legislative Studies active on the subject.

As noted in Section 5.6, the water deficit (the volume of water required to be supplied from alternative sources to allow water to be abstracted from the current wellfields at a sustainable rate) is projected to be 12,438 acre-feet per year by 2050, based upon the published recharge rate. Additional water to meet this water deficit could be provided by developing an additional wellfield in the Cimarron Terrace Aquifer, as this is the largest aquifer. Based upon a sustainable production rate of 0.20 acre-feet per acre per year for this aquifer, the City would need to acquire approximately 61,000 acres (or 95 square miles) of water rights for the new wellfield. This area is in excess of the combined area of the existing wellfields (52,500 acres).

5.8.5 WATER RIGHTS

5.8.5.1 Stream Water Rights

To obtain a permit to use surface water, an applicant must meet the requirements of the Oklahoma Stream Water Law and the regulatory provisions of the OWRB (OAC 785:20). Before taking final action on an application, the OWRB must determine from the evidence presented in the application and hearing (if required) whether:

- Unappropriated water is available in the amount applied for;
- The applicant has a present or future need for the water and the use to which the applicant intends to put the water is a beneficial use. In making this determination, the Board will consider the availability of all stream water sources and such other relevant matters as they

deem appropriate, and may consider the availability of groundwater as an alternative source;
- The proposed use does not interfere with domestic or existing appropriative uses; and
- If the application is for the transportation of water for use outside the stream system of origin, in-basin uses must be protected first.

5.8.5.2 Groundwater Rights

To obtain a groundwater permit, an applicant must meet the requirements of the Oklahoma Groundwater Law and the regulatory provisions of the OWRB (OAC 785:30). Before taking final action on an application, the OWRB must determine from the evidence presented in the application and hearing (if required) whether:

- The applicant owns or has a valid lease to the dedicated land;
- The land overlies a fresh groundwater basin or sub-basin;
- The applicant's intended use for the water is a beneficial use; and
- Waste by depletion and waste by pollution will not occur.

5.8.6 COMPARISON OF ALTERNATIVE WATER SUPPLY SOURCES

At some point in the near future, the City will have to commit to the development of an alternative water supply source in order to meet the projected water deficit of 4,373 MGY by the year 2050 (See Section 5.6). Detailed feasibility studies will be required to determine the most appropriate solution to meet the City's future water requirements. To aid the City in deciding which supply source(s) warrant such study, a high-level cost estimate has been developed to demonstrate to the City the likely magnitude of funding required to develop each identified water supply source. The developed costs are the estimated present worth value to construct and operate a 4,373 MGY (12 MGD) water supply over a period of 50 years.

Costs have been developed for four water supply options. A further two water supply options were considered but were not costed. Details of the six water supply options are given in the following sections.

5.8.6.1 Kaw Lake

The costs for this option have been based upon the installation of a 73-mile long, 48-inch diameter pipeline from Kaw Lake to the City. The pipe has been sized to accommodate a peak day flow of 24 MGD i.e. twice the projected average day water deficit. The total projected 2050 peak day flow is 33.4 MGD; therefore the remaining 9.4 MGD will be supplied by the existing wellfields. As discussed in Section 5.8.2, the opportunity exists to develop a regional water supply, but this has not been considered at this stage; therefore the pipeline has been sized to meet the City's requirements only.

A potential pipeline route is shown in Figure 5-13, Sheets 1 to 8. From Kaw Lake, the route heads west, following the north bank of the Arkansas River into Ponca City. From there, the route continues west, running alongside to U.S. Highway 60 to Lamont, bypassing Tonkawa and crossing I-35 on the way. At Lamont, the route turns south alongside State Highway 74 to

U.S. Highway 412, passing through Salt Fork and Garber. The route turns west to run alongside the highway and then the Atchison, Topeka and Santa Fe Railroad, to terminate in the west end of the City at a new 24 MGD water treatment plant (WTP).

The estimated construction cost assumes that the pipe material is concrete pressure pipe. In order to keep pressures within the pipeline below the pressure rating of the pipe, more than one pumping station will be required: one at Kaw Lake, and probably one somewhere between Salt Fork and Garber. The estimated construction cost for the WTP assumes that the pipeline will terminate at a concrete storage tank, from which the water will be drawn for treatment. The construction cost also includes new distribution pumps.

Estimated operating costs are based upon the following items: the cost to pump 4,373 MGY from Kaw Lake to the City over a 50-year period; the cost to treat the water over the same period; and the cost to replace the raw water pumps at Year 25. As the purpose of the exercise is to compare supply options, the cost of distribution pumping and the cost of replacing distribution pumps has not been considered.

5.8.6.2 Hennessey Lake

The Tulsa District Corps of Engineers (COE) have carried out some preliminary studies with regards to a potential dam site located on Turkey Creek. The dam site is located 5 miles west of Bison and is referred to in COE documents as the Hennessey Dam. One document states that the reliable maximum water supply yield from the watershed at this site is 16.8 MGD (6,132 MGY); however, using information contained within the document to calculate supply yield from watershed area and surface runoff, the calculated water supply yield is more nearer 20 MGD (7,300 MGY). The site therefore has the potential to supply plenty of water in excess of the City's projected 2050 water deficit.

The COE document provides a top of dam elevation of 1194.0 feet, a maximum pool elevation of 1188.5 feet, a top of flood control pool elevation of 1175.8 feet and a conservation pool elevation of 1166.7 feet. However, at the maximum pool elevation stated (which would only occur in the most extreme case e.g. a 200-Year flood), the Burlington Northern railroad would be inundated for some distance as it crosses Turkey Creek north east of Drummond.

As the dam site is oversized for the City's water requirements, it was decided to estimate a cost to construct a dam with a lower elevation, such that the maximum pool elevation would not affect the railroad. Therefore a top of dam elevation of 1185.0 feet and a maximum pool elevation of 1180.0 feet were selected, with the conservation pool elevation remaining the same. The effect of this change is that the dam will have lost virtually all of its flood storage, and so the dam would be a water supply only dam and not a multi-purpose dam. The U.S. government has a goal to provide multi-purpose dams, and so the dam would have to be taken out of the federal dam building program ("de-federalized").

Figure 5-14 shows the location of the dam, and the maximum and conservation pool elevation contours. The dam would be approximately 12,730 feet long with a maximum height of approximately 90 feet. The conservation pool elevation would cover an area of approximately

6,750 acres, and the maximum pool elevation would cover an area of approximately 10,500 acres. The construction costs assume:

- An earthen dam with a concrete, gated spillway to control the release of water
- An intake tower for water supply, and
- An emergency spillway cut into one of the dam's abutments to ensure that the dam is never over-topped during extreme flooding events.

Figure 5-15, Sheets 1 to 3 shows a potential route of a 16-mile long, 48-inch diameter pipeline to convey a peak flow of 24 MGD from the lake to the City's Plant #2. From a pumping station, the route of the pipeline would head east for approximately 1.5 miles to a county road. From there, the route would turn north and run alongside the county road (which turns into Garland Road) for 14 miles into the City. At Chestnut Avenue, the route would turn west for 0.5 mile to terminate at Plant #2.

The estimated construction cost assumes that that the pipe material is concrete pressure pipe, and that Plant #2 would be expanded to meet the projected 2050 demand (distribution pumps) and to include facilities to treat surface water. The dam construction cost includes an estimate to purchase land that would be covered by water when it is at the maximum pool elevation.

Operating costs have been estimated on the same basis as the Kaw Lake option. It has been assumed that water arriving at Plant #2 from the Cleo Springs and Ringwood wellfields will not be treated, but merely blended with the treated surface water prior to dosing with chlorine and hydrofluorosilic acid.

5.8.6.3 Lahoma Lake

As the Hennessey Dam site provides a water yield greater than that required by the City, it was decided to determine if there are other potential dam sites closer to the City (i.e. upstream of the Hennessey Dam site) that would provide a smaller water yield that would still meet the City's requirements.

One good location on Turkey Creek for a dam would be immediately north of Lahoma. This location has two advantages:

1. There are no towns, major highways, railroad etc upstream of a dam that would be flooded out by its impoundment.
2. There is the potential to use the existing 42-inch diameter pipeline from the Cleo Springs and Ringwood wellfields to Plant #2, which runs through Lahoma, as a means of conveying the water into the City.

The location of the dam is shown in Figure 5-16. Using the previously mentioned CoE data, the watershed of a dam located at any point on Turkey Creek upstream of Lahoma was found to be too small to provide a maximum reliable yield of 12 MGD. However, by also damming two small tributaries in addition to the main creek, and locating the dam as close as practicable to Lahoma, a maximum reliable yield of 12 MGD can be provided. This "three-dam" solution shown therefore represents the 'minimum case' scenario in that it will just meet the projected

2050 water demand, but will not reliably meet any demand beyond 12 MGD. It should be noted that the geology of the site has not been assessed as part of this high-level study, so its suitability as a dam site from an engineering prospective has not been assessed. Such assessment would be made as part of a feasibility study.

The dam would consist of a main dam approximately 10,800 feet long, with two minor dams approximately 3,150 feet and 850 feet long. The main dam would have a height of approximately 65 feet. A top of dam elevation of 1280.0 feet was selected as being the most appropriate for the surrounding countryside, which sets the maximum pool elevation at 1275.0 feet. Allowing for the operation of an emergency spillway, the conservation pool elevation was set at 1263.0 feet, with no flood pool. At this location, the dam would provide a lake that would be too small to provide flood storage; it would be just large enough to provide a water supply only, and would not be a multi-purpose dam.

The damming of three streams would create three separate lakes. In order that all three can operate as one lake, a channel would have to be excavated between the three valleys to interconnect the lakes. The combined conservation pool elevation would cover an area of approximately 3,800 acres, and the combined maximum pool elevation would cover an area of approximately 6,600 acres. The construction costs assume:

- Three earthen dams with a concrete, gated spillway on the main dam to control the release of water,
- Excavation of two interconnecting channels, approximate total length of 2,200 feet.
- An intake tower for water supply, and
- An emergency spillway cut into one of the dam's abutments to ensure that the dam is never over-topped during extreme flooding events.

The 42-inch diameter ductile iron transmission pipe that passes through Lahoma was found to be adequate to convey not only the peak day flow of 24 MGD from the lake, but also a peak day flow of 5.45 MGD from the Cleo Springs and Ringwood wellfields (taken from Table 5-1), giving a peak day flow of 29.45 MGD. The maximum pressure in the existing line (allowing for a residual 20 psi pressure at Plant #2) was estimated to be 103 psi; therefore pressures in the pipeline would not exceed the working pressure of the lowest pressure class (Class 150) of ductile iron pipe. As a result, the construction cost assumes that in order to convey water to the City, only a pumping station and approximately 0.5 mile of 42-inch diameter ductile iron pipe tied into the existing pipeline is sufficient. Flows from the Cleo Springs and Ringwood wellfields could either be pumped into the lake, or connected into the suction lines of the pumps.

The estimated construction cost assumes that Plant #2 would be expanded to meet the 2050 demand (distribution pumps) and to include facilities to treat surface water. The dam construction cost includes an estimate to purchase land that would be covered by water when it is at the maximum pool elevation. Operating costs have been estimated on the same basis as the Kaw Lake option, except that they have been based upon pumping and treating 5,851 MGY (16 MGD), which is the average projected 2050 combined flow from the lake and the wells.

5.8.6.4 New Wellfield

As stated in Section 5.8.4, the City would need to acquire approximately 61,000 acres (95 square miles) of water rights in the Cimarron Terrace Aquifer to meet the projected 2050 water deficit. Figure 5-17 shows the existing wellfields and transmission lines and an indication of the size of new wellfield in relationship to the above. The locations of the new wellfield have been arbitrarily chosen for illustrative and costing purposes; the actual locations can be anywhere in the aquifer where the City can acquire water rights.

In order to determine the approximate number of wells required to meet the projected 2050 water deficit, the average pumping rate of all the Cleo Springs and Ringwood wells when newly installed was calculated from Table 4-4; this yielded a pumping rate of 118 gpm. Assuming that the sustainable pumping rate of each new well was 118 gpm, the number of wells required is 71. To allow for some wells being inactive at any given point in time, the construction cost estimate is based upon 75 wells. To meet peak day demand, it was assumed that each well would be fitted with a pump capable of pumping 236 gpm. Each well would then be required on the peak day, but only half the wells would be required on an average day. As each well in theory would only pump 50% of the time, the sustainable pumping rate would not be exceeded over the course of a year.

It was assumed that the wells would pump to new wellfield plants, which in turn would utilize the existing 30-inch diameter (Cleo Springs and Ringwood) and 42-inch diameter (Ames and Drummond) transmission lines to convey the water to Plant #1 and Plant #2 respectively. As the ratio of the cross section area of a 42-inch pipe to a 30-inch pipe is 2:1, it was assumed that one-third (25) of the wells would pump to the 30-inch diameter transmission line and two-thirds (50) of the wells would pump to the 42-inch diameter transmission line. Given the well split and the areas selected for the new wellfield, it was assumed that the new wellfield would be served by three wellfield plants, with 25 wells pumping into each plant. The construction cost estimate assumes that each plant would consist of a 1.5 MG concrete, raw water storage tank and a pumping station. Each plant would have a peak day pumping capacity of 8 MGD.

The construction cost estimate is also based upon the assumption that water would be conveyed from the wells to the well field plants via 12-inch collection mains, and that the plants would discharge into new 30-inch diameter transmission lines that would connect into the two existing transmission lines. Based upon spacing the wells evenly throughout the wellfield, it is estimated that 76 miles of collection mains would be required. The length of transmission lines was estimated at 8 miles.

The 42-inch diameter transmission line was found to be adequate to convey a peak day flow of 21.45 MGD (5.45 MGD from Cleo Springs and Ringwood, 16 MGD from two new plants). Pressures did not exceed the working pressure of the lowest pressure class (Class 150) of ductile iron pipe. The 30-inch diameter transmission line was found not to be adequate to convey a peak day flow of 12 MGD (4 MGD from Ames and Drummond, 8 MDG from one new plant). The pressures and flow exceeded the current capacity of the pipeline (55 psi and 10.2 MGD, from Section 4.11.8), which have been restricted due to its age. For this reason, the estimated construction cost includes the replacement of this 15.5-mile long transmission line.

Both Plant #1 and Plant #2 would require upgrading. The construction cost estimate assumes that the chemical dosing facilities and the distribution pumps would be upgraded, but additional treatment would not be required. It is assumed that no additional storage would be required (See Section 8.5).

Operational costs are based upon replacing the wellfield plant pumps at Year 25, and replacing every well pump every ten years. Treatment costs are based upon the cost of dosing chlorine and hydrofluorosilic acid only. In calculating energy costs, it was assumed that each well had a 5 hp pump.

5.8.6.5 Canton Lake

Another potential existing water source is Canton Lake, located on the North Canadian River approximately 47 miles WSW of the City; therefore it is located closer to the City than Kaw Lake. A potential pipeline route is shown on Figure 5-18, Sheets 1 to 2. From Canton Lake the route heads east across country towards Okeene, crossing State Highways 58 and 51A. Before reaching State Highway 51 the route turns ENE to skirt North around Okeene until it reaches State Highway 8. From there, the route turns NE and heads across country towards Ames. After crossing the Cimarron River, the route parallels the existing southern Ames Wellfield trunk main and terminates at the Ames Plant. The length of this route is approximately 29 miles.

In order to get the water from Canton Lake into the City, the Ames Plant would have to be upgraded to convey not only the peak flow of 24 MGD from the lake, but also peak day flows from the Ames wellfield. The current maximum pumping output of the plant is 9,000 gpm (12.96 MGD). In addition, the existing transmission line from the Ames Plant to Plant #1 would also require replacement or the installation of a second line (approximately 15 miles) to handle the increased flows. Plant #1 would also require an upgrade to handle the increased flows and be capable of treating surface water.

Canton Lake has two advantages over Kaw Lake, the other currently available surface water option:

1. It is closer to the City than Kaw Lake and so the cost to install a transmission line should be less.
2. Kaw Lake is at a lower elevation than the City, whereas Canton Lake is at a higher elevation than the City. Due to the distance and elevation difference, the transmission line from Kaw Lake would probably require two pumping stations. In contrast, only one pumping station would be required at Canton Lake to pump water to a high point (approximately 50 ft above normal pool elevation) on the route several miles from the lake, from which point the water can gravitate to the Ames Plant. Pumping costs for this option would therefore be lower for this option.

The disadvantage of this option is that the City does not currently hold any right to take water from Canton Lake. The City was granted 38,000 acre-feet of water supply storage in the lake by the Flood Control Act of 1948, but the Water Resources Development Act of 1990 reassigned this water storage (plus 69,000 acre-feet of irrigation water storage) to the City of Oklahoma City.

While there are unassigned water rights at Kaw Lake that could be assigned to the City (See Section 5.8.2), there are no such unassigned water rights at Canton Lake. Currently the entire available water supply yield (18,480 acre-feet per year, or 16.5 MGD) has been assigned to the City of Oklahoma City by OWRB. In order to obtain water rights from this lake, the City would have to come to an agreement with the City of Oklahoma City to reassign some of their water rights back to the City. Initial discussions were held with OWRB regarding this possibility, but it is their opinion that at the current time such agreement is unlikely to occur. For this reason no costs have been developed for this water supply option.

5.8.6.6 Black Bear Lake

The construction of a dam on Black Bear Creek at a location beside State Highway 74 and two miles S of U.S. Highway 412 (as shown in Figure 5-19) has been proposed to the City Commission to create a recreational lake, which would also have the potential to be used as a water supply.

At the site of the proposed dam, Black Bear Creek has drainage area of approximately 59 miles^2, and therefore any precipitation falling in this area would flow into the lake created by the dam. However the drainage area contains 10 NRCS flood control dams. The dams are designed to reduce flooding in the watershed by holding back flood flows and slowly releasing them back into the water way in a controlled manner, and when the flood flows have been dissipated the flow out of the dam generally stops.

The effect of these dams in the drainage area is that while flood flows will be passed downstream to discharge into a new lake, under normal flow conditions surface runoff will be stored in the dams and some water will be lost through evaporation until the next discharge occurs. The amount of water flowing into a new lake would therefore be reduced and so the dams act as though the lake has a smaller drainage area, to produce a reduced dependable water yield.

Figure 5-19 shows the NCRS flood control dams, and the drainage area that is uncontrolled by the NRCS dams and thus can contribute fully to supplying surface runoff water to a new lake. This uncontrolled drainage area is approximately 13 miles^2, and so approximately 46 miles^2 of the total drainage area is controlled by the dams. The effect of the dams in reducing the dependable water yield can only be determined by further study; however an estimate of dependable water yield for a Black Bear Lake can be made by comparing its drainage area to that of the proposed Lahoma Lake (Section 5.8.6.3).

In order to provide a dependable water yield of 12 MGD, the Lahoma Lake requires a drainage area of 123 miles^2 based upon a usable runoff of 2.85 inches per year. Assuming the same runoff basis, the dependable water yield of Black Bear Lake drainage area without the NRCS dams is calculated to be 8.3 MGD, and the dependable yield of the uncontrolled drainage area is calculated to be 2.5 MGD; in reality, the actual dependable water yield will lie between these two values.

While the construction of a recreational lake on Black Bear Creek would provide the City with an additional water supply, it would not meet the projected 2050 water deficit without further development of other water supplies. For this reason no costs have been developed for this water supply option.

5.8.6.7 Sheridan Lake

Another possible reservoir site is a location south and east of Enid on Skeleton Creek, commonly referred to as Sheridan Reservoir. The site depicted on many planning documents is one-half mile south of SH-51. From a sustainable yield perspective, this is a good location as it is downstream from the confluence of Skeleton Creek with Lyon Creek and Spring Creek, thus adding drainage area to the reservoir. However, this location would require a long dam and the relocation of SH-51. If the drainage area from the two smaller creeks is not needed to develop the required water supply yield, a site one and one-half north of SH-51 should be considered. This location would not require the relocation of SH-51 and would require a shorter dam. Further study is required to determine the feasibility of each potential reservoir location on Skeleton Creek.

5.8.6.8 Cost Comparison

A summary of the main features of each of the four costed alternative water supply sources are given in Table 5-24. The corresponding construction, operational, total and present value costs for each one are given in Table 5-25. Present day energy and water treatment costs were inflated at 2.5% per annum over 50 years to obtain total costs over this period. The present value of the operational costs was calculated using a discount rate of 4.875%. Operational costs do not include staff labor costs.

The Kaw Lake and Hennessey Lake options have the highest total present values ($237.2m and $235.1m respectively). Although Hennessey Lake has the highest construction cost ($221.0m), it has the lowest present value operating costs ($14.1m) of the three surface water options. Kaw Lake suffers from the amount of energy required to pump the water over such a long distance.

Lahoma Lake has the lowest construction cost ($189.0m) and total present value ($207.2m) of the surface water options; however it has the highest present value operational costs of all the options ($18.2m). The reason for these figures is that it utilizes an existing asset, but in doing so a greater volume of water must be pumped and treated due to the inclusion of groundwater from the Cleo Springs and Ringwood wellfields.

The new wellfield option has the lowest construction cost ($188.8m), the lowest present value operational costs ($3.0m) and total present value ($195.5m) of all the options. The main factor for these figures is that the groundwater does not require treatment. If nitrate treatment becomes necessary, the present values will increase. It should also be borne in mind that if the wells cannot produce the assumed average pumping rate of 118 gpm, further wells and collection lines will be required.

In deciding which options should be studied in further detail, the City should bear in mind the following factors:

- The Kaw Lake option appears to be the most uneconomic option for the City. However, if a regional water supply were developed, where the City treated and sold water to other municipalities, this option may become more attractive to the City.
- Kaw Lake already exists. The Hennessey and Lahoma Lake options would require more detailed studies and negotiations with various entities before a lake can be constructed.
- The Kaw Lake option would require the City to operate and maintain three water treatment plants, as opposed to two with all the other options.
- Both Kaw and Hennessey Lake options provide a source of water in excess of the projected 2050 requirements; therefore both options could be reliably dependent to provide additional water for demands beyond 2050.
- The Lahoma Lake option will just meet the projected 2050 water demand. If the increase in water demand is less than projected, this option will continue to meet the City's water needs beyond this date. Alternatively, if growth predictions are correct, further water resources (perhaps additional wells) would be required beyond 2050.
- The Cimarron Terrace Aquifer is prone to drought; therefore the new wellfield option may not provide the expected water yield during periods of drought. However, as pumping from the wellfield should be at a sustainable rate, the water table should be up at the onset of a drought. Therefore, water production may only be really affected during a severe drought.

6.0 WATER RIGHTS ANALYSIS

6.1 INTRODUCTION

As stated previously, the City obtains its water from groundwater wells. In order to withdraw water from their wells, groundwater rights must be owned or leased by the City and a valid permit for the wells and land must be in place.

6.1.1 WATER RIGHTS OWNERSHIP AND LEASES

Groundwater rights are typically acquired through one of two processes: (1) the purchase of a parcel of land that has not leased the water rights or (2) by leasing water rights from another property owner. With the latter option, the water rights lease may require that the lessee pay an annual lease fee or a royalty for the rights. Lease royalties are typically paid based on production rates and may have a specified minimum production rate or minimum fee.

6.1.2 WATER RIGHTS PERMITS/PRIOR RIGHT

To use groundwater for a *beneficial use* within the State of Oklahoma, a valid, appropriate permit must first be acquired. A *beneficial use,* as defined by the Oklahoma Water Resources Board (OWRB), includes, but is not limited to, the use of groundwater for municipal, industrial, agricultural, irrigation, or recreation purposes. Therefore, since the City is withdrawing groundwater for municipal purposes, the City must have valid permits for their wells.

The City currently has *regular* permits, *temporary* permits, and *prior groundwater rights* for numerous wells. The *regular* permit is the typical permit issued for a groundwater well that has a beneficial use for other than domestic purposes. The *temporary* permit is similar to the *regular* permit though the hydrologic survey and the maximum annual yield determination must be revalidated during the term of the permit.

A *Prior groundwater right* (prior right), as defined by the OWRB within Title 785, Chapter 30, Subchapter 1 of the Oklahoma Administrative Code, is:

> *…the right to use ground water established by compliance with laws in effect prior to July 1, 1973, the effective date of the Groundwater Act…*

Beneficial use wells that were used prior to the enactment of the Groundwater Act and were in compliance with the existing laws are deemed to have a prior right if applications were submitted prior to July 1, 1973.

6.2 EXISTING WATER RIGHTS AND PERMITS

GUERNSEY performed a review of the water rights information provided by the OWRB and also participated in discussions with the OWRB. The OWRB has also previously completed an analysis on the water permits for the City based on current OWRB records and updated well locations provided by the City.

6.2.1 EXISTING WATER RIGHTS

The City supplied information regarding water conveyances, leases, and deeds, which provided the current water rights information for the various wellfields. The City has water rights in the Ames, Cleo Springs, Drummond, Enid, and Ringwood wellfields though limited records were provided for the Enid wellfield. Based on available documentation, the acreages of water rights for the various wellfields, including the surrounding areas, are as follows:

- Ames Wellfield– 9,520 acres
- Cleo Springs Wellfield – 6,600 acres;
- Drummond Wellfield – 8,520 acres;
- Enid Wellfield – 480 acres; and
- Ringwood Wellfield – 5,120 acres.

Additional water rights information is included within Tables 6-1 and 6-2 and the wellfield maps, Figures 5-2 to 5-6.

A detailed review of county records for the City's water rights was not performed and is not included within the scope of this Plan.

6.2.2 EXISTING WATER PERMITS

In addition to the water rights, the water permits and prior right status for the wellfields were also reviewed. The results of the review determined that numerous wells are out of compliance due to one of the following issues:

- The well is not located within the correct authorized location but is located on an active permit/prior right and within dedicated lands to Enid;
- The well is not located within the correct authorized location but is located within dedicated lands to Enid;
- The well is located on non-dedicated lands and is not specified within an active permit/prior right;
    - o Non-dedicated land is specified on an inactive permit;
    - o A well is located adjacent to authorized lands and might have been incorrectly located due to errors during surveying or installation;
    - o A well is located on lands that do not have an active or inactive permit but is located within lands with water rights owned or leased by Enid;
    - o A well is located on lands that do not have an active permit/prior right or inactive permit; and

- A well is located on lands that do not have a Final Order for the recorded permit/prior right.

Table 6-2 provides a summary of the water permit/prior right information for Enid’s wellfields. Detailed discussions regarding the water permits/prior right for the wells are provided below and within Tables 6-3 through 6-7. This analysis is limited to information provided by the City and the OWRB.

The allocations specified within Tables 6-2 through 6-7 are based on allocations specified in the permit Final Order or temporary permit. However, allocations for wells that do not have a recognized prior right can be reduced if maximum yield studies demonstrate that the actual maximum yield is lower than previously specified. Specifically, wells that are located within the Cimarron River Terrace Aquifer and the Cedar Hills Sandstone Aquifer and do not have prior rights could be reduced from an allocation of 2.0 acre-feet per acre per year to 1.0 acre-feet per acre per year based on studies performed by the OWRB. The OWRB has not yet approved the order to reduce the limitation from 2.0 to 1.0 acre-feet per acre per year due to community opposition and the necessity for hearings.

If the Cedar Hills Sandstone Aquifer limitation was reduced to 1.0 acre-feet per acre per year, wells constructed after July 1, 1973 and future wells within the Cedar Hills Sandstone could be limited to only 1.0 acre-feet per acre per year. The only current wells within the Cedar Hills Sandstone that would by affected by this reduction are D-26, D-28, D-29, and D-31 thru D-33.

Additionally, the OWRB has set the maximum annual yield for the Enid Isolated Terrace Aquifer at 0.5 acre-feet per acre per year. However, this limit does not affect any current permitted wells but could be applied to existing wells that are incorrectly permitted and future wells that are not replacements of existing wells or are not located within an area of recognized prior right.

6.2.3 AMES WELLFIELD

There are a total of 33 wells, of which 26 are currently “active,” within the Ames Wellfield. The City and OWRB records were reviewed and the results are summarized in Table 6-3.

Based on the information in Table 6-3, the majority of the wells within the Ames Wellfield are “active” but do not have a valid permit. Only five of the wells have valid permits, though three additional wells are specified within a permit but have incorrect locations. Twenty-three wells are located on lands that have inactive permits. Only one well is located on non-dedicated land this is not associated with an inactive permit. The total acreage of authorized lands for the Ames Wellfield is 1,080 acres and the authorized allocation is 3,642 acre-feet per year.

6.2.4 CLEO SPRINGS WELLFIELD

The Cleo Springs Wellfield currently has a total of 31 wells, of which 28 are currently “active." The City and OWRB records were reviewed and the results are summarized in Table 6-4.

Based on the information in Table 6-4, all Cleo Springs wells are located on City dedicated lands. However, 19 of the 31 wells have an incorrect location compared to the location within the temporary permits. The total acreage of authorized lands for the Cleo Springs Wellfield is 7,458 acres (12,798 total acres with 1982-0966) and the authorized allocation is 25,462 acre-feet per year though 1,600 acre-feet per year is for mining purposes (1982-0965). This total acreage and allocation also includes the Ringwood Wellfield since both fields are authorized by Permit 1982-0966.

6.2.5 DRUMMOND WELLFIELD

The Drummond Wellfield currently has a total of 33 wells, of which 20 are currently "active." The City and OWRB records were reviewed and the results are summarized in Table 6-5.

As shown in Table 6-5, eight wells currently have valid permits. Seven wells are specified within a permit and are on dedicated lands but are in incorrect locations specified on the Final Order for the permit. Nine wells are located within lands that have inactive permits. Additionally, six wells are located within non-dedicated and non-authorized lands though a few wells are located adjacent to dedicated lands. These wells could have been erroneously reported due to errors in surveying or during installation. The total acreage of authorized lands for the Drummond Wellfield is 2,380 acres and the authorized allocation is 4,109 acre-feet per year.

6.2.6 ENID (PLANT/SERVICE CENTER, VAN BUREN, NORTHWEST, AND CARRIER) WELLFIELD

The Enid Wellfield, which is comprised of the Carrier, Northwest, Plant, and Van Buren wellfields, currently has a total of 41 wells, of which 21 are currently "active." All seven Plant wells were previously demolished. The City and OWRB records were reviewed and the results are summarized in Table 6-6.

As shown on Table 6-6, 20 are specified within a permit and are located in the correct area. Six wells are located on non-dedicated lands but are adjacent to dedicated lands. These wells could have been erroneously reported due to errors in surveying or during installation. The total acreage of authorized lands for the Enid Wellfield is 310 acres and the authorized allocation is 3,995 acre-feet per year.

6.2.7 RINGWOOD WELLFIELD

The Ringwood Wellfield currently has a total of 27 wells, of which 20 are currently "active." The City and OWRB records were reviewed and the results are summarized in Table 6-7.

Based on the information in Table 6-7, all Ringwood wells are located on City dedicated lands. However, all wells, except for R-9 and R-21, are not in the correct locations specified within the temporary permit. The total acreage of authorized lands for the Ringwood Wellfield is 5,020 acres (9,678 total acres for Permit 1982-0966) and the authorized allocation is 19,008 acre-feet per year, but the total acreage and allocation also includes a portion of the Cleo Springs Wellfield since both fields are authorized by Permit 1982-0966.

6.3 FUTURE WATER RIGHTS

Section 5.7.4 presents a discussion of possible future changes to Oklahoma groundwater allocation law. Using the potential changes from temporary annual allocations to regular annual allocations, the potential future water rights in each well field can be determined.

6.3.1 AMES WELLFIELD

Current annual allocation of 3,642 acre-feet is primarily from prior rights. The four newest wells (Numbers 30 to 33) permitted in 1980 have a combined temporary annual allocation of 400 acre-feet. If this allocation was dropped from 2.0 to 1.0 acre-foot pre acre per year, the total Ames yearly allocation would be reduced by 200 to 3,442 acre-feet.

6.3.2 CLEO SPRINGS & RINGWOOD WELLFIELDS

All of the wells in the Cleo Springs and Ringwood fields are subject to a temporary allocation of 2.0 acre-feet per acre per year. The current combined annual allocation for Cleo Springs and Ringwood is 25,462 acre-feet. If the allocation is dropped to 1.0 acre-foot per acre per year, the combined Cleo Springs and Ringwood allocation would be cut in half to 12,731 acre-feet.

6.3.3 DRUMMOND WELLFIELD

All of the wells in the Drummond Well Field have prior rights with a total annual allocation of 4,109 acre-feet.

6.3.4 ENID WELLFIELDS

All of the wells in the Carrier, Northwest and Van Buren Well Fields have prior rights with a total annual allocation of 3,995 acre-feet.

6.3.5 TOTAL

Table 6-8 shows the annual groundwater allocations as they currently exist and as if the proposed proportionate share for the Cimarron Terrace Aquifer would be reduced from a temporary permit allocation of 2.0 acre-feet per acre per year to a regular permit allocation of 1.0 acre-foot per acre per year. The total annual well field allocation would be reduced from 37,308 acre-feet, equivalent to 33.2 MGD, to 24,277 acre-feet, equivalent to 21.7 MGD.

Water demands through the year 2050 are projected in Section 8. The total average daily demand in 2050 is projected to be 18.4 MGD. This value is less than even the reduced allocation of 21.7 MGD shown above. However, as discussed in Section 5.6, projected production exceeds the recharge rate in all of the existing wellfields, and they are predicted to 'run dry' before 2050. The existing wellfields do not represent a sustainable source for the projected demand. Alternative water sources required to meet the projected demand are discussed in Section 5.8.

6.4 RECOMMENDATIONS

The City currently has wells that are located on lands without proper water rights or OWRB groundwater permits. Additionally, future groundwater rights might be necessary for the City to provide sufficient water to meet future demands. The following sections provide recommendations for the City regarding existing water rights and water permits.

6.4.1 UPDATE/CONFIRM EXISTING WATER RIGHTS

The water rights analysis included a detailed review of records provided by the City. However, the results from the water rights analysis should be compared with county records to confirm their accuracy and to locate any additional records not provided by the City.

6.4.2 CORRECT EXISTING WATER PERMITS

In order to reconcile the current prior right/permit problems for the various well fields, numerous actions must be taken by the City. The following actions are recommended to be taken to correct the current permit violations:

1. Meet with the OWRB to report the results of the investigation and together develop a strategic process to correct the identified problems.
2. Request amendments for the wells within the Cleo Springs and Ringwood Well Fields in order to show the correct locations of the wells on current permits 1982-0965 and 1982-0966.
3. Perform a review of the Prior Rights filings in order to understand the history/documentation that caused various permits within the Ames and Drummond Well Fields to become inactive.
4. After the review of the Prior Rights filings, submit a request to re-open the Prior Right Final Order to add the historical wells that should have received Prior Rights allocation or are located outside, though adjacent to, the land assigned to the Prior Rights through errors in locating the wells during the original installation and reporting errors during the original Prior Rights process.
5. Submit water rights applications for the remaining wells that do not have a current permit.

7.0 WELLHEAD PROTECTION PROGRAM ANALYSIS

The City currently has a wellhead protection plan for each of its five wellfields. The wellhead protection plan is a very important tool for City officials to protect the City's water supply and to reduce the risk of potential pollutants emanating from existing and future development. The existing wellhead protection plans were addressed approximately 10 years ago and are in need of evaluation and updating, as the potential exists for new contamination sources since the previous plans were developed.

Revised wellhead protection plans are currently being prepared by Oklahoma State University (OSU) for the City. Upon the completion of those plans, the City was to be provide copies to GUERNSEY for review; however, at the time of this report, the revised wellhead protection plans have not been completed; therefore a review of the plans are not included within this report.

8.0 TREATMENT AND DISTRIBUTION SYSTEMS EVALUATION

8.1 POPULATION PROJECTIONS AND FUTURE WATER DEMANDS

Population projections are required to estimate future water consumption; anticipate water supply needs; size conveyances and storage; design treatment improvements; identify short and long-range water distribution requirements; and ultimately determine financing scenarios. A "conservative low" projection results in inadequate future service and potentially constrains growth. Conversely, an "aggressive high" projection creates excessive indebtedness in that the debt service expense is allocated to the base of taxpayers and ratepayers that exist during a 30-year financing period. If no significant additional growth occurs, these citizens pay excessive rates for a partially used resource.

The ideal population projection reflects the practical and historical realities of the State, County, and the City. In addition, the projection embraces the philosophy of growth held by the City's management and constituents. Stated another way, many communities are satisfied with its current size and consciously limit expansion. Other communities are aggressively seeking new industrial development and are investing in both short- and long-term growth. The remaining cities and towns fall somewhere in between. The population projection must represent the probable long-term (year 2050) development culture and growth dynamic characteristics of Enid.

Additions to the groundwater system and distribution piping can be made in phases as growth in demands and revenues are imminent. Some water supply projects, notably reservoirs, are a major up-front investment with little opportunity for phasing.

8.1.1 SERVICE AREA

The population projections must predict the boundaries of the proposed service area. Historically, communities in Oklahoma have chosen to operate and maintain their infrastructures independently. There are a variety of reasons for this independence, ranging from a desire to maintain control of revenues, to the civic pride inherent in the self-sufficiency, to the desire not to be "held hostage" by another city, and in some cases rivalry between communities. Although not completely the norm, there appears to be increasing interest in regionalization of utilities. The reasons are compelling and relatively straightforward. Many municipalities, particularly the smaller communities, do not have the resources necessary to fully comply with regulatory requirements or the current levels of good engineering practice. Similarly, collaboration between communities provides some "economy-of-scale" benefits and a lower unit cost of the services. Finally, pooling of management, technical services, operations, and maintenance resources invariably results in a higher level of service and an optimum level of Operation & Maintenance (O&M) expense. The City clearly has a predisposition for providing water service to the region, and in fact, currently serves much of Garfield County. Consequently, the service area is deemed to include Garfield County and potential customers within a 50 to 100-mile radius.

8.1.2 POPULATION FORECASTS

Oklahoma Department of Commerce (ODOC) projections prepared by the Bureau of Census address a duration of 25-30 years. For capital planning purposes, 20 years is reasonable for the life span of major equipment at a plant or supporting pump stations, 40 to 50 years for structures, and 50-100 years for major water supply projects.

8.1.2.1 Population Projection Methodology Number 1 (Government Method)

8.1.2.1.1 State Demographics

A number of statistically valid models have been developed for projecting population growth. Essentially the models use census data, mortality and birth rates, migration analysis, age demographics, trend analysis, and other data to formulate projections. The "official" projections used in this report were prepared by the ODOC using a methodology termed a "Five-Year, Cohort Survival, Regression Model" and essentially predicts that the level of economic development will remain constant and that growth will be influenced by migration and changes in demographics. Some of the statewide trends and characteristics of Garfield County and the City are described below:

- In the year 2000, Oklahoma was ranked the 27th most populous state in the nation (3,372,514 people). In 2002, Oklahoma was ranked 28th with a population of 3,493,700.

- By the year 2025, Oklahoma is projected to increase in population to 4,056,536. This 16.1% increase ranks as the 23rd highest rate of increase in the nation.

- Of this 562,836-person projected increase, approximately 65,000 will enter Oklahoma as a result of international migration and approximately 290,000 people will enter Oklahoma as a result of internal (US) migration (13th highest in the US).

- The remaining 202,000-person increase will result from natural increases (birth minus death).

- The age demographics in Oklahoma and the rest of the nation will change dramatically by the year 2025. The proportion of Oklahoma's population classified as elderly is expected to increase from 13.5% to 21.9% in 2025.

- Oklahoma's dependency ratio, the number of youth under age 20 and elderly age 65 and over, versus people of working age (age 20 to 64) is expected to rise from 76.1% to 89.5% by the year 2025.

The significance of this "graying" of Oklahoma scenario is that the debt service for a water system expansion will be increasingly paid by people on retirement or reduced/fixed incomes.

8.1.2.1.2 Garfield County and Enid Demographics

Regional and local relevant statistics include:

- From 1990 to the year 2000 the population of Garfield County increased an average of 108 people per year, equating to a 2% overall growth rate for the decade. The 2000 county population was 57,813.

- There were 23,175 households in the County in 2000; with an average occupancy of 2.4 people (56,088 people were deemed living in "Households").

- The median household income is $33,006 based on the 2000 census.

- In Garfield County, 21% of the population is rural, and the remaining 79% is urban.

- The City of Enid had a 2000 population of 47,045.

- From 1990 to 2000, the City of Enid gained 1736 residents.

- Enid has 18,955 households, with an average size of 2.39 occupants.

- The median income for a household is $32,227. Of the households in Enid, 11.1% of families were below the poverty level ($17,603 for a family of four).

- The median age of Enid residents is 37 years.

The City and Garfield County have historically grown at a sustained rate of less than 0.3% per year. Having no other subjective data, the ODOC/Census Bureau model predicts that this trend will continue through the year 2030. The City population will increase from 47,045 to 50,860 by the year 2030. This is an average increase of 127 new citizens per year. Concurrently, Garfield County will increase from the 2000 population of 57,813 to a population of 62,500 in the same time period. This annual increase is certainly validated by history and is definitely achievable, but may or may not reflect the growth philosophy of the City.

Figure 8-1 shows the historical population of Enid and Garfield County, ODOC projections and the projected population connected to the water system (see Section 8.1.3). The projections extended through 2050 result in a Garfield County population of 65,625 and a City population of 58,803.

8.1.2.2 Population Projection Methodology Number 2 (Similar Cities Projection)

Another recognized method used in population projections is termed a "Similar Cities Projection." Essentially, cities of the same size or larger are selected for analysis. At some point in the development of these communities each had a population identical to Enid. Graphically, the population curves are adjusted in time to the point where the populations coincide. For example, the population of Enid and most other significant Oklahoma communities was approximately 4,000-5,000 between 1900 and 1930. The population graphs were shifted to coincide with Enid's population of 3,444 in 1900. The populations of the various cities are shown on Figure 8-2.

The logic behind this model is straightforward. The management and citizens of the City are familiar with the communities presented in the model. Therefore, it is easy to visualize the events that occurred in these communities that led to growth. It is also revealing to note other similar communities which have closely paralleled the historic growth rate of the City. From this graph it is evident that relatively high rates of growth are possible. The next step is to reasonably project if the circumstances that catalyzed high growth in other cities will also apply to the City.

The "similar cities" that attained populations between 30,000 and 100,000 had one or more of the following occur:

- Introduction of a major military installation (Midwest City, Lawton, Enid).
- Growth of a major University (Norman, Stillwater).
- Industrial growth (Bartlesville).
- Proximity to a Standard Metropolitan Statistical Area [SMSA] (Midwest City, Norman).
- A less obvious "growth" catalyst (Muskogee).

It is probably fair to assume that neither a major expansion at Vance AFB nor a new major university is likely to locate within the City. Future population growth will result from "regionalization" and expansion of the industrial base.

8.1.3 METROPOLITAN AREA PLANNING COMMITTEE (MAPC) INPUT

It is difficult, and arguably unwise, for an "outsider" to produce a population projection without the involvement of the citizens and City staff. A cross-section of the populace has both objective and subjective knowledge of the trends likely to shape Enid's future. On February 12, 2007, the MAPC met with GUERNSEY project engineers and members of the City staff (Section 3.4). The input from that meeting is summarized below:

- It was reiterated that there is likelihood that the current two acre-feet per acre per year groundwater allocation could be reduced as a collateral result of an ongoing, five-year study of the Arbuckle Simpson Aquifer. Conventional "mining" of Oklahoma aquifers will probably be replaced by a groundwater allocation that protects the surface water stream flow. How much the current allocation will be reduced, the phasing of the reduction (if any), and the legal and political options available to the City, etc., are all unknowns.
- The MAPC members are very well aware of the nominal growth the City has experienced since 1990 and are not predicting an unrealistic or irresponsible high growth scenario.

- The current population growth occurring because of an expansion of Advanced Foods (200 jobs), a new ethanol plant (35 jobs), etc. were viewed as an anomaly and not as a trend towards explosive growth.

- The MAPC supported regionalization and concurred that the entire County should be the service area.

- The MAPC members are uniformly in favor of growth and economic development. It is, however, more desirable to seek industries that add population but little water demand.

- It was previously thought that the City had an untapped surplus of water but with decreasing water table levels and a possible reduction of groundwater allocations it is realized that this may not be the case. Therefore it was deemed advisable to err on the side of a conservative (high) projection to insure adequate future water. It was determined the City should plan on providing water to the entire 2050 Garfield County population.

The relevant conclusions are that the maximum population that will benefit from the expanded water supply is a 2050 County population projection of 65,625. It is also noted that new industries can provide a substantial population equivalent demand that should be factored into the growth scenarios. The population projections must now be related to water demand, number of customers, and billings/revenues.

8.1.4 STATE WATER DEMAND PROJECTIONS

Table 8-1 shows that in 1995, the OWRB projected that year 2050 water use would reach 25.5 MGD in Garfield County. With the then existing groundwater allocation of 2 acre-feet per acre per year and untapped water in Kaw reservoir, Garfield County had a substantial surplus of water on the order of 17.1 MGD. If the groundwater allocation is reduced to 0.5 acre-feet per acre per year, the surplus becomes 13.9 MGD, but only when groundwater pumping does not adversely affect stream flow.

These OWRB projections are one data point for evaluating future demands, but a more exacting analysis is necessary to plan for phasing of capital improvements. This analysis is provided below.

8.1.5 ENID SPECIFIC DEMAND PROJECTIONS

This detailed demand projection was based on meetings with the City Engineer, Water Department staff, City Finance personnel, members of the Chamber of Commerce and as mentioned previously input from the MAPC.

Using 2006 billing records and plant pumping records it was determined that average day demand was 9.59 MGD, with a peak day demand reaching 17.1 MGD. The demand was classified as 46% residential (4.4 MGD), 20% commercial (2 MGD) and 34% industrial (3.2 MGD). The existing flows were then proportioned up to reflect increases in residential, industrial and commercial growth consistent with the 2050 county population of 65,625. Table 8-2 illustrates the various demands and the accompanying assumptions.

8.1.6 APPORTIONING DEMANDS

8.1.6.1 General

As described in Section 8.3, the City operates two water treatment plants which basically serve the east and west halves of the city. It is necessary to determine where the new demands will occur so that improvements to the distribution pumps and system can be planned.

8.1.6.2 Methodology

Initially the April 2005 *Enid Metropolitan Area Comprehensive Plan* was reviewed and a water distribution system map marked with the limits of the seven neighborhood districts delineated in the plan. Members of the GUERNSEY team then met with representatives of the city staff (planning, engineering and water department) to determine where development was occurring, where longer term growth was likely and where actual construction was deviating from the comprehensive plan. This provided acreages and locations of development expected to occur near term (2010) and longer term development expected to occur between 2010 and 2050.

8.1.6.2.1 Density

To estimate the water demands of future developments the density of representative existing developments in Enid was estimated. Aerial photographs (Figure 8-3) were obtained and the surface area calculated. The dwellings were counted and a population of 2.42 persons per household was used to estimate the population density. These estimates ranged from 7 to 10 persons per acre, so the higher 10 person per acre was used. When combined with an average per-capita residential demand of 125 gallons per capita per day, the peak demand for new residential development was estimated to be 1.7 gallons per minute per acre.

8.1.6.2.2 East & West Plant Analysis

The majority of future industrial demand will occur in the east quadrant of the city with the bulk of the residential and commercial development occurring in the west and northwest. An analysis was performed to determine what future modifications would be required to insure adequate capacity at both plants. Table 8-3 illustrates the result of this analysis.

8.1.6.2.3 County Consumption Assumption

Some of the existing and future water consumption will occur outside of the city limits. Consequently it was estimated that approximately 3.75 MGD of demand would be supplied to customers directly from the well fields and or existing pipelines from the well fields. Since it is unknown which well field will provide the water, the pumping rates projected for the east and west plant were not reduced.

8.1.6.2.4 Demands for Distribution System Analysis

Areas within the City limits were identified where future development is likely. It is impossible, however, to determine exactly where the growth will occur and at what densities. For the purpose of properly sizing piping improvements, all the growth areas were assumed to be fully developed with a peak day demand of 1.7 gallons per minute per acre. This assumption has the effect of indicating greater population and water demand than is projected in the tables above, but only within the distribution system model. This is prudent to ensure that new distribution piping is not undersized.

8.2 WATER TREATMENT EVALUATION

The City operates two water treatment plants (WTP): Plant #1 and Plant #2. Plant #1 is located off Chestnut Avenue, slightly north of the City center and slightly west of Van Buren Street (US Highways 60, 64 & 81). Plant #2 is located on Chestnut Avenue, west of Garland Road on the western edge of the City. The two WTP are identified and located on Figure 8-4.

An overview of the two plants is provided below.

8.2.1 PLANT #1

Plant #1 receives water from the Ames, Drummond, and Enid (Plant/Service Center, Van Buren, Northwest and Carrier) wellfields, details of which are provided in Section 4.0 "Collection System Evaluation". The plant consists of:

- Two 6.5 MG, concrete, circular, above ground clear wells, constructed in 1952.
- A chemical treatment building and a pump station, constructed in 1976.

An aerial photograph of the plant facilities is shown in Figure 8-5.

Treatment at the plant consists of chlorination and fluoridation of the incoming raw water upstream of the clear wells. Water is then pumped out of the clear wells into the distribution system. Details of the distribution system are provided in Section 8.3 "Distribution System Evaluation".

The two clear wells are 300 ft in diameter, with a maximum water depth of 13'-2". At this depth of water, the capacity of each clear well is closer to 7 MG than the stated 6.5 MG. The north clear well experiences significant leaks and as of March 2009 was currently under repair.

The chemical treatment building (Photograph 8-1) houses the chemical dosing equipment and 1-ton chlorine cylinders. Outside of the building, there is a bulk hydrofluorosilic acid storage tank. Within the building, the chlorine bottle room contains two 1-ton chlorine cylinders (Photograph 8-2), a cylinder scale, a lifting beam and a hydrofluorosilic acid dosing pump mounted on top of a polypropylene day tank. A second room within the building houses the chlorination equipment. This consists of a modern Wallace and Tiernan chlorinator (installed in 2003) and the original Wallace and Tiernan chlorinator, which remains in service as a back-up (Photograph 8-3). The target chlorine and fluoride residuals leaving the plant are 1.5 mg/l and 1.2 mg/l respectively.

The chlorine bottle room does not have automatic changeover of chlorine cylinders. The scale is old and has a dial display. The City is planning to purchase a replacement digital scale, and install automatic cylinder change over to the standby cylinder, but as of March 2009 this has yet to be undertaken.

A significant issue is the location of the 30-inch diameter raw water line running beneath the building. The pipe is concrete with a steel lining cylinder and reinforcing wires. This RCCP performs fairly reliably when buried but is less reliable when exposed above ground. The line is located in a pit (Photograph 8-4) on the east edge of the building and is subject to damp conditions and corrosion of the connections. This corrosion necessitates the periodic replacement of chlorine injection points. Also the toxicity of chlorine presents a safety hazard in a confined space environment.

8.2.2 PLANT #2

Plant #2 was constructed in 1984 to receive water from the Cleo Springs and Ringwood wellfields, details of which are provided in Section 4.0 "Collection System Evaluation". The plant consists of:

- One 10 MG concrete, rectangular, buried clear well.
- A building containing chemical dosing equipment and distribution pumps.

Treatment at the plant consists of chlorination and fluoridation of the incoming raw water upstream of the clear well. Water is then pumped out of the clear well into the distribution system. Details of the distribution system are provided in Section 8.3 "Distribution System Evaluation".

Rooms within the treatment building (Photograph 8-5) house the chemical dosing equipment, 1-ton chlorine cylinders and a bulk hydrofluorosilic acid storage tank. The chlorine cylinder room contains two 1-ton chlorine cylinders, two digital bottle scales, a lifting beam and a hydrofluorosilic acid dosing pump mounted on top of a polypropylene day tank. The room has automatic change over of the cylinders.

A second room within the building houses the chlorination equipment. This consists of a modern Wallace and Tiernan chlorinator (installed in 2003) and the original Wallace and Tiernan chlorinator, which remains in service as a back-up. The target chlorine and fluoride residuals leaving the plant are 1.5 mg/l and 1.2 mg/l respectively.

8.2.3 RECOMMENDATIONS

Regarding the location of the 30-inch raw water line at Plant #1, it is recommended that at least 39 feet of exposed RCCP pipe be replaced with either flanged ductile iron pipe or fabricated steel pipe. In addition the chemical treatment building needs to be retrofitted or replaced with a retaining wall that permits access to the pipe at ground level. Within the duration of the planning period the pipeline will require replacement.

No other significant capital requirements are anticipated beyond routine annual maintenance at Plant #1 and Plant #2 to maintain the current water treatment regime. However should nitrate concentrations (discussed in Section 5.6 Water Quality) within the raw water become problematic, the installation of a nitrate removal plant at both Plant #1 and Plant #2 is an option.

8.2.4 NITRATE REMOVAL

8.2.4.1 General

(Reference: www.freedrinkingwater.com/water-contamination/nitrates-nitrites-contaminants-removal-water.htm)

Nitrates and nitrites are nitrogen-oxygen chemical units which combine with various organic and inorganic compounds. Once taken into the body, nitrates are converted into nitrites. The greatest use of nitrates is as a fertilizer.

In 1974, Congress passed the Safe Drinking Water Act. This law requires EPA to determine safe levels of chemicals in drinking water which do or may cause health problems. These non-enforceable levels, based solely on possible health risks and exposure, are called Maximum Contaminant Level Goals (MCLG).

The MCLG for nitrates has been set at 10 parts per million (ppm), and for nitrites at 1 ppm, because EPA believes this level of protection would not cause any of the potential health problems described below.

Based on this MCLG, EPA has set an enforceable standard called a Maximum Contaminant Level (MCL). MCLs are set as close to the MCLGs as possible, considering the ability of public water systems to detect and remove contaminants using suitable treatment technologies.

The MCL for nitrates has been set at 10 ppm, and for nitrites at 1 ppm, because EPA believes, given present technology and resources, this is the lowest level to which water systems can reasonably be required to remove this contaminant should it occur in drinking water.

Excessive levels of nitrate in drinking water have caused serious illness and sometimes death. The serious illness in infants is due to the conversion of nitrate to nitrite by the body, which can interfere with the oxygen-carrying capacity of the child’s blood. This can be an acute condition in which health deteriorates rapidly over a period of days. Symptoms include shortness of breath and blueness of the skin.

Lifetime exposure to nitrates and nitrites above the MCL has the potential to cause dieresis, increased starchy deposits and hemorrhaging of the spleen.

Most nitrogenous materials in natural waters tend to be converted to nitrate, so all sources of combined nitrogen, particularly organic nitrogen and ammonia, should be considered as potential nitrate sources. Primary sources of organic nitrates include human sewage and livestock manure, especially from feedlots.

The primary inorganic nitrates which may contaminate drinking water are potassium nitrate and ammonium nitrate both of which are widely used as fertilizers.

According to the Toxics Release Inventory, releases to water and land totaled over 112 million pounds from 1991 through 1993. The largest releases of inorganic nitrates occurred in Georgia and California. Oklahoma has the 14th largest statewide release rate in the US.

Since nitrates are very soluble and do not bind to soils, they have a high potential to migrate to ground water. Because they do not evaporate, nitrates/nitrites are likely to remain in water until consumed by plants or other organisms.

8.2.4.2 Enid Situation

The City has done an excellent job of maintaining nitrate levels less than 10 ppm by managing the contribution of the individual wells. System wide, the nitrate levels in each well discharge is monitored. Some are as low as 1.1 ppm and others are high at 20.7 ppm. The majority of wells fall between 5 to 10 ppm nitrates. The City effectively balances the operations of low nitrate wells and high nitrate wells to maintain a consistent water quality below 10 ppm.

The nitrate concentrations do, however, fluctuate. During dry or drought conditions the nitrates tend to concentrate in the upper levels of the soils. When heavy rains occur such as those experienced in 2007, a significant portion of the nitrates reach the aquifer. The concentration and dilution effect are impacted by the amount of fertilizer used, the rainfall, length of drought, soil conditions, hydraulics of the subsurface, etc.

8.2.4.3 Treatment Options

Obviously, the first choice is to meet Safe Drinking Water Standards by continuing the current method of operation. In the event nitrate levels continue to rise additional treatment may be required at Plant #1 and Plant #2. Fortunately it will not be necessary to treat 100% of the peak day demand, but only enough of a side stream to maintain treated water concentrations of <10 ppm. If nitrate levels were 11 ppm, for example, treatment would be required for approximately 2.5 MGD between both plants. Fortunately most of the unit process for nitrate removal are modular and can be added sequentially as the flow or nitrate levels dictate. Three recognized methods of nitrate removal are:

1. **Electrodialysis** - Electrodialysis is an electromembrane process in which ions are transported through ion permeable membranes from one solution to another under the influence of a potential gradient. The electrical charges on the ions allow them to be driven through the membranes fabricated from ion exchange polymers. Applying a voltage between two end electrodes generates the potential field required for this. Since the membranes used in electrodialysis have the ability to selectively transport ions having positive or negative charge and reject ions of the opposite charge, useful concentration, removal, or separation of electrolytes can be achieved.

2. **Ion exchange** – Ion Exchange is an exchange of ions between two electrolytes or between an electrolyte solution and a complex. In most cases the term is used to denote the processes of purification, separation, and decontamination of aqueous and other ion-containing solutions with solid polymeric or mineralic ion exchangers. Typical ion exchangers are ion exchange

resins (functionalized porous or gel polymer), zeolites, montmorillonite, clay, and soil humus. Ion exchangers are either cation exchangers that exchange positively charged ions (cations) or anion exchangers that exchange negatively charged ions (anions). There are also amphoteric exchangers that are able to exchange both cations and anions simultaneously. However, the simultaneous exchange of cations and anions can be more efficiently performed in mixed beds that contain a mixture of anion and cation exchange resins, or passing the treated solution through several different ion exchange materials. Ion exchangers can be unselective or have binding preferences for certain ions or classes of ions, depending on their chemical structure.

3. **Reverse Osmosis (RO)** - Reverse osmosis is a separation process that uses pressure to force a solvent through a membrane that retains the solute on one side and allows the pure solvent to pass to the other side. More formally, it is the process of forcing a solvent from a region of high solute concentration through a membrane to a region of low solute concentration by applying a pressure in excess of the osmotic pressure. This is the reverse of the normal osmosis process, which is the natural movement of solvent from an area of low solute concentration, through a membrane, to an area of high solute concentration when no external pressure is applied. The membrane here is semi permeable, meaning it allows the passage of solvent but not of solute. The membranes used for reverse osmosis have a dense barrier layer in the polymer matrix where most separation occurs. In most cases the membrane is designed to allow only water to pass through this dense layer while preventing the passage of solutes (such as salt ions). This process requires that a high pressure be exerted on the high concentration side of the membrane, usually 2–17 bar (30–250 psi) for fresh water. This process is best known for its use in desalination (removing the salt from sea water to get fresh water), but has also purified naturally occurring freshwater for medical, industrial process, and municipal applications. Compared to the other options RO does have a relatively high waste stream of up to 25% of flow rate.

8.2.4.4 Concept Design

Figure 8-6 shows a typical schematic for an ion exchange plant, and Table 8-4 provides estimated budgetary equipment costs for both Plant #1 and #2 to treat current peak flows and predicted peak future flows. Depending on future nitrate trends, such modular processes could be added to Plants #1 and #2. To reiterate, these plants appear not to be needed now but remain an option if nitrates become problematical. The schematic was provided by Ethan Edwards, P.E. of Environmental Improvements, Inc (918-396-4670) representing Tonka Equipment Company, Plymouth, MN.

8.3 DISTRIBUTION SYSTEM EVALUATION

The City operates two water distribution supply zones, served by the pumps located at Plant #1 and Plant #2, which are separated by closed valves. The plants and other features of the two water distribution systems are identified and located on Figure 8-4.

An overview of the overall distribution system as of the time of the infrastructure survey (September 2006) is provided below.

8.3.1 PLANT #1 DISTRIBUTION SYSTEM

The Plant #1 pumps serve the eastern half of the City, which includes all of the industry and Vance AFB. The pump station currently contains three split case turbine distribution pumps:

- 1 x 200 hp Peerless, rated 4,300 gpm @ 150 ft head (Pump #2)
- 1 x 100 hp Paco, rated 2,800 gpm @ 150 ft head (Pump #4)
- 1 x 150 hp WDM, rated 3,500 gpm @ 150 ft head (Pump #5)

Each pump is equipped with a variable frequency drive (VFD). There was a fourth distribution pump (Pump #3), but this has been removed. In addition to the above pumps, there is a 150 hp emergency back-up pump (Pump #1) powered by a natural gas engine, which has an approximate output of 3,400 gpm. The pump house is equipped with a 500 kW standby generator and automatic transfer, so the back-up pump would only ever be used if the standby generator failed.

The plant is operated to maintain a water pressure of 58 psi (134 ft head) out of the plant. The pumps currently operate in the following sequence:

1) Pump #2 (200 hp) is the main pump, and runs continuously
2) Pump #4 (100 hp) is the next pump to come online
3) Pump #5 (150 hp) is the last pump to come online

As water demand in the distribution system increases, and the water pressure at the outlet of the plant drops, the VFD of Pump #2 increases the pump speed to increase water pressure. If the water pressure drops by 3 psi and Pump #2 VFD is at 100% for more than 1-2 minutes, Pump #4 is brought online. If a similar situation occurs with both pumps in operation, Pump #5 is brought online. When the system water pressure increases as water demand decreases, Pumps #4 and #5 are taken offline in reverse order.

There are two booster stations in the water distribution system. The Vance Booster Station pumps water to a 0.3 MG standpipe located at Vance AFB via a dedicated 10-in water distribution line. The booster station contains one Paco 30 hp pump and one Gould 30 hp pump, both rated 500 gpm @ 173 ft head. The booster station is located at the junction of Randolph Avenue and Grant Street, close to the City center (Figure 8-4). During periods of low pressure, the Gray Ridge Booster Station increases pressure to the line that runs to the Gray Ridge addition, located south of the City on US Highway 81 and close to Vance AFB. The booster station contains three Cornell 7.5 hp pumps. The booster station is located close to the junction of Rupe Avenue and Van Buren Street, on the southern edge of the City (Figure 8-4).

The water distribution system also has an unused 3.5 MG underground storage reservoir. It is located in Government Park at the junction of Owen K. Garriott Road (US Highway 412) and 9th Street.

8.3.2 PLANT #2 DISTRIBUTION SYSTEM

Plant #2 pumps serve the western half of the City, which is the main residential area. Water is pumped out of the clear well into the distribution system to feed a 0.75 MG water tower located west of the junction of Phillips Avenue and Oakwood Road, on the north-western edge of the City (Figure 8-4). The pump station contains four vertical turbine distribution pumps:

- 2 x 100 hp Gould, rated 1,500 gpm @ 167 ft head (Pumps #1 and #2)
- 2 x 200 hp Gould, rated 3,500 gpm @ 178 ft head (Pump #3 and #4)

Pump #4 is equipped with a variable frequency drive (VFD). In addition to the above pumps, there is an emergency back-up pump (Pump #5) powered by a diesel engine, which has an approximate output of 3,500 gpm. The pump station does not have a standby generator.

The pumps are operated to maintain the water level in the water tower. The City has determined that the distribution system can operate satisfactorily with the tower operating half full, with a 4 ft operating range. To achieve this, the plant is operated to maintain a water pressure of 62 psi (143 ft head) in the morning and 75 psi (173 ft head) in the evening out of the plant. The pumps operate in the following sequence:

1) Pump #4 (200 hp) is the main pump, and runs continuously
2) Pumps #1 and #2 (100 hp) alternate coming online
3) Pump #3 (200 hp) is a redundant standby to Pump #4

As the water level in the water tower drops, the VFD of Pump #4 increases the pump outlet pressure in 2 psi increments. If the water level continues to drop and Pump #4 VFD is at 100%, a 100 hp pump is brought online. If a similar situation occurs with both pumps in operation, the second 100 hp pump is brought online. When the water level increases as water demand decreases, the two 100 hp pumps are taken offline in reverse order.

8.3.3 DISTRIBUTION SYSTEM ISSUES

There are several pressure-related issues in the Plant #1 distribution system. The topography of the City is such that the land falls from the north-west to the south-east by approximately 150 ft. Plant #1 is located close to the western edge of its supply zone; therefore most of its supply zone is located at a lower elevation than the plant. The effect of this is that while the pressure out of the plant is 58 psi, pressures approach 100 psi at the eastern edge of the distribution system. Conversely the western edge of the distribution system, on the dividing line with the Plant #2 supply zone, is at a higher elevation than the plant. This results in system pressures of approximately 50 psi, which are the lowest water pressures in the City.

Plant #1 provides water to the Great Lakes Carbon plant, located a few miles north of the City and close to the town of Kremlin. When the plant is filling its water tanks, some subdivisions in North Enid suffer from extremely low pressure.

As there are no water towers in the Plant #1 distribution system, pressures in the system are maintained exclusively by the plant’s distribution pumps. In July 2006, the City had a situation where there was a slipped coupling on the 100 hp pump. When the pump was called into operation, the motor was turning but the pump was not pumping. The SCADA system showed that the pump was in operation, as it confirms 'run' off the contact on the motor starter. As a result, the pressure dropped and the control system brought the next pump online. However, the pump was too big for the demand on the distribution system, causing a pressure spike and also causing the control system to take the pump offline. The control system would then "start" the 100 hp pump, setting up a cyclical starting and stopping of the larger pump. The effect of the problem was to cause large pressure differences in the distribution system, leading to multiple burst water mains. The City has put pressure limits on the SCADA system so that pressure spikes cannot occur. However, if pumps, the SCADA system or the electricity supply fail, the pressure will drop in the distribution system and will rise rapidly when the problem is resolved.

In both Plant #1 and Plant #2 water distribution systems, there are some cast iron pipes that are over 75 years old. The City has commenced a program of replacement of these old mains.

8.3.4 SCADA SYSTEM

A detailed description of the SCADA system is provided in Section 4.0 "Collection System Evaluation". On the water distribution system, the SCADA system is used to monitor the discharge pressure from Plant #1 and Plant #2, monitor the water level in the water tower, and control the operation of the distribution pumps. A description of how the system controls both plants is described in the previous section.

The main overview web page for the system shows the water level at the storage reservoirs of each plant, the discharge pressure out of Plant #1, and the water level in the water tower fed by Plant #2. The main web page for each plant shows includes:

- The status of each pump
- The VFD speed of each operational pump fitted with a VFD
- The water level in the storage reservoirs, plus the high and low limits
- The plant discharge pressure, and the pressure set point
- The chlorine residual leaving the plant, plus the high and low limits
- The discharge flow leaving the plant, and
- For Plant #2, the water level in the water tower, plus the high and low limits

From the main page, access is provided to a Disable page and a Chart page. The disable page allows the operator to disable limit levels, to allow for manual control of the distribution system without initiating an alarm or trip. The Chart page allows for the plant output parameters (VFD speed, pressures, water levels, flows and chlorine concentration) to be monitored in graphical format for the previous 10 hours. The system also allows for historical graphs to be produced for individual plant parameters.

8.3.5 CONDITION OF INFRASTRUCTURE

The Plant #1 pump house was constructed in 1976, and appears to be in good condition. All three current distribution pumps have been re-built in the past. The 200 hp Peerless pump was installed in 2001, but the bronze impeller was replaced with a ductile iron impeller in 2003, as it had corroded due to high chlorine concentrations. The fourth removed distribution pump is due to be replaced in March 2009, and it will be a 150hp pump with an approximate rating of 3,500 gpm at 150 ft head. The City has stated that the unused 3.5 MG underground storage reservoir is in very good condition.

Plant #2 and the elevated water tower were constructed in 1984, and appear to be in good condition. At this point in time, the City has not stated any problems with current distribution pumps.

8.3.6 FUTURE INFRASTRUCTURE REQUIREMENTS

The City's water distribution system will have to grow to accommodate future increases in water demand. From Section 8.1 "Population Projections and Future Water Demands", the water demand over the period 2006-2050 is predicted to almost double. Average daily demand is predicted to rise from 9.59 MGD to 18.44 MGD (a 92% increase), and the peak day demand is predicted to rise from 17.06 MGD to 33.39 MDG (a 96% increase).

To assess the future infrastructure improvements required to accommodate the predicted rise in water demand, the future demands were put into the City's existing WaterCAD computer model of the water distribution system. The demands were added in increments to model the system in the years 2010, 2020 and 2050, to determine how the implementation of infrastructure improvements should be phased. Details of how the computer model was updated to reflect the predicted increase in water demand, the results of the modeling and the recommended infrastructure requirements are covered in the following sections.

8.3.6.1 Existing System Model

The system model furnished for use in this study was first reviewed for accuracy and completeness. The pipes reviewed were found to be properly located with correct diameters. Node elevations were found to be consistent with elevations on the USGS quad maps. The level of skeletonization was somewhat inconsistent in that the newer subdivisions outside of the core city were modeled in more detail, i.e. more service lines were included. The impact of this on the overall distribution of flows and pressures was evaluated and found to be negligible. Therefore, no revisions were made to the basic model geometry.

Both Plant #1 and Plant #2 are modeled as reservoirs in the existing system model. Plant #1 is modeled with a hydraulic grade line elevation of 1,408.5 feet, giving a pressure of about 60 psi at the location where it supplies the system. This is consistent with current operations, which call for a pressure of 58 psi. Plant #2 is modeled with a hydraulic grade of 1,458.8 feet, giving a pressure of about 64 psi at that location. This is also consistent with reports of how that zone of the system is operated.

The 0.75 MG elevated storage tank in the western part of the city is modeled as having a ground surface elevation of 1,346 feet, bottom of bowl elevation of 1,439 feet, and overflow elevation of 1,479 feet. The initial water level was specified as 1,458 feet, which is consistent with the current means of operation wherein the tank floats on the system and is maintained at about that level.

The 0.3 MG standpipe that serves Vance AFB was modeled as having a base elevation of 1,272 feet and an overflow elevation of 1,419 feet, both of which were determined to be appropriate. Water levels in this tank are maintained by the Vance Booster Station.

The Vance Booster Station was modeled with two pumps, one having a design point of 560 GPM at 137 feet of head and the second having a design point of 440 GPM at 133 feet of head. However, both pumps are reported to pump 500 GPM at 173 feet of head (Section 8.3.1). The model was revised to make this change. The maximum change in water level in the Vance AFB standpipe was about 2 feet. Also, since the only function of these pumps is to keep the Vance standpipe full, there was no change to flows and pressures in the system otherwise.

The Gray Ridge Booster Station is presently modeled with one pump operating at a design point of 350 GPM with 60 feet of head. The pump manufacturer (Cornell) was contacted to request the pump curves for these pumps, but no response was received. Factoring in efficiency, the modeled pump is consistent with one 7.5-hp pump so this booster station was not revised for the existing system model.

According to a previous report regarding the model, node demands were assigned based on housing density and location of large users. Table 8-5 summarizes the demands in the model furnished for use in this study.

Two demand patterns were used. Nodes that were primarily industrial plants were modeled with a constant rate of demand, "Fixed" pattern. The nodes in primarily residential areas were modeled with a 24-hour pattern that was developed based on the production reported throughout the day for each plant. Figure 8-7 shows the patterns used for each plant.

The model demands were compared to the demands estimated using updated (2006) information. Table 8-6 lists the updated demand information.

Since the demands in the existing model were originally distributed according to population and housing density, it was concluded that the relative magnitudes of the demands within each zone were correct. To bring the system model into agreement with the updated demand information, the base average and maximum day demands in each plant zone from Table 8-5 were scaled by a factor to bring them into agreement with the values in Table 8-6.

The patterns shown in Figure 8-7 were also evaluated to determine if revision was needed. Particularly for residential areas, these patterns appeared to underestimate the peak morning and late afternoon/early evening demands. They also appeared to exaggerate the overnight usage. These demands were reportedly based on plant production records. Since plant production throughout the day is not expected to correlate with consumer usage, we concluded that adopting a textbook residential pattern would result in a more representative model. Accordingly, the pattern given in *Water and Wastewater Technology* (Hammer, M.J. and M.J. Hammer, Jr. 2001. 4th ed. Prentice-Hall, Upper Saddle River, NJ.) was adopted as the "Typical Residential" pattern, shown in Figure 8-8. The residential nodes in the model were revised to use this pattern and the industrial nodes were left as "Fixed."

For industrial plant nodes in the future system models, a two-shift plant model was adopted. This pattern was based on the assumption that from the hours of 7 a.m. and 11 p.m. water demand would be double what it was between the hours of 11 p.m. and 7 a.m. Figure 8-8 also shows this pattern. Another revision to the existing system model was to use the patterns step-wise across each one-hour time step, as shown in the figure.

The Existing System model developed as described was then used as the basis for the future system models.

8.3.6.2 Development of Future System Model

To model the effect of future water demand on the water distribution system, it is necessary to know not only the predicted demand, but also where the demand is going to occur in the City. This will influence whether a particular infrastructure improvement is required to either one or both of the water supply zones. For example, if all of the increase in water demand were to occur in the western half of the City, then it is likely that infrastructure in the eastern half of the City (served by Plant #1) would not require any improvements. To ensure that the recommended infrastructure requirements are sufficiently robust to accommodate all possible future demands, modeling should be performed using peak day flows.

To model the future water demands, the assumptions stated in Section 8.1.5 "Enid Specific Demand Projections" and the information stated in Section 8.1.6 "Apportioning Demand" were used to predict the peak day flow (in gpm) demand generated by future residential, commercial and industrial development in particular parts of the City. These peak day demands occurring by 2010, 2020 and 2050 were then added to the 2006 peak day demand to produce a total peak day demand for the three years in question.

As stated in Section 8.1.6.2.4 "Demand for Distribution System Analysis", for the purpose of properly sizing piping improvements, a peak day flow of 1.7 gallons per minute per acre of development was used to produce model peak day demands that were greater than those predicted by the population growth. By modeling higher peak demands than predicted, any infrastructure improvements should be robust in the event that the pattern of water demand growth is different.

Using the procedure detailed above, the areas of assumed future residential, commercial and industrial water demand across the City for 2010, 2020 and 2050 are shown in Figures 8-9 to 8-11, along with the predicted peak day water demand for that area.

To serve these future growth areas, new water lines were added to the model. Where water lines were to be extended to previously un-served areas, water mains that were to be located on section lines were originally designated to be 12-inch DIP pipe. Service lines extended up into the sections, either for infill or new areas, were originally designated to be 8-inch PVC. The additional demands were assigned to nodes within the areas where population growth was projected. If the growth area was relatively small – a quarter section or less – one node was typically used to represent all the demands. For larger areas, the new demands were typically assigned to three or four nodes distributed throughout the section. If the model indicated that the 12-inch or 8-inch pipes could not support the 2050 demand, the pipe sizes were increased until the model showed that there was adequate capacity. Pipes 12-inch and larger were designated to be DIP and 8 and 10-inch pipes were designated to be PVC.

The process used to define the improvements needed to meet the demand in each time frame was to first determine the improvements needed to support the 2050 system. Then, for each previous time frame, those improvements not necessary to support the lower demand were identified. Accordingly, a plan for phasing in the improvements was developed.

As stated in Section 8.1.6.2.4 "Demands for Distribution System Analysis", a peak day demand of 1.7 gallons per minute per acre has been used to ensure that new distribution piping is not undersized. With all the projected demands in place, the actual calculated demand for the 2050 system model was 40.5 MGD for maximum-day conditions and 20.3 MGD for average-day demand. This compares to the predicted demands of 33.39 MGD for maximum-day conditions and 18.45 MGD for average-day demand. Improvements were sized to generally maintain pressures of 40 psi under maximum-day conditions and then checked under average-day conditions to ensure that excessive pressures would not occur during low-flow hours, particularly in the older sections of the city.

8.3.6.3 Recommendations

The improvements described below are primarily recommended to solve pressure and capacity problems that are projected to occur as the demand increases. They include lines required to provide water previously un-served areas, but do not include lines extended up into currently served sections to serve new development within them.

8.3.6.3.1 2010 Improvements

Improvements for the Plant #1 zone recommended for 2010 (or whenever system maximum-day demands approach 29 MGD) include:

1. Maintain the hydraulic grade at the plant at 1,408.5 feet. Raising the hydraulic grade was considered, but was discounted due to City staff concerns of pipes bursting to due higher pressures.
2. Install a 30-inch DIP line from the plant north along Van Buren Street, to Spruce Avenue. Estimated length is 4,200 feet.
3. Install a 24-inch DIP line from the above mentioned 30-inch DIP line on Van Buren Street east to 4th Street, along the line of Spruce Avenue and Hemlock Avenue. Estimated length is 5,400 feet.
4. Install an 18-inch DIP line from the above mentioned 24-inch DIP line on 4th Street, north to the intersection of Phillips Avenue and US Highway 81. Estimated length is 13,300 feet.
5. Install a 6-inch PVC line parallel to the existing 10-inch line on Van Buren Street, from Maine Street south to West Owen K. Garriott Road. Estimated length is 1,800 feet.
6. Install a 10-inch PVC line parallel to the existing 6-inch line on Van Buren Street, from West Owen K. Garriott Road south to an existing 10-inch line at Moore Avenue. Estimated length is 4,600 feet.
7. Upgrade the Gray Ridge Booster Station. Install new pumps, 350 gpm with 230 feet of head.

8. Construct a 15-foot diameter standpipe near Vance AFB in the vicinity of the US 81/Fox Drive intersection. There is high ground to the south of the intersection. High water elevation should be 1,410 feet. The ground elevation at that location is approximately 1,280 feet. This standpipe can be eliminated if the water tower at the Gray Ridge Booster Station is constructed. (See discussion below and in Section 8.5).

Items 2, 3 and 4 reinforce the distribution system from the plant to the northern part of the City. Providing additional elevated water storage in the northeast part of the City was evaluated as an alternative to installing a line extending from Plant #1 to the industrial area. Under such arrangement, the distribution system would still require reinforcement (but with smaller diameter lines) to enable the tank to fill by floating on the system. While an elevated water tower is not required with the proposed improvements, the City has decided to install a 0.75 MG elevated water tower in the vicinity of the Willow Road/30th Street intersection. The purpose of the tower is to increase storage for emergency back up and to produce operational savings through decreased operation of the distribution pumps.

Items 5 and 6 permit the Gray Ridge Booster Station to be upsized. The area north of the booster station suffers from low pressure due to the suction of the booster pumps. Without increasing the capacity of the line that feeds the booster station, upgrading it would only increase the low pressure problems. Items 7 and 8 provide increased fire flows and system pressures to the Gray Ridge addition and surrounding area.

In regards to Item 8, the City has decided to install a 0.75 MG elevated water tower in Meadowlake Park on Rupe Avenue (close to the Gray Ridge Booster Station) as an alternative to a standpipe. The purpose of the tower is to increase storage for emergency back up and to produce operational savings through decreased operation of the distribution pumps.

Improvements for the Plant #2 zone recommended for 2010 include:

1. Install a 12-inch DIP cross connection between the existing 8-inch and 24-inch lines on Chestnut Avenue in the vicinity of Cleveland Street. Estimated length is 60 feet.
2. Construct loops of 12-inch DIP lines around the sections that are projected for future developments. This includes:

   - Install a 12-inch DIP line along Phillips Avenue, from Oakwood Road east to Cleveland Street. Estimated length is one mile (5,280 feet).
   - Install a 12-inch DIP line along Cleveland Street, from Phillips Avenue south to an existing 12-inch line at Sherry Lee Avenue. Estimated length is ½ mile (2,640 feet).

Items 2, 3 and 4 reinforce the distribution system from the elevated water storage tank to the northern part of the City.

8.3.6.3.2 2020 Improvements

The projected increase in residential population in the Plant #2 zone, and residential and industrial expansion in the Plant #1 zone, results in recommended improvements being scheduled for 2020 (or when overall system maximum-day demand approaches 31 MGD).

Improvements for the Plant #1 zone recommended for 2020 include:

1. Extend the 24-inch DIP line (item 3 in 2010 improvements) due east from 4th Street to 16th Street. Estimated length is one mile (5,280 feet).

This improvement reinforces the distribution system to the eastern part of the City.

Improvements for the Plant #2 zone recommended for 2020 include:

1. Install a 36-inch DIP line from the plant parallel to the existing pipes along Chestnut Avenue, to Garland Road. Estimated length is 3,080 feet.
2. Install a 24-inch DIP pipe parallel to the existing 12-inch pipe along Garland Road, from Chestnut Avenue to Willow Road. Estimated length is one mile (5,280 feet).
3. Install an 18-inch DIP pipe parallel to the existing 12-inch pipe along Willow Road, from Garland Road to Oakwood Road. Estimated length is one mile (5,280 feet).

These improvements reinforce the distribution system from the plant to the elevated water storage tank.

8.3.6.3.3 2050 Improvements

Additional improvements recommended to serve the projected 2050 maximum-day demand of 34 MGD include (for the Plant #1 zone):

1. Extend the 24-inch DIP line (item 1 in 2020 improvements) due east from 16th Street to 30th Street. Estimated length is one mile (5,280 feet).

This improvement is an extension of the recommended 2020 improvement to reinforce the distribution system to the eastern part of the City.

Improvements for the Plant #2 zone recommended for 2050 include:

1. Raise the operating range in the elevated storage tank from 1,460 to 1,479 feet. Filling the tank during maximum usage days will require a discharge hydraulic grade at the plant of 1,520 feet or about 215 feet of head.
2. Install a 12-inch DIP line parallel to the existing 12-inch line along Oakwood Road, from Willow Road to Purdue Avenue. Estimated length is one mile (5,280 feet).
3. Construct loops of 12-inch DIP lines around the sections that are projected for future development. This includes:

    - From the plant along Chestnut Avenue to Wheatridge Road. Estimated length is 2,200 feet.
    - Along Wheatridge Road, from West Owen K. Garriott Road to Willow Road. Estimated length is two miles (10,560 feet).
    - Along West Owen K. Garriott Road, from Wheatridge Road east to an existing 12-inch line. Estimated length is ½ mile (2,640 feet).

- Along Willow Road, from Wheatridge Road to Garland Road. Estimated length is one mile (5,280 feet).
- Along Garland Road, from Willow Road to Purdue Avenue. Estimated length is one mile (5,280 feet).
- Along Purdue Avenue, from Garland Road to Oakwood Road. Estimated length is one mile (5,280 feet).
- Along Phillips Road, from Cleveland Street to Van Buren Street. Estimated length is one mile (5,280 feet).
- Along Van Buren Street, from Phillips Road to Purdue Avenue. Estimated length is one mile (5,280 feet).
- Along Purdue Avenue, from Cleveland Street to Van Buren Street. Estimated length is one mile (5,280 feet).
- Along Van Buren Street, from Purdue Avenue south to the end of the existing line. Estimated length is .018 miles (950 feet).

Item 2 is an extension of the recommended 2020 improvement to reinforce the distribution system from the plant to the elevated water storage tank.

Table 8-7 gives a listing of the new pipes that will be needed to carry out the above improvements. All of the recommend infrastructure improvements for 2010, 2020 and 2050 are shown in Figures 8-9 through 8-11.

Figures 8-12 through 8-17 show the predicted maximum day system pressures and water demands throughout the system for 2010, 2020 and 2050. Table 8-8 provides a summary of the lowest pressures in each zone calculated at the critical (peak) hour in the maximum-day simulation and the maximum pressure calculated for the critical (least demand) hour in the average-day simulation. These values represent the extreme range of predicted pressures in the system. The maximum pressures reported for the Plant #1 zone occur at scattered areas with low elevations, generally to the extreme south and east ends of the system. If damage is expected from these pressures, installation of "blow-offs" should correct any problems. In general, any other measures that could be taken to reduce the pressures in these areas are likely to result in inadequate pressures in several industrial areas when usage is at a peak.

8.4 DISTRIBUTION PUMPS EVALUATION

In the development of the future system model, a system head curve was developed for each water distribution supply zone using the steady-state model of the 2050 system. The 2050 model was selected because it includes all the recommended improvements.

The curve was developed by gradually incrementing the demand in the zone and using the model to determine the discharge hydraulic grade at the pumping plant required to maintain pressure throughout the zone. The pressure to be maintained was the pressure obtained from running the 2050 steady-state model with maximum-day demands.

The distribution pumps at Plant #1 pump out of ground-level tanks. Since the water level will fluctuate in the clear wells, it is not possible to obtain a precise value for the suction grade at the pumps. Therefore, the suction grade was assumed to be the ground elevation of 1,270 feet.

Table 8-9 shows the discharge hydraulic grade and pumping head required for discharges ranging from 5,000 GPM to 30,000 GPM.

The distribution pumps at Plant #2 pump out of the underground clear well. For this curve, the suction grade was assumed to be 5 feet below ground level. Table 8-10 has the Plant #2 system curve.

The existing pump capacity was evaluated to the extent feasible, since it was not possible to obtain all the curves for the existing pumps. For Plant #1, the original manufacturer's curve was obtained for the 200 Hp Peerless pump and a manufacturer's curve for a similar model was obtained for the 100 Hp Paco pump. However, there was no information available for the 150 hp WDM pump, so the algorithms in WaterCAD were used to determine a hypothetical curve based on the reported design operating point of 3500 GPM at 150 feet of head.

Figure 8-18 shows the curves for each Plant #1 pump, plus combinations, superimposed on the system curve. This curve indicates that the present pumping capacity is marginally adequate up to about 10,000 GPM or about 14 MGD in that part of the system. The year 2010 model has a maximum-day demand of 16.5 MGD in the Plant No. 1 zone.

The pumps in Plant No. 2 are Gould vertical turbine pumps. The manufacturer's curve could not be obtained for the 200 hp pumps, but provided a rough curve for a generic model. The 100 hp pump is no longer made, but a pump curve for the replacement model was obtained. This was used as a template for the shape of the pump curve and scaled so the design point was the same as the reported design point of 1500 GPM at 167 feet of head. Figure 8-19 shows the Plant No. 2 pump curves, in various combinations, superimposed on the system head curve. This shows that the existing pumps are marginally adequate for a maximum-day demand of about 12 MGD, which falls slightly short of the 2010 maximum-day demand in the model of 13.1 MGD.

8.4.1 RECOMMENDATIONS

As long as the amount of storage in the system remains similar to existing conditions, the use of variable drive pumps for replacement pumps in the plants is recommended to maintain a more consistent pressure. This is particularly important if the recommendations to raise the pressure plane in the Plant #2 zone (Section 8.3.6.3.3) are adopted. For Plant #1, an overall capacity of about 15,000 GPM and a discharge grade of 1,450 feet (or about 180 feet of head) will satisfy the future system demands. Plant #2 requires a capacity of 15,000 GPM with a discharge grade of about 1,520 feet or 215 feet of head to satisfy future system demands.

8.5 DISTRIBUTION STORAGE EVALUATION

At present, there is one 0.75 MG elevated storage tank in the system and clear wells located at the plants. At Plant #1 there are two 6.5 MG clear wells and at Plant #2 there is one 10 MG clear well. There is also the standpipe at Vance AFB, but this standpipe is filled by a dedicated line from the Vance Booster Station and therefore does not have any impact on the rest of the system. In addition, there is an unused 3.5 MG reservoir at 9th Street.

Determination of the amount of storage required involves computing the amount of storage required for production equalization, the amount of storage required for fires and other emergencies, and complying with ODEQ standards. ODEQ has a generic requirement to "maintain sufficient storage capacity to meet domestic demands and fire flow demands, where fire protection is provided." With respect to fire flow, the requirement is: "Satisfy fire flow requirements established by the American Insurance Association where fire protection is provided." Since the adoption of those ODEQ rules, many communities have adopted the International Fire Code and International Building Code, whose provisions are generally more stringent than American Insurance Association or Insurance Service Office requirements for fire flow. The IFC/IBC fire flow requirements are based on the size and potential hazard classification of the individual building. A flow rate of 4,000 GPM for four hours satisfies the fire flow demands for a low-hazard building of up to 145,900 square feet and a high-hazard building of up to 23,300 square feet. For planning purposes, this represents a conservative upper limit of the future fire flow requirements. Individual facilities with requirements in excess of this amount can be required to provide supplementary fire protection.

AWWA Manual M31, *Distribution System Requirements for Fire Protection* states about storage in general: "Rarely can distribution storage be economically justified in an amount greater that what will take care of normal daily variation and provide the needed reserve for fire protection and minor emergencies." The manual goes on to state that in "moderate size" systems, the amount of storage needed for production equalization is typically 30 to 40 percent of total storage.

For the 2010, 2020, and 2050 future systems, the daily variations in consumption were determined and the amounts of storage required to satisfy the peak demands under maximum-day conditions were calculated for each Plant Zone. Figures 8-20 through 8-25 show the results for 2010 Plant #1, 2010 Plant #2, 2020 Plant #1, 2020 Plant #2, 2050 Plant #1, and 2050 Plant #2, respectively. Table 8-11 summarizes the storage volumes required.

While it is desirable to have more storage distributed throughout the system, the capacity of the clear wells at the plants is adequate for production equalization and fire protection in each zone. The storage required for peak demand is close to the 30 to 40 percent of total storage cited as typical in the AWWA manual, so there is additional volume available to cover temporary decreases in production from the well fields. Therefore, additional storage is not required at this time. If the existing storage reservoirs need to be retired in the future, consideration should be given to replacing them with decentralized storage located throughout the zone.

A primary issue with the location of the current storage is that both zones in the system are completely dependent on continued operation of the pump stations. Since the recommended future hydraulic grade in the Plant #2 zone is higher than that in the Plant #1 zone, it should be possible to provide emergency service to the Plant 1 zone by opening isolation valves. For the Plant #2 zone, some form of back-up pumping will be required to maintain service. Elevated storage out in the system would relieve some of the need for back-up pumping.

With the current pump station configuration, there have been pressure fluctuation problems reported in the Plant #1 Zone (Section 8.3.3). This can be controlled through addition of a 0.75 MG elevated tank located between the plant and the "load center" to the northeast in Sections 2 and 3, T23N, R6W. Typical 0.75 MG elevated tanks have a total working head range on the order of 35 feet and the water level in the tank would be expected to fluctuate 10 to 15 feet throughout the day. The proposed tank should have an overflow elevation of approximately 1,448 feet.

The City plans to install an elevated water storage tanks in the vicinity of the Willow Road/30th Street intersection and beside the Gray Ridge Booster Station in order to:

- Increase storage for emergency back up, so relieving some of the need for back up pumping;
- To control pressure fluctuation problems, and;
- To reduce operational costs through reduced pumping.

8.6 DISTRUBUTION SYSTEM FIRE FLOW ANALYSIS

During 2007 the City Fire Department had noted that there had been a reduction in available fire flow at quite a few hydrants throughout the City. Inspections of isolation valves throughout the City located some left-hand closing valves instead of the standard right-hand closing valves, and so some valves had been inadvertently closed when it was believed that they had been opened. However, the discovery of these valves did not account for all of the reduction.

As there had been some changes in the water distribution network through the installation of new and replacement pipework that could have affected the available fire flow, the City decided that an analysis of the available fire flow was required to ensue that critical areas of the City had adequate fire flow coverage. It was decided that the fire flow capacity would be checked using the existing system model for 25 critical areas in the City that required the greatest fire flow capacity as determined by Needed Fire Flow (NFF). The NFF would then be compared against the actual capacity of the water distribution system to deliver fire flow in these areas. These critical areas would be determined by the spatial distribution of properties within the City that had the highest NFF, as determined by the Insurance Service Office (ISO).

8.6.1 Highest Needed Fire Flow Properties

NFF is the amount of water that should be available for providing fire flow protection at selected locations throughout a community. To help establish premiums paid by a community for fire insurance, ISO evaluates its fire suppression delivery system using a criterion known as Fire Suppression Rating Schedule (FSRS). As part of FSRS, ISO determines the adequacy of the water supply and delivery system by calculating the NFF for selected properties and then compares this value against the availability of water at these sites. ISO also publishes a document on how to determine the NFF for a property.

A list of industrial, commercial and public properties within the City for which the NFF had been determined was obtained. In consultation with City staff and the Fire Department, it was decided that the 25 critical areas would be determined based upon the top 30 NFF properties. A list of the selected properties and their required NFF is given in Table 8-12. This shows that the NFF required for these properties is in the range of 2,500 – 6,000 gpm.

In order to determine the 25 critical areas, the spatial distribution of these properties had to be incorporated within the model. In order to achieve this, the latitude and longitude of each property was obtained. This data was then entered into the City's Geographical Interface System (GIS) to create a new layer that showed the position of each property along with its NFF. This GIS layer was then imported into the existing system model such that the properties could be referenced against the features (mains and hydrants) of the water distribution system. The spatial distribution of the properties with their associated NFF is shown in Figure 8-26.

8.6.2 Hydrant Selection

In order to establish the existing fire flow capacity at the 25 critical areas, it was necessary to undertake an ISO fire flow test at fire hydrants serving these areas.

An ISO fire flow test requires the use of a pair of fire hydrants located close to each other. One hydrant is selected as the flow hydrant and the second hydrant is selected as the residual hydrant. The flow hydrant is selected such that water flows past the residual hydrant to the flow hydrant. The flow hydrant is fitted with a Pitot tube and the residual hydrant is fitted with a hydrant gauge.

The fire flow test consists of first measuring the static pressure in the water main. This is achieved by taking a reading from the hydrant gauge at the residual hydrant while the flow hydrant is closed. Then the flow hydrant is opened and when flows have stabilized, a reading is taken of the residual pressure at the residual hydrant and a reading of the velocity pressure is taken at the flow hydrant from the Pitot tube. From these readings, the available fire flow at a standard pressure can be established (Section 8.6.3).

From the spatial distribution of the highest NFF properties, 25 hydrants were identified as flow hydrants for the purpose of undertaking ISO fire flow tests; the selection of the residual hydrants then followed from this initial selection. The distribution of these hydrants was such that the existing fire flow capacity for 29 of the 30 properties identified in Section 8.6.1 could be determined from the ISO fire flow tests. A list of the selected flow and residual hydrants along with the highest NFF properties nearest to the hydrants is given in Table 8-13. The spatial distribution of the selected flow hydrants is shown in Figure 8-26.

8.6.3 ISO Fire Flow Tests Results

An overview of the ISO fire flow test is given in Section 8.6.2. The fire flow tests were undertaken by the City Fire Department. From the data collected (static pressure, residual pressure and velocity pressure), both the fire flow at the recorded residual pressure and the fire flow referenced to a 20 psi residual pressure can be calculated.

The fire flow at the recorded residual pressure is given by the following equation:

$$Q = 29.83\ c\ d^2 \sqrt{P}$$

Q is the hydrant discharge in gpm
c is the discharge coefficient of the orifice

d is the diameter of the outlet in inches
P is the recorded Pitot (velocity) pressure in psi

A fire hydrant typically has three outlets: a pumper nozzle (to allow connection to a fire truck from which water can be pumped from the main) and two hose nozzles. To obtain flow from the flow hydrant, the City Fire Department used either the pumper nozzle, or one or both hose nozzles. The nozzle diameters can vary with hydrant models, but each hydrant used by the City has 4.5" diameter pumper nozzle and 2.5" diameter hose nozzle. For fire flow tests where two hose nozzles are used, the Pitot velocity for each nozzle must be recorded, therefore the fire flow must be calculated for each nozzle and the two values combined to give the overall fire flow.

The coefficient of discharge for each nozzle also varies by the type of orifice in the nozzle. It typically varies between 0.70 and 0.95, and a value of 0.99 is assigned to an Underwriters Laboratories (UL) playpipe if it is attached to a nozzle. The City Fire Department determined that the orifices of the hydrant nozzles used in the fire flow tests were best described as "Hydrant butt smooth and rounded transition from the barrel", which is assigned a coefficient of discharge of 0.90.

From the calculated residual pressure fire flow, the fire flow can be predicted at a reference residual pressure of 20 psi. This pressure represents the lowest residual pressure that should be available in a water distribution system with a fire demand occurring on the day of maximum demand. The NFF of a property is calculated on this basis; therefore calculation of the residual pressure on the same basis allows for direct comparison between the NFF and available fire flow.

The fire flow at a residual pressure of 20 psi is given by the following equation:

$$Q_R = Q_F \frac{(\text{static-20})^{0.54}}{(\text{static-residual})^{0.54}}$$

Q_R is the predicted fire flow at 20 psi in gpm
Q_F is the fire flow at the residual pressure in gpm (taken from the previous equation)
static is the static pressure measured (in psi) at the residual hydrant without water flowing
residual is residual pressure measured (in psi) at the residual hydrant with water flowing

The results of the ISO fire flow tests are given in Table 8-14. At the time of the fire flow testing, the hydrant in the vicinity of Peppers Restaurant was unavailable for testing due to construction activity and so a flow test was not carried out at this location. The table compares the predicted available fire flow at the hydrant against the NFF of the nearby properties, which shows that 15 of the selected hydrants could not provide the required NFF. Of these 15, 3 hydrants were very close (less than 5% difference) to providing the NFF and in reality would be adequate.

8.6.4 Hydrant Modeling (Current Conditions)

Following completion of the ISO fire flow testing, the next step in the process was to compare the actual fire flow results against the available fire flow capacity predicted by the existing system model. The purpose of this step was to identify any large anomalies between the two values that could indicate a problem within the distribution network. In order to achieve this, the flow hydrants used in the ISO fire flow testing were added to the existing model. In some cases new pipework had to be added to the model since some of the hydrants were on mains that were not in the model due to the level of skeletonization used in the model.

With the hydrants added to the model, each flow hydrant was modeled in turn by applying a flow to the hydrant and then noting the pressure at the hydrant and in the surrounding model nodes. If the pressures at the given flow were predicted to be above 20 psi, the flow at the hydrant was increased until the model predicted that the pressure at the hydrant or a surrounding node decreased to no lower than 20 psi. Conversely, if the pressures at the given flow were predicted to be below 20 psi, the flow at the hydrant was decreased until the model predicted that all of the pressures at the hydrant or surrounding nodes were all above 20 psi. To be consistent with the NFF, the modeling was undertaken based upon the current maximum day demand.

Table 8-15 provides a comparison between the calculated available fire flow based upon the ISO fire flow test results and the predicted available fire flow based upon modeling of the hydrants. The table also compares the predicted available fire flow against the NFF of the nearby properties, which shows that 16 of the selected hydrants could not provide the required NFF.

The discrepancy between this table and Table-14 (which shows that 15 hydrants could not provide the required NFF) is due to Hydrant # 1003, located near to Waller Junior High School on W. Randolph Avenue. The NFF for each building is 4,500 gpm and the ISO fire flow test result gives an available fire flow of 4,823 gpm; however the model gives an available fire flow of only 3,000 gpm. One possible explanation for this discrepancy is that the maximum day demand would occur in the summer and the fire flow tests were undertaken in July (i.e. close to the maximum day demand) but during this period there would be little demand from the school itself, as no pupils would be present. System pressures would therefore be higher than if the school were open on the peak demand day, therefore the available fire flow would be higher as more water could be taken from the main before the 20 psi limit was reached. On the other hand, the model assumes that peak flows at the school would occur at the same time as peak flows in the surrounding area. In reality, this would not be the case and so the available fire flow would be higher than 3,000 gpm.

In comparing the fire flow test results and the modeled fire flows, any discrepancies of +/- 10 to 20% would be considered a good match given that the flow conditions at the time of each fire flow test will not exactly match the flow conditions of the model. On this criterion there was a good match on 8 hydrants. For 4 hydrants the modeled fire flows were more than 40% lower than those derived from the fire flow test, with the greatest discrepancy being 50%. These discrepancies could be due to a number of factors. For example, if the coefficient of discharge for the hydrant nozzle (on which the hydrant discharge was calculated) was lower than the stated 0.9, the calculated hydrant discharge and available fire flow would be lower than the stated figure. Another potential factor is that the calculated fire flow available at a hydrant is based upon a residual pressure of 20 psi at that hydrant, whereas the modeled fire flow is based upon maintaining 20 psi both at the hydrant and in the surrounding network. It is possible that the 20 psi limit is reached somewhere else in the network before the hydrant, so reducing the available fire flow.

For 4 hydrants the modeled fire flows were more than 70% higher than those derived from the fire flow test, with the greatest discrepancy being 88%. This could indicate a potential problem with the hydrant or with the surrounding network e.g. a flow restriction, such as a closed or partially closed valve, or that the model does not accurately reflect the local conditions e.g. a pipe diameter in the model is incorrect or the roughness of a pipe in the network is greater than that in the model, which would lead to greater headloss.

Although no ISO fire flow test was undertaken at the Peppers Restaurant hydrant, modeling suggested that the available fire flow (9,500 gpm) is more than adequate to meet its NFF (3,500 gpm) after taking into consideration some of the issues discussed above.

8.6.5 Hydrant Modeling (Future Conditions)

As discussed in the previous section, the available fire flow (either determined by the ISO fire flow test or by modeling) was insufficient to meet the NFF in approximately 60% of the 25 critical areas. In order to improve fire flows to these areas, the final step in the process was to apply the NFF for each property to the appropriate hydrant in the model to identify what improvements would be required to the water distribution system so that the available fire flow would meet the required NFF. Following discussions with the City, it was decided that the hydrants would be modeled for the 2020 Maximum Day Demand scenario. The reason for this is that by this date the majority of the water distribution system improvements identified in 8.3.6 to reinforce the system for pressure and capacity requirements will have been implemented. Modeling could then be undertaken on the improved system to ascertain if these improvements alone were sufficient to meet the NFF in all of the 25 critical areas, or if additional improvements were required beyond those already identified. The model was changed prior to modeling to reflect the decision by the City to install two 0.75 MG elevated water towers (Section 8.3.6.3.1).

The results of the modeling are given in Table 8-16 along with additional recommended system improvements. The table shows that additional improvements were not required in 10 areas in order to meet the NFF. The recommended improvements to the remaining 15 areas were reviewed by the City Fire Department, who concluded that only 7 areas required the recommended improvements and that the improvements recommended for 8 areas was not required for a variety of reasons. One of the main reasons is that quite a few of the hydrants selected closest to the high NFF properties are on small diameter mains (6-inch and 8-inch), as shown in Table 8-14. This has the effect of reducing available fire flows due to the higher head losses incurred as the high volume flows pass though the smaller diameter mains. In some instances the City Fire Department was able to identify a nearby hydrant located on a much larger main that was close enough to serve the high NFF property in the event of a fire. In the event of a fire this hydrant and not the hydrant selected for modeling would be used. In the case of Atwoods Warehouse, the company has moved to another location within the City and the property is now empty. The City will restrict any future usage of the property to activities that require a lower NFF than the 6,000 gpm required for its previous use.

Detailed descriptions of the improvements determined by the City Fire Department to be necessary are given in Table 8-17. This table includes the 2010 or 2020 Improvements identified in Section 8.3.6 that are required to be in place to gain the benefit of the recommendations. These improvements are shown in Figure 8-27 along with the 2010 and 2020 Improvements.

9.0 WATER RATES ANALYSIS / COST OF SERVICES STUDY

9.1 INTRODUCTION

Three factors must be considered in the financial analysis of a municipal water system:

- Expenses of the system.
- Revenue generating by water sales, typically in the form of water rates charged to customers.
- Payment of debt incurred to finance major system improvements.

For this plan, each of these factors has been preliminarily quantified. This plan provides multiple options for how raw water is obtained. Additional financial analysis will be required, perhaps in a follow-up study, to further quantify the system finances.

9.2 WATER RATE FACTORS

Recommendations for water utility rates must be based upon a variety of factors. These include:

- Employee labor, equipment and supplies to produce and distribute the water, as well as to maintain the system, including surface water treatment if a lake is constructed
- Energy costs (primarily electricity) to produce and distribute the water
- Employee labor and expenses to bill customers for water used
- Costs to replace older components in the system, including water lines and water wells
- Payment of debt incurred to finance major system components
- Contribution to the City's general fund to pay for City services that cannot be self-funded

Each of these factors has to be included in the water rates. In analyzing these factors for the City of Enid, a projection was made, using costs from recent years and discussions with the City staff, for the amount needed in the year 2010. These costs will need to be escalated for inflation and expansion effects in future years. An increase per year was assigned to each factor based on the type of factor and the potential volatility of the item's cost. Table 9-1 shows each of these factors, except for the payment of debt, which will be addressed separately below.

- Current costs for production and distribution were used for the base year cost. When treatment of surface water from a lake is required, an additional cost has been estimated based upon the proposed size of the treatment plant.
- Utility costs were based upon current costs expressed as dollars per 1,000 gallons of water billed. As most of this cost is for electricity, a higher per year increase was assigned to utility costs compared to other increases.
- Billing for all utilities is included on one monthly statement. This analysis assigns 45% of the billing costs to the water system.

- Older components of the City's system will require periodic replacement. This analysis includes a water line replacement program and a water well replacement program. The water well replacement program includes three water wells per year.
- Debt payment will be addressed separately.
- Because they generate revenue, the City's utility systems make a contribution to the City's General Fund. This analysis assigns 45% of the utility contribution to the water system.

9.3 SALES TAX PROJECTS

A portion of the City's sales tax revenue can possibly be dedicated to paying for a few water system improvements. Referring to the recommended capital improvement projects shown in Tables 11-1 through 11-4, the report recommends that the projects identified as Fire Flow Improvements be paid for with sales tax revenue. The amounts shown to be spent on Fire Flow Improvements prior to 2010 totals $223,000 for three projects. Between 2010 and 2015, four additional Fire Flow Improvements are shown, totaling $2,298,100. Rounding up, the total for Fire Flow Improvements is $2,600,000.

Installation of two 0.75 million gallon elevated water storage towers are assigned as sales tax projects in this analysis. The locations of these elevated tanks have already been chosen and are shown on the improvement drawings for this Master Plan. A budget of $7,520,000 has been assigned for installation of the two elevated tanks.

Previous sections of this plan recommended that water wells be replaced at a faster rate. As shown in item 4 above, the water rates include replacement of three wells per year. This analysis assigns additional water well replacements as sales tax projects. The budgeted amount for these replacements is $8,200,000. Continued use of the existing well fields may require a portion of the water to be treated for nitrate removal. This analysis assigns a future cost of $10,000,000 for this project as a sales tax project.

9.4 ESCALATION FACTOR

The costs for recommended capital improvement projects are shown in Tables 11-1 through 11-4. Each of these tables is based upon a different option for expansion of the raw water supply. The costs shown in these tables are estimates based upon 2008 construction prices for materials and labor. However, the time period for these improvements extends from 2010 through 2050. Therefore, an escalation factor must be applied to estimate the cost of each improvement in the year(s) in which construction will take place.

Control of the escalation rate is not available to Enid City Officials. Therefore, estimates of future cost escalations are highly variable. For the purposes of this analysis, an escalation rate of 2.5% per year, compounded yearly, is used to project future costs. Thus, projects set for construction in 2010 have two years of escalation above the estimated costs using 2008 construction prices. The resulting multiplier is 1.0506 for 2010 projects. A project assigned to 2020 will have 12 years of escalation, resulting in a multiplier of 1.3449 times the estimated costs shown in Tables 11-1 through 11-4.

9.5 LONG-TERM FINANCING

The City will either need to put money aside from current rates to save for major improvements, obtain grants or use long-term financing. A portion of future rates (or funds from other sources) will need to be dedicated to paying the debt service. The size of the major projects and the dates by which the projects need to be completed indicate that long-term financing will be required. There are several sources of long-term financing. To simplify this analysis, it is assumed that the long-term financing will be provided by the sale of municipal bonds. During the financial analysis portion of the Master Plan effort, the bond market has been so volatile that making predictions on future bond sales is very difficult. This analysis assumes a return to some form of normalcy by the time funding is required for major projects. Bond terms used in this analysis are:

- 30-year maturity
- 5% interest rate
- 1% cost of issuance fee
- Yearly debt payment from water rates

9.6 FUTURE WATER RATES

Sufficient revenues will need to be generated from rates to cover the yearly debt payments. Various trial rates were considered so that the money available each year would cover the operations cost of the water utility system as well as making the required debt payments. In addition, the rates were analyzed based on not doing any of the capital improvement projects. The purpose of this was to determine the extent to which the existing operations might be underfunded or overfunded if no rate changes are made.

For purposes of this analysis, a water system "fund" is assumed to collect all of the revenue and disperse all of the expenses. This fund gathers the revenue from water rates, the revenue from long-term financing, and the system expenses for operations, maintenance and improvements. A "fund balance" total is generated for each year, beginning with 2010. The purpose of this total is to show during which years there are sufficient funds generated by water rates to meet the water system financial obligations (positive fund balance), and during which years the revenue falls short (negative fund balance). Rates are then adjusted to maintain a positive running fund balance for the duration of the analysis.

Based on the findings of this analysis, the existing operations are underfunded from the existing rate system. This analysis recommends that the current "Incremental" rates (rates charged for each additional 1,000 gallons of water used beyond the base rate which includes the first 1,000 gallons) be raised. Table 9-2 compares the current rates (as they existed at the beginning of this study) with the rates that are recommended to be in-place at the beginning of 2010.

The estimates for yearly increases in operational costs shown in Table 9-1 cannot be accommodated by the estimated growth in the system's water usage. Therefore, this analysis shows that a yearly increase after the 2010 incremental adjustment will be necessary. This increase will apply to both the base rate and the incremental rate. Estimated increases vary from

2.25% per year during the first five years (from 2010 to 2015) to 3.50% per year from 2025 to 2030 as shown below:

- 2.25% to 2015
- 2.75% to 2020
- 3.25% to 2025
- 3.50% to 2030, with smaller percentages thereafter

When the capital improvement projects are considered, the rate increases will be higher. Two cases were preliminarily analyzed, the new wellfield option and the Hennessey Lake option. For the new wellfield option, rate increases per year are estimated as shown below:

- 5.50% to 2022
- 3.00% to 2035
- 2.50% to 2040
- 1.00% thereafter

Construction of Hennessey Lake is estimated to be more expensive than the new wellfield option and will require the additional cost of a surface water treatment plant. Reservoir construction also cannot be phased as can wellfield additions. Therefore, the initial rate increases are higher for this option in the earlier years, with smaller rate increases in later years as shown below:

- 7.00% to 2019
- 9.00% to 2025
- 0.60% to 2045
- 0.00% thereafter

10.0 RECOMMENDATIONS

Recommendations for each component of the water system are included within the various sections of this plan. The following are brief summaries of these recommendations.

10.1 EXISTING WELLFIELDS - INFRASTRUCTURE

- **Ames Plant:** Install an emergency backup system to enable water to be pumped during power outages.
- **Ames, Drummond and Enid Well Houses:** Increase the rate of replacement of the well houses.
- **Imo Booster Station:** Complete the retrofit that has already begun.
- **Ames Wellfield Collection Lines:** Replace transite pipes with ductile iron pipes.
- **Ames and Drummond Transmission Line:** Replace the existing line, or install a second line to complement it.

10.2 EXISTING WELLFIELDS - OPERATION

- Switch production from the Cleo Springs and Ringwood wellfields to the Drummond Wellfield, such that the percentage production from each wellfield is more in line with its percentage of usable water stored.

10.3 ALTERNATIVE WATER SUPPLY SOURCES

- **Surface Water Sources:** Conduct concept-level study on the use of surface water from Kaw Lake and the Turkey Creek Watershed (Hennessey and Lahoma lakes). This study should also consider the use of the proposed Sheridan Lake.
- **Groundwater Sources:** Conduct concept-level study on the development of additional groundwater wellfields.

10.4 EXISTING WATER RIGHTS AND PERMITS

- **Existing Water Rights:** Compare records to county records.
- **Groundwater Permits:** Amend Cleo Springs and Ringwood Wellfield permits and re-open Prior Right Final Order to correct errors.

10.5 EXISTING WATER PLANTS

- **Plant #1 Chemical Treatment:** Replace chemical treatment building and expose raw water pipeline.
- **Nitrate Removal:** Monitor nitrate concentrations and install nitrate treatment at Plant #1 and Plant #2 when appropriate.
- **Plant #1 Pumps:** Phase replacement of pumps for higher volume at a slightly higher head.
- **Plant #2 Pumps:** Phase replacement of pumps for higher volume at a slightly higher head.

10.6 DISTRIBUTION SYSTEM

General

- **Development Extensions:** 12″ DIP section line mains, as required.
  8″ PVC mains off section line mains to serve new developments within a section, as required.

2010 (29 MGD) Improvements

- **Plant #1 Zone:** 30″ to 24″ to 18″ DIP from plant to US 81/Phillips intersection.
  Parallel 6″ and 10″ PVC along Van Buren from Maine to Moore.
  Upgrade Gray Ridge Booster Station.
  Standpipe at US 81/Fox intersection, or new tower at Meadowlake Park.
- **Plant #2 Zone:** 12″ DIP cross-connection at Chestnut & Cleveland.

2020 (31 MGD) Improvements

- **Plant #1 Zone:** Extend 24″ DIP from 4th Street to 16th Street.
- **Plant #2 Zone:** 36″ DIP parallel from plant along Chestnut to Garland.
  24″ DIP parallel along Garland from Chestnut to Willow.
  18″ DIP parallel along Willow from Garland to Oakwood.

2050 (34 MGD) Improvements

- **Plant #1 Zone:** Extend 24″ DIP from 16th Street to 30th Street.
- **Plant #2 Zone:** Raise the operating range in the elevated storage tank from 1,460 feet to 1,479 feet.
  12″ DIP parallel along Oakwood from Willow to Purdue.

Improvements to meet Needed Fire Flow (2020)

- **Autry Tech:** 12″ DIP parallel along Cleveland from Chestnut to Willow.
  12″ DIP parallel along Willow from Cleveland to Wagon Trail.
  18″ DIP parallel along Willow from Wagon Trail to Autry Tech.
- **Enid HS:** 8″ PVC parallel along Owen K Garriott from Pierce to Van Buren.
  8″ PVC parallel along Wabash from Van Buren to Monroe.
  8″ PVC parallel along Monroe from Wabash to York.
- **Best Western:** 8″ PVC parallel along Van Buren from new water tower to Richland.
- **First Baptist:** 6″ PVC parallel along Maine from Washington to Adams.
- **Longfellow JHS:** 10″ PVC parallel along 11th from Walnut to Maple.
  6″ PVC parallel along 11th from Maple to Randolph.
  6″ PVC parallel along Randolph from 11th to 10th.
  6″ PVC parallel along 10th from Randolph to Broadway.
- **Airport:** 8″ PVC parallel to existing 6″ on S side of Jerauld Gentry.
- **Boys Market:** 8″ PVC parallel along Van Buren from Cherry to Hackberry.

11.0 PRIORITIZED CAPITAL IMPROVEMENTS PROGRAM

A brief summary of the recommendations for each component of the water system is given in Section 10. From these recommendations a prioritized capital improvement program has been developed to aid the City in the planning and funding of future capital works related to the water system. For each recommendation a planning-level cost estimate has been developed. The exception to the above is that costs have not been developed for the following items:

- Increasing the rate of replacement of the well houses (Section 4.12.2). The replacement of well houses is already being undertaken and budgeted for by the City. The increase in budget required to increase the rate of well house replacement can be easily established by the City by pro-rata of the existing budget.
- Installation of two 0.75 MG elevated water towers (Section 8.3.6.3.1). The City has already developed a cost estimate of $6m to install these towers.
- Complete retrofit of the Imo Booster Station. The City has already completed some of this work.

The prioritized capital improvement program includes the development of a new water supply (Section 5.8.6). As four alternative water sources have been considered (pumping water from Kaw Lake, constructing a dam on Turkey Creek either at Hennessey of Lahoma, or expanding the well fields), a capital improvement program has been developed for each water supply option. In developing each program, basic principles were followed in order to prioritize work:

1. It is likely that a new water supply is required prior to 2040. For the surface water options, this will require a significant period of conceptual study, design work and purchasing of land before construction can commence. It is therefore assumed that studies and design work will commence in 2010 and will continue through to 2015 and beyond, with regulatory approval, purchasing of land and construction work occurring over the following 15 years, finishing in 2030. This provides a 20-year time frame in which to complete the project, and provides a time buffer before aquifer levels become critical. For the expanded groundwater option, it is assumed that additional well fields will be developed evenly over a 25-years period (2010-2035).
2. It is unlikely that all of the 2010 and 2020 Improvements identified in Section 8.3.6.3 will be required prior to these dates and so some of these improvements occur later. Some of the increase in demand due to this period has been assumed to occur by the City supplying water to the rest of Garfield County (Section 8.1.3). Currently there are no plans for the City to expand their water distribution system beyond the City boundaries, so this demand increase is unlikely to happen in the near future. In addition, the improvements have been based upon a maximum day demand being reached in the system, and the two dates are arbitrary dates that are dependent upon the all of the predicted growth occurring. Although some of the improvements have been scheduled beyond 2020, the 2010 Improvements have been scheduled to be implemented before the 2020 Improvements.
3. Some improvements required to improve fire flows require the 2010 Improvements to be implemented and have been scheduled accordingly. The remaining fire flow improvements

are small value projects and it has been assumed that these can be undertaken within the City's current water main replacement program by the end of 2010.

4. The remaining improvements have been scheduled such that capital expenditure commences slowly in 2010 and then peaks around 2020-2025 when the bulk of the required capital is being spent on construction of the new water supply, before declining to completion in 2050.

The prioritized capital expenditure programs based around the four water supply options are given in Tables 11-1 through 11-4. The improvements have been grouped by each component of the water system. The costs are based upon present day costs and include land purchases, 10% for engineering and 25% for contingencies. For each option the cost of the improvements not related to the water supply amount to $61.4m, with the bulk of this value account for by the replacement of the 30-inch transmission line from the Ames Wellfield ($28.2m).

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