Water System Facility Plan North Liberty, Iowa

June 2013

Prepared for the

City of North Liberty

FOX Engineering Associates, Inc.
Ames, Iowa

Certification

Water System Facility Plan
North Liberty, Iowa
June 2013

LAURIE A. TWITCHELL
LICENSED
14861
IOWA
PROFESSIONAL ENGINEER

I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly licensed Professional Engineer under the laws of the State of Iowa.

6/14/13

LAURIE A. TWITCHELL, P.E. — DATE
License number 14861

My license renewal date is December 31, 2014.

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EXECUTIVE SUMMARY

1.1 Background

The demand for water in North Liberty continues to rapidly increase as population growth once again exceeds optimistic projections from the most recent water study completed in 2006 by close to 30%. This same phenomenon was experienced from the previous 2001 to 2006 report. This year's anticipated water demand may reach levels not previously projected to occur until 2018. Demands are projected to exceed the existing water plant's softening capacity, in conjunction with the ASR well, in the next 2-3 years when the population reaches approximately 18,000. Beyond that, the water plant will have enough capacity to operate another 2-3 years (≈ 2018) to a projected population of approximately 20,500; however the finished water will be a lower quality (i.e. higher hardness). It is recommended that the city begin the planning process for water system improvements in the near-term to maintain high-quality softened water to residents.

The need for water system improvements is due to the city's escalating population. An Aquifer Storage and Recovery (ASR) well was constructed as a recommendation of the 2006 report as a lower-cost alternative to lengthen the life of the existing water plant and push-back the date when a more costly plant expansion or new plant would be required. At the time of the 2006 report, it was projected that the ASR well may help the water plant meet demands for 15 to 20 years. Since current projected population and water demands are estimated to occur approximately 5 years earlier than 2006 projections, this reduces the time frame for when improvements will be needed to 10 to 15 years which is in line with the findings of this report. The ASR well has served its purpose in delaying the need for increased treatment capacity; however, rapidly increasing population and water demands have reduced the time period for when additional capacity will be needed.

1.2 Design Conditions

North Liberty's current population is estimated to be approximately 15,500. Projections provided by the City show that the population is anticipated to grow to around 28,100 by 2027 and 36,500 by 2037. This high rate of growth will accelerate the need for infrastructure improvements, including water supply, treatment, and storage.

Current average day and peak day water demands are approximately 1.14 million gallons per day (MGD) and 1.80 MGD respectively. This translates to about 74 gallons per capita per day average (gpcd) and 116 gpcd peak. Over the past ten years, average usage has ranged from as low as 65 gpcd to as high as 84 gpcd, but has been around 70 gpcd for the last five years. Peak day usage has ranged from 102 gpcd to 164 gpcd over the last ten years, but has been around 110 gpcd for the last five years. As a conservative approach, water demands were projected using 80 gpcd average and 150 gpcd peak. This is higher than the last five years, but is within the range over the last ten. Because of North Liberty's rapid growth, this conservative approach was selected to provide some flexibility in meeting future demands. The projected demands for a population of approximately 28,100 (Phase 1 design) are 2.25 MGD average and 4.22 MGD peak. For approximately 36,500 population, the projected average and peak demands are 2.92 MGD and 5.48 MGD respectively. Throughout the report, these are referred to as Phase 1 and Phase 2 demands.

1.3 Existing Water Treatment Facilities

The existing treatment plant is an ion-exchange (IX) softening plant. IX softening is the technology that is used in most home water softeners. In general, the process involves passing the untreated water through a vessel packed with the ion-exchange resin beads. During the time that the water is in contact with the IX resin, hardness minerals (calcium and magnesium molecules) are transferred to the resin in exchange for sodium molecules.

The plant's current capacity is approximately 1.56 MGD based on 20 hours per day of operation, although it was designed for a capacity of about 1.36 MGD. At this rate of treatment, it produces an average hardness of around 170 mg/L, which is considered moderately hard water (most softening plants target 80 -120 mg/L). The City has expressed a desire to provide a higher level of treatment to around 120 mg/L hardness. The plant has operated as high as 1.8 MGD, but the quality of water rapidly deteriorates as flows increase (but the finished water still meets all primary drinking water standards). The main limitation on the plant's capacity is the softeners. Additional softening capacity will be needed in order to meet increasing demands and the City's goal of providing higher quality water. Related to the softeners is the brine storage tank, which is inadequately sized for the current capacity. The other component of the treatment plant that needs to be addressed is the raw water detention tank. This is an above grade steel tank through which all the water that is treated must pass through. A recent inspection has revealed that this tank is in poor structural condition and should be replaced in the near future.

Recent improvements to the water system include the addition of an aquifer storage and recovery (ASR) well. Treated water is injected into the well during periods of low demand then recovered during periods of high demand. This allows treatment plant capacity to be closer to the average demand rather than the peak demand, and in North Liberty's case, allowed the major expense of constructing a new treatment plant to be delayed by several years. The peak day capacity of the ASR is about 1.32 MGD, but in order to fully utilize it, the treatment plant must have enough additional capacity to inject an adequate volume of water during low demand periods. An analysis of the ASR in conjunction with the existing treatment plant (rated capacity 1.56 MGD) reveals that they can provide adequate service for a population of about 19,000 based on current demands. This was similar to the analysis shown in the 2006 report. One noted deficiency of the ASR is lack of an adequate back-up if the pump should fail. The original intent was to provide back-up for this well through connection to a neighboring system, such as Coralville or Iowa City. A connection does existing to Coralville, but a service agreement is not in place for an emergency situation.

Figure 1.1 shows the projected water demands for the average day based on the conservative design approach of 80 gpcd. It also shows projections based on average per capita demands over the last five years (70 gpcd average). The average day demand plant capacity (1.5 MGD) illustrated assumes the plant maximum capacity must be approximately 20% higher than the average day demand to adequately fill the ASR well. The ability of the existing plant to meet water demands is controlled by the average day demand and the ability of the existing plant to have enough excess capacity to fill the ASR well during the injection season. The graph is based on a maximum treatment capacity of 1.8 MGD with reduced softening over the next few years. Figure 1.1 illustrates that the plant capacity will likely be exceeded in the next 3 to 5 years.

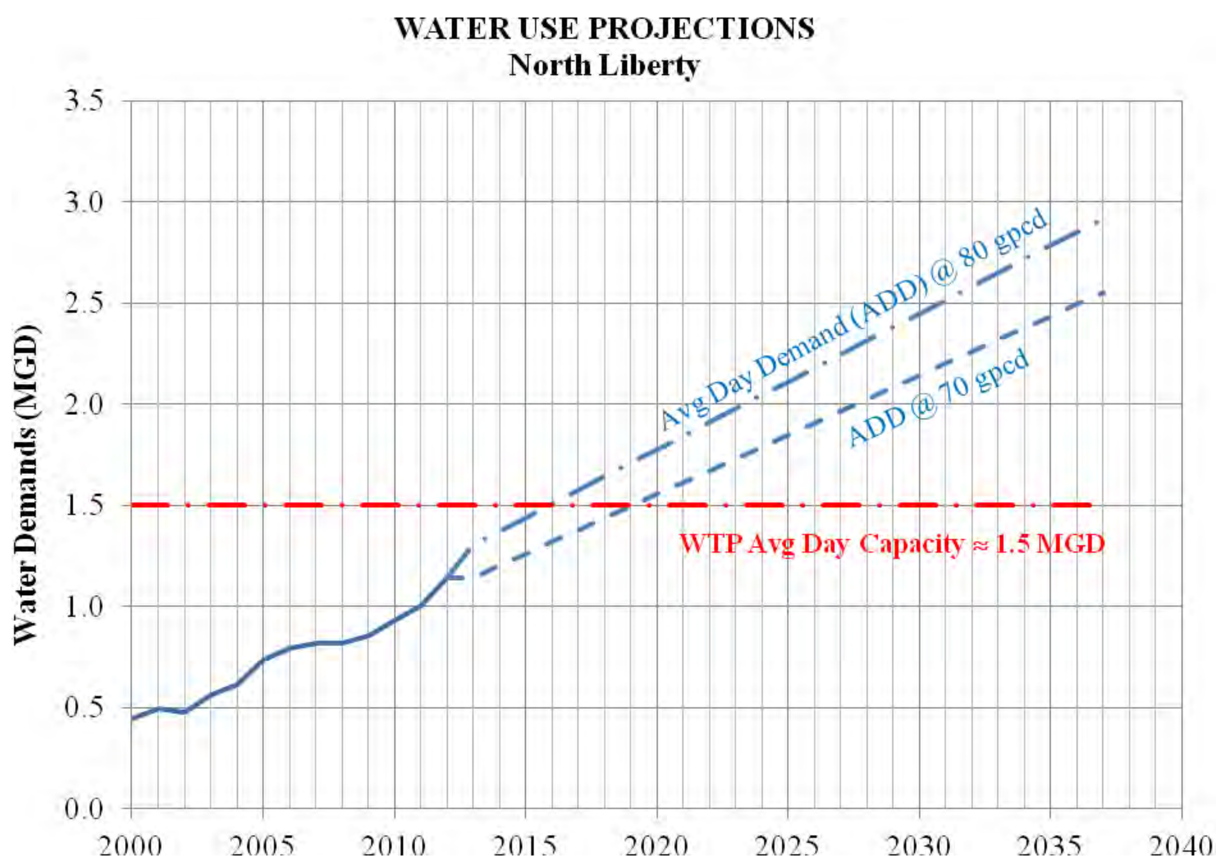

Figure 1.1. Water Use Projections

1.4 Water Supply and Treatment Alternatives

In order to meet the growing water demands of the City, and provide a higher quality finished water, several options were considered. The options are summarized in Section 7, and include water supply, treatment, and storage. Section 8 presents an evaluation of the alternatives on both capital and life-cycle costs, as well as non-economic factors. The options included upgrading the water treatment facilities to meet Phase 1 (population 28,100) and Phase 2 water demands (population 36,500).

In general, the alternatives considered within this report fall into three main categories including: (1) upgrading the existing water treatment plant to the extent possible (Phase 1) prior to building a new RO treatment plant (Phase 2) at another location, (2) maintaining the existing plant at its current capacity and building a new supplemental RO treatment facility at another location, or (3) replacing the existing water treatment plant with a new RO treatment facility at another location.

Various treatment and supply alternatives were considered for the new plant construction options and the viable choices were narrowed down to RO softening as the most suitable treatment technology for the City of North Liberty. IX softening which the plant currently uses was ruled out for the new plant options due to several factors. The primary disadvantage of the IX technology is the addition of sodium to the drinking water and chloride to the wastewater. Chloride added to the wastewater during softener regeneration leads to elevated chloride levels which is a major concern at the wastewater plant due to chloride effluent limits. Another downside to IX softening is the large operating cost for purchasing brine salt.

Table1.1. summarizes the capital cost opinion for the alternatives presented in this report. A summary of the present worth analysis, including O&M costs, is presented in Table 1.2. The results of the present-worth analysis found that Alternatives 2, 3 and 4 were all within 10% and considered equivalent for purposes of this evaluation.

Table 1.1 Summary of Opinion of Probable Capital Cost.

Description		Phase 1 (mil $)	Phase 2 (mil $)	Total (mil $)	Rank
1	Optimize Existing WTP	Not Feasible			
2	Upgrade WTP within Site (Ph. 1) & New RO WTP at New Site (Ph. 2) (3.0 to 4.2 MGD)	$12.2	$15.3	$27.5	1
3(1)	New RO WTP at New Site & Supplement Existing WTP (1.5 to 2.7 MGD)	$17.6	$7.8	$25.4	1
4(2)	New RO WTP at New Site & Replace Existing WTP (3.0 to 4.2 MGD)	$19.2	$8.0	$27.2	1

(1) Silurian wells piped to existing WTP only.

(2) Abandon Silurian Wells

Table 1.2. Water Supply and Treatment Alternatives

Alternative		Present Worth: Ph. 1 & 2 Capital Cost (Mil $)	Salvage (Mil $)	O&M Cost (Mil $)	**Total (Mil $)**	Rank
1	Optimize Existing WTP		Not Feasible			
2	Upgrade WTP within Site (Ph. 1) & New RO WTP at New Site (Ph. 2) (3.0 to 4.2 MGD)	$27.5	($5.6)	$24.9	$46.8	1
3[(1)]	New RO WTP at New Site & Supplement Existing WTP (1.5 to 2.7 MGD)	$25.4	($5.0)	$25.2	$45.5	1
4[(2)]	New RO WTP at New Site & Replace Existing WTP (3.0 to 4.2 MGD)	$27.2	($5.9)	$23.2	$44.4	1

(1) Silurian wells piped to existing WTP only.

(2) Abandon Silurian Wells

The water treatment and supply options were also evaluated on a noneconomic basis considering factors such as land requirements, system control, operational requirements, reliability, finished water quality and others as presented in Section 8.7. A combined analysis was performed including monetary factors given a 75% weighting and non-monetary factors weighted at 25%. The results of the combined analysis are presented in Table 1.3.

Results of the combined analysis showed that Alternative 4 – New RO Plant at a New Site and Replacement of Existing WTP appeared to be the most beneficial and cost effective.

Alternative 4 involves abandoning the existing plant and building a new reverse osmosis (RO) plant. The plant would be built in phases, with Phase 1 sized for a population of about 28,100 and Phase 2 for a population of about 36,500. The capital cost opinion for Alternative 4 (see Table 1.3), including water supply, treatment, and storage for a population of 36,500 (both Phase 1 and Phase 2), is approximately $27.2million.

The advantages of Alternative 4 are that it utilizes a membrane technology that provides a higher quality water and does not require the addition of salt as part of the softening process. Membrane technology can also be relatively easily expanded in modular units. The disadvantage of Alternative 4 is that membrane technologies require more raw water, since they have a 20-25% loss of water during water production as compared with 5-10% loss for cation exchange softening. Alternative 4 also has the highest initial capital cost.

Table 1.3 Summary of Combined Analysis of Alternatives.

	Description	Total Net Present Worth (NPW) Rank	Weighting Factor (%)	(1) Total NPW Weighted Rank	Non-Monetary Factors Rank	Weighting Factor (%)	(2) Total Non-Monetary Factors Weighted Rank	Total Combined Weighted Rank (1)+(2)	Overall Rank
1	Optimize Existing WTP	Not Feasible							
2	Upgrade WTP (Ph. 1) & New RO Plant at New Site (Ph. 2)	1	75%	0.8	3	25%	0.8	1.5	**3**
3	New RO Plant at New Site & Supplement Existing WTP	1	75%	0.8	2	25%	0.5	1.3	**2**
4	New1 RO Plant at New Site & Replace Existing WTP	1	75%	0.8	1	25%	0.3	1.0	**1**

(1) Rating Factor, 1 = worse; 5 = best.
(2) Lower Ranking is most Favorable
(3) Shaded area is the recommended option.

Because of the rapid population growth in North Liberty, actual population served will have the greatest impact on when the improvements will be required. Table 1.4 shows the improvements and the corresponding population when the improvement(s) will be needed. The table also shows the estimated year, based on population projections. As can be seen in Table 1.4, Alt. 4 requires a total capital investment for Phase 1 improvements of $19.2 million phased over a 10 year period based on when improvements would be necessary

For Phase 1, the first step (Phase 1A) would be to begin RO pilot testing in approximately one year, then proceed forward to design and construction of a new RO plant and new Jordan well with plant start-up in late 2017. Additional elevated storage (Phase 1B) could be delayed until approximately 2020 while the construction of another Jordan well (Phase 1C) could be delayed until the end of the planning period.

Table 1.4. Phased Implementation of Improvements

ALTERNATIVE 4 – New RO WTP at New Site to Replace Existing WTP				
	Capital Cost ($ Million)	Population[(1)]	Estimated Year[(2)]	Total By Population (or Est. Year)
Phase 1				
<u>Phase 1A:</u>				
Treatment Plant	$9.5	19,700	2017	$13.2
Raw Water Main	$1.3			
New Jordan Well	$2.4			
<u>Phase 1B:</u>				
Elevated Storage	$2.8	22,000	2020	$2.8
<u>Phase 1C:</u>				
Raw Water Main	$0.8			
New Jordan Well	$2.4	27,500	2023	$3.2
Total Phases 1A – 1C	**$19.2**			
Phase 2				
Phase 2A:				
Treatment Plant	$2.0	29,000	2028	$2.0
Phase 2B:				
Elevated Storage	$2.8	31,000	2030	$2.8
Phase 2C:				
Raw Water Main	$0.8	33,000	2033	
New Jordan Well	$2.4	33,000	2033	$3.2
Total Phases 2A-2C	**$8.0**			

Notes: (1) Population when improvement is recommended to be in place.
(2) Estimated year when improvement is recommended, base on population projections.

1.5 Impact on User Rates

Based on the proposed improvements, the City Administrator and financial advisors performed a rate analysis to determine the impact on user rates. Refer to Appendix B for details of the rate analysis and proposed increases. The rate projections were prepared based on projected revenues and expenditures through fiscal year 2025. The projections were based on estimated revenue increase of 2% per year. The projections show that rate increases will be necessary through fiscal year 2021 to fund the needed improvements. Rate increases will vary from year to year, but will range between 3% to 15%.

1.6 Recommendations

In moving forward, the City should begin planning for implementation of the recommended alternative (Alternative 4 – New RO Plant at New Site and Replace Existing WTP) in the near future. The recommended option utilizes reverse osmosis (RO) technology that requires 90-days of pilot testing on a small scale to verify applicability of the process to the City's source water and to determine final design parameters and obtain acceptance from the State. Table 1.5 shows the proposed implementation schedule. See Section 9 for the detailed recommendations.

While the City is moving forward with the planning of water system improvements, it is also recommended that the City have Shive Hattery re-inspect the interior of the Raw Water Detention Tank at the plant and plan for routine inspections over the next few years until the new plant is built and the tank can ultimately be abandoned. The existing Raw Water Detention Tank is a 29,000 gallon welded steel tank that was installed with the original plant in the 1970s. Shive Hattery inspected the existing tank in May 2011 and recommended that the tank be taken out of service and the roof cap repaired in the next couple years. Repairing the tank is a major undertaking which would require the tank to be out of service for an extended period with special accommodations in place to operate the existing plant without the tank. If the new plant is constructed within the recommended time frame, repair of the tank may not be necessary.

Table 1.5. Proposed Implementation Schedule

Approve Facility Plan and Submit to IDNR	June 2013
Submit SRF Intended Use Plan Application	June 2013
R.O. Pilot Testing	May - Aug. 2014
Design Engineering	Oct. 2014 - June 2015
Bidding	August 2015
Construction/New WTP Start-Up	Sept. 2015 – June 2017
Construction/New WTP Final Acceptance	September 2017

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INTRODUCTION

2.1 Purpose and Scope

The City of North Liberty continues to experience very rapid growth and, as a result of this, has already experienced water supply demands nearing the capacity of the city's current water supply wells and treatment facilities. The recent addition of an aquifer storage and recovery well (ASR) will help offset the peak treatment demands, but the population projections show that the design population intended to be served by the ASR well will be reached in the next 4-6 years. That, coupled with the city's desire to provide a higher quality finished water, will make expansion of the treatment facilities necessary in the near future.

As a critical component of City services to its citizens, the City needs to insure that the current and future water requirements of the community will be met by the City's Water Supply System. This preliminary engineering report was prepared to review the existing and projected water system requirements of the City and to determine what changes or additions may be needed to allow the City Water Department to meet those needs. Where deficiencies are discovered in the existing facilities, alternatives for overcoming those deficiencies are considered and recommendations made for improvements and additions.

The scope of the study as agreed to between the City and the Consultants was limited to a review of the raw water supply, raw water transmission facilities, treatment facilities, high service pumping, and storage. The distribution system was considered in only a very limited way as to the ability of water mains in close vicinity to the water plant to carry increased treated water supplies into the broader system.

2.2 Organization of This Report

This facility plan report is divided into nine sections. Section 1 contains an executive summary of the work embodied in this report. This Section 2 consists of introductory material. Section 3 establishes the design conditions, including water demand projections, on which the facility planning is based. Section 4 presents a description of the existing facilities and a discussion of the evaluation of those facilities. Section 5 contains a description and evaluation of the storage facilities and distribution system. Section 6 discusses the water supply source investigation and the various sources considered. The description of the proposed alternatives for meeting the future water demands are outlined in Section 7. Section 8 presents an evaluation of the various alternatives considered, including planning level capital, and operation and maintenance cost opinions as well as non-economic considerations. Recommendations are included in Section 9. Several pieces of supporting information are found in the Appendices.

2.3 General Description of the Water System

The North Liberty Water Supply system consists of several wells, raw water transmission lines, treatment equipment for iron removal and softening, pumping systems, finished water storage and water distribution lines. The ASR well is used to meet peak day demands during high demand periods. These various components of the system will be discussed below in varying levels of detail as they relate to the scope of this study.

The current system can provide treated water to the system at a peak day rate of about 1.56 Million Gallons per Day (MGD) based on 20 hours of operation. This accounts for periods of time when the plant cannot operate at full rate due to softener regeneration or filter backwash. Peak day demands required the plant to operate at up to 1.80 MGD during recent summer months, but the quality of water at that high of flow is greatly reduced. At the time the plant was upgraded in 2001, this capacity was believed to be sufficient to provide for peak day demands through the year 2010 at which time population levels in the City were expected to reach 10,000 people. With the addition of the ASR well in 2009, the system was anticipated to handle a population of about 19,000 to 22,000 people, depending on the production capability of the plant and the desired water quality. That population wasn't anticipated to be reached until year 2022 at the earliest. Population growth has been even faster than anticipated. The City has estimated 2012 population at 15,500, and projections show that a population of 19,000 will be reached in next 3 to 4 years. Expansion of the water system will be necessary in the near future.

2.4 Planning Area

The City of North Liberty is a community of approximately 16,000 with an incorporated area of approximately 6.5 square miles. The developed section of the City is located north of the Coralville/Iowa City metropolitan area east of Interstate Highway 380. The service area for water supply consists of the entire City of North Liberty (see Figure 2.1) and adjacent developable areas.

STYLE:

PATH:

VICINITY MAP

WATER SYSTEM FACILITY PLAN
NORTH LIBERTY, IOWA

FIGURE:	2.1	
REVISION	NO.	DATE
DRAWN JBA	PROJECT NO. 3373-012A	DATE 12-21-12

FOX Engineering Associates, Inc.
414 South 17th Street, Suite 107
Ames, Iowa 50010
Phone: (515) 233-0000
FAX: (515) 233-0103

FOX engineering

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DESIGN CONDITIONS

3.1 Planning Period

Municipal facilities such as water treatment plants are capital intensive, cannot be easily expanded in capacity, and have a long service life. For these reasons, when planning major modifications to an existing facility or designing a new facility, a planning period of 15 to 20 years is usually selected. Because of the very high rates of growth in North Liberty and the financial impact on current users of developing water supply facilities with capacity significantly in excess of current demands, we have considered a 25-year planning period from the time of the report preparation. This allows both a 20-year and a 10-year design life after installation of recommended Phase 1 and Phase 2 improvements assuming Phase 1 improvements would be in operation in approximately 5 years. Throughout this report the Phase 1 (10 year) design will refer to population and flow demands projected for the year 2027 and the Phase 2 (20 year) design will refer to population and flow demands projected for the year 2037. In reality, the actual rates of growth experienced, in both population and per capita water use, will determine when additional facilities may be required regardless of the passage of time.

3.2 Population Projections

Water demand for domestic and commercial use is very closely related to the population served. In North Liberty, most of the water demand in the past several years is from residential and light commercial usage, and that trend is anticipated to continue in the future. The future water demand can therefore be based on the projected future population.

3.2.1 Historical and Projected Population

Historical population data for North Liberty was obtained from the U.S. Census Bureau for 1970 through 2010. Table 3.2.1 summarizes this data and the 2012 population estimate provided by the City.

Table 3.2.1 Historical Population Data

Year	Population
1970	1,055
1980	20,46
1990	2,926
2000	5,367
2010	13,374
2012	15,500*

*Estimate provided by city

Projected population data was based on an estimated population growth of 840 persons per year through the year 2037 in accordance with information obtained from the City of North Liberty. This results in a population growth rate that varies from approximately 5.4% in early years to 2.4% in later years through the year 2037. The historical and projected population data for the current facility plan are presented in Table 3.2.2 and graphically presented in Figure 3.1. Table 3.2.2 also shows the population projections from the 2006 facility report. The 2006 and the current 2013 report are primarily based around design populations rather than time, since the actual rate of growth for the city is difficult to predict. Population based planning is the best approach for cities with large growth rates. An estimate of time is presented to aid in planning, but it is only an estimate. A prime example is the underestimation of the city's growth rate in the 2006 facility report. The city's population in 2012 is actually 26% higher than the population as predicted in the 2006 facility plan with a difference of over 3,100 persons.

Table 3.2.2 Projected Population Data

Year	*2006 Facility Plan Populations*	2012 Facility Plan Update Populations	% Population Change from 2006 Report
2012	*12,318*	15,500	+26%
2015	*14,076*	18,020	+28%
2020	*17,580*	22,220	+26%
2025	*21,957*	26,420	+20%
2030	*NA*	30,620	NA
2037	*NA*	36,500	NA

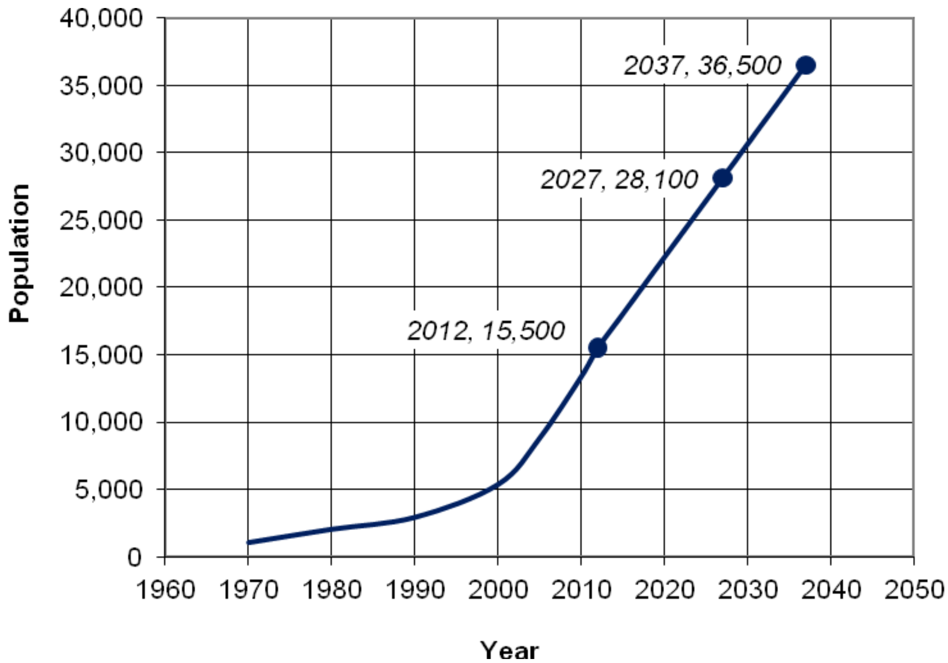

Figure 3.1. Population Growth for the City of North Liberty.

3.3 Existing Water Demands

An evaluation of existing water demands is important to assess current conditions and provide a basis of projecting future water usage for use in planning. Table 3.3.1 summarizes the total water use for the past 13 years from 2000 to 2012. The data presented represents the volume of treated water pumped into the water system which includes both water sold and water losses due to leakage and other factors. It does not include miscellaneous water used in the treatment process that is sent to waste such as filter backwashing, softener regeneration etc.. The table shows both actual water use for average day and peak day for each year and also the related per capita usage based on estimated populations for each year. Observation of the table will show that overall per capita usage has significantly declined in the past 5 years. This phenomenon may be a result of water conservation due to past water rate increases and also the trend of low-flow fixtures and appliances used in new homes construction.

Table 3.3 shows that as the population has increased steadily, so has the average and peak day water usage, even while the per capita water usage has declined. The average and peak day flows have each increased approximately 8% per year on average since 2000. Per capita flows have been notably lower in the past five years. Particularly for the peak day per capita flows since 2008.

Table 3.3.1 Historical Water Use in North Liberty

Year	Estimated Population	Average Day (GPD)	Peak Day (GPD)	Average Per Capita Use (GPCD)	Peak Per Capita Use (GPCD)	Peak:Avg Ratio
2000	5,367	447,251	838,000	83	156	1.9
2001	5,957	497,433	938,000	84	157	1.9
2002	6,268	474,003	849,000	76	135	1.8
2003	6,866	561,378	989,000	82	144	1.8
2004	7,637	614,492	1,053,000	80	138	1.7
2005	8,806	738,852	1,443,000	84	164	2.0
2006	9,993	791,589	1,552,000	79	155	2.0
2007	10,983	819,060	1,759,000	75	160	2.1
2008	11,761	819,459	1,321,000	70	112	1.6
2009	12,413	853,490	1,383,000	69	111	1.6
2010	13,374	929,660	1,395,000	70	104	1.5
2011	14,437	1,002,595	1,579,000	69	109	1.6
2012	*15,500*	1,140,708	1,799,000	74	116	1.6

IDNR guidelines recommend 100 GPCD for average usage and 200 GPCD for peak usage in planning. In comparison to IDNR recommendations, the current per capita water usage is very low. Average per capita water usage based on finished water pumped to the system is approximately 70 GPCD based on data from the last 5 years as compared to 83 GPCD in the 2006 report. Similarly, peak day per capita usage is approximately 111 GPCD as compared to 154 GPCD in the 2006 report. This represents a 16% decrease in average per capita water demand and a 28% decrease in peak day per capita water demand since 2008 which is significant.

Figure 3.3 illustrates the 2012 water demand based on daily records. The figure shows that there were several months when the daily demand exceeded the existing treatment plant capacity based on the limitation of the existing softeners. During these days, the treatment plant was operated at a higher flowrate and the quality of finished water was reduced with respect to elevated hardness levels.

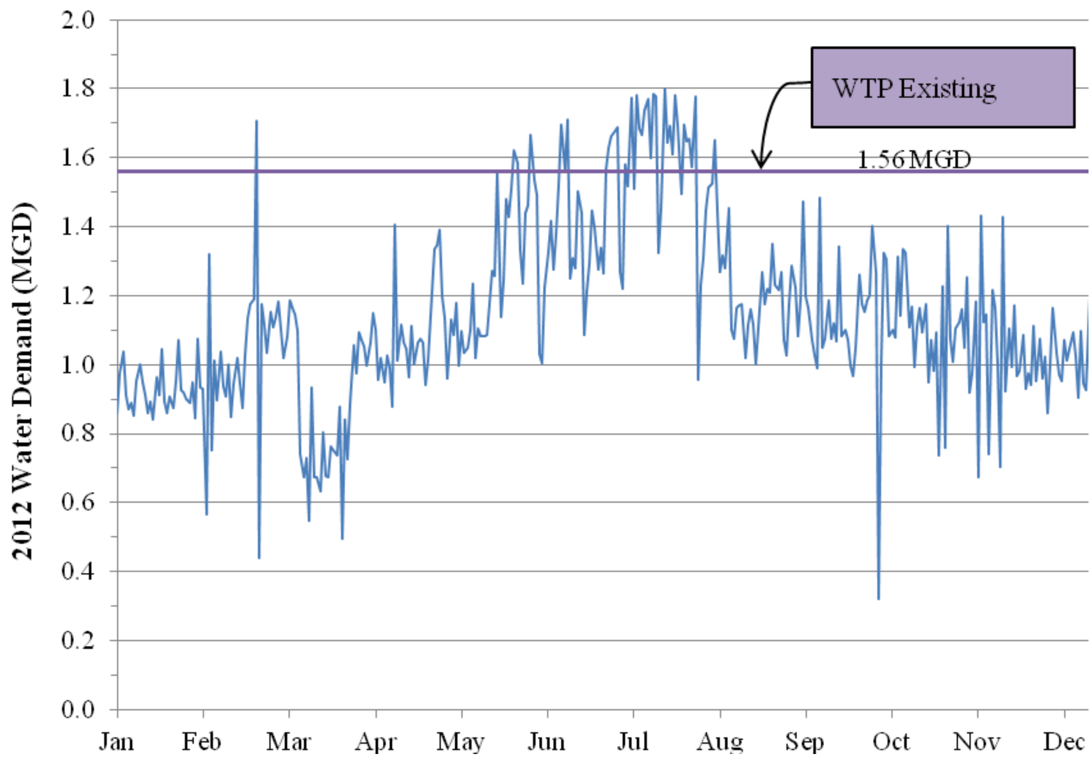

Figure 3.3. 2012 Daily Water Demands

3.4 Projected Future Water Demands

Projected future water demands are summarized in Tables 3.4.1 and 3.4.2. Table 3.4.1 shows two sets of projections, one based on historical water use and water use projections from the 2006 planning report and the second set shows current projections. The basis for the future water use projections includes the following considerations:

1. For planning in the 2013 report, the average day water demand (ADD) is based on the projected population times an average per capita use of 80 GPCD which includes a 14% allowance over historical water usage records for the past five years. This is similar to what was used in the 2006 report. The peak or maximum day water demand (MDD) is based on a peak day per capita use of approximately 150 GPCD. While this is significantly higher than current peak demands, it is similar to what was used in the 2006 report and provides some level of conservatism.

2. The industry standards recommended by the IDNR in predicting water use is much higher than the recent water usage trends at North Liberty. Adding a small 14% increase allows some conservatism in the event that water usage trends would increase in the future. The selected design values are still lower than IDNR recommended values, but are appropriate for North Liberty based on historical records.

The water demand projections described above result in a 10-year (2027) average day demand (ADD) of approximately 2.25 MGD at a population of 28,100 persons. The maximum day demand (MDD) for the same period is projected at approximately 4.22 MGD. It is important to note that the actual population growth rate may occur at a faster or slower rate than assumed within this report and the best way to refer to future water use projections is actually in terms of future population, although reference will also be made to a design year throughout this report to provide a basis for future planning.

For the design population of 36,500 persons, which is estimated to occur in in approximately 20 years (2037), the projected ADD is approximately 2.92 MGD and the projected MDD is approximately 5.48 MGD. Table 3.4.2 summarizes the projected water demands that will be used for the 2013 planning report including the water treatment plant design capacities for scenarios considered without the operation of an aquifer storage and recovery well (ASR) well. A design treatment plant capacity of **4.22 MGD for Phase 1 (population 28,100)** improvements and **5.48 MGD for Phase 2 (population 36,500)** improvements will be used throughout this report to evaluate options that don't include an ASR well.

For evaluations that include the operation of the ASR well in conjunction with a water treatment facility, the design water demands presented in the section are not the same as the required treatment plant capacities. The required future treatment plant capacities for ASR wells scenarios are discussed and presented in Section 3.4.1.

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Table 3.4.1 Historical and Projected Water Demands

Year	2006 Report Projections			2013 Projections			
	Population	Annual Average (GPD)	Max Day (GPD)	Population	% Change from 2006 Report	Annual Average @ 80 GPCD (GPD)	Max Day @150 GPCD (GPD)
2012	*12,318*	*1,022,400*	*1,896,984*	15,500	+26	1,140,708	1,799,000
2013	*12,878*	*1,068,884*	*1,983,230*	16,340	+27	1,307,000	2,451,000
2014	*13,464*	*1,117,481*	*2,073,398*	17,180	+28	1,374,000	2,577,000
2015	*14,076*	*1,168,287*	*2,167,665*	18,020	+28	1,442,000	2,703,000
2016	*14,716*	*1,221,403*	*2,266,218*	18,860	+28	1,509,000	2,829,000
2017	*15,385*	*1,276,934*	*2,369,251*	19,700	+28	**1,576,000**	2,955,000
2018	*16,084*	*1,334,990*	*2,476,969*	20,540	+28	1,643,000	3,081,000
2019	*16,815*	*1,395,685*	*2,589,585*	21,380	+27	1,710,000	3,207,000
2020	*17,580*	*1,459,140*	*2,707,320*	22,220	+26	1,778,000	3,333,000
2021	*18,379*	*1,525,480*	*2,830,408*	23,060	+25	1,845,000	3,459,000
2022	*19,215*	*1,594,836*	*2,959,093*	23,900	+24	1,912,000	3,585,000
2023	*20,088*	*1,667,345*	*3,093,628*	24,740	+23	1,979,000	3,711,000
2024	*21,002*	*1,743,151*	*3,234,280*	25,580	+22	2,046,000	3,837,000
2025	*21,957*	*1,822,403*	*3,381,326*	26,420	+20	2,114,000	3,963,000
2026				27,260		2,181,000	4,089,000
2027				28,100		2,248,000	4,215,000
2028				28,940		2,315,000	4,341,000
2029				29,780		2,382,000	4,467,000
2030				30,620		2,450,000	4,593,000
2031				31,460		2,517,000	4,719,000
2032				32,300		2,584,000	4,845,000
2033				33,140		2,651,000	4,971,000
2034				33,980		2,718,000	5,097,000
2035				34,820		2,786,000	5,223,000
2036				35,660		2,853,000	5,349,000
2037				**36,500**		**2,920,000**	**5,475,000**

Table 3.4.2 Design Water Treatment Plant Capacity Without ASR Well

Year Estimate	Population Estimate	Projected Water Demand		Design WTP Capacity (without ASR Well 7)	
		Annual Average (MGD)	Max Day (MGD)	**(MGD)**	(GPM)(1)
Phase 1 Design 2027	28,100	2.25	4.22	**4.22**	3,520
Phase 2 Design 2037	36,500	2.92	5.48	**5.48**	4,570

(1) GPM is based on operation in 20 hours. The production rate of flow (GPM) for processes with higher production losses such as RO will have a higher value.

3.4.1 Projected Water Treatment Plant Requirements with ASR Well 7

Aquifer Storage and Recovery (ASR) Well 7 was constructed in 2009 to supplement peak day water demands rather than increasing the capacity of the existing water treatment facility. Excess treated water is pumped into the underground reservoir during low-demand periods and provides a treated water reservoir that can be utilized during peak demand days that exceed the capacity of the treatment plant. A detailed discussion of ASR Well is included in Section 4.5.

A summary of the water treatment plant requirements in conjunction with ASR Well 7 is included in Table 3.4.1.1 below. For a typical water system without an ASR well, the water treatment facilities are sized to treat the maximum day water demand. In North Liberty, the construction of ASR Well 7 allows the treatment plant capacity to be reduced.

Table 3.4.1.1 shows that the current ASR well reduces the required treatment plant capacity by more than 25-35% for the projected 10-year and 20-year planning periods compared the treatment capacity designs presented in Table 3.4.2. For example, in the year 2037 the required treatment plant capacity without an ASR well is 5.48 MGD, while the required treatment plant capacity in conjunction with ASR Well 7 is only 4.20 MGD, representing a 23% reduction in required treatment plant size.

The required flowrates (GPM) presented in Table 3.4.1.1 to meet the required plant capacities are based on producing the required water demand in 20 hours operation. This allows for 4 hours per day of filter backwashing and softener regeneration which provides enough additional flow for production losses due to water that is sent to waste during softener regeneration or filter backwash. Production losses for ion-exchange softening typically range from 5-10% of the daily water demand. Other treatment processes considered for future upgrades in this report may have higher production losses and require a higher treated flowrates, although the target daily treated water production of **3.00 MGD for Phase 1 (population 28,100)** improvements and **4.20 MGD for Phase**

2 (population 36,500) improvements that incorporate the ASR well operation will remain the same.

Table 3.4.1.1 Projected Water Treatment Plant Capacity Required w/ ASR Well No. 7

Year Estimate	Population Estimate	Projected Water Demand: Annual Average (MGD)	Projected Water Demand: Max Day (MGD)	Required WTP Capacity (w/ ASR Well 7)(1): **(MGD)**	Required WTP Capacity (w/ ASR Well 7)(1): (GPM)2)
Phase 1 Design 2027	28,100	2.25	4.22	**3.00**	2,500
Phase 2 Design 2037	36,500	2.92	5.48	**4.20**	3,500

(1) Required WTP capacity discussed in Section 4.5.
(2) GPM is based on operation in 20 hours. The production rate of flow (GPM) for processes with higher production losses such as RO will have a higher value.

3.5 Treatment Standards

The north Liberty Water Supply is subject to the Federal Safe Drinking Water Act (SDWA) and regulations issued by the Iowa Department of Natural Resources (IDNR) and the US Environmental Protection Agency (EPA). The City currently meets these required quality standards and goes beyond the required standards to provide their customers with a softened water supply. All options considered for expansion and or replacement of the existing supply and treatment facilities will be developed based on the premise that the water quality will continue to be as good as or better than current quality and of course will continue to meet required primary drinking water standards.

A main constituent of concern with the existing treatment facilities is meeting the new arsenic limits of 10 parts per billion (ppb) when ASR Well 7 is in operation. Arsenic is naturally present in the Jordan aquifer above the drinking water standards; therefore, any mixing or withdrawal of the natural aquifer during ASR recovery periods could have arsenic levels above allowable levels. The main strategy to prevent this is to try to develop a protective "bubble" of treated water surrounding the new ASR well injections. Section 4.5 includes a more detailed discussion of the ASR well.

4

DESCRIPTION AND EVALUATION OF THE EXISTING SYSTEM

4.1 Raw Water Supply Wells

The City currently draws its water supply from six wells. Four of these wells (numbered 1 through 4) are completed in the Silurian formation. Two wells (Wells 5 and 6) are completed in the Jordan aquifer. The city also has one aquifer storage and recovery (ASR) well (Well 7) that is discussed in Section 4.5. While not truly a raw water supply well, during an emergency event the ASR would simply pump treated water into the distribution system rather than raw water to the plant, and thus can be considered when analyzing firm capacity of the wells. Table 4.1 summarizes significant features of the raw water supply wells. The capacities of Jordan Wells 5 & 6 have increased from the 2006 report due to well acidizing and pump replacement which was a recommended improvement. Figure 4.1 shows a map with the location of the various wells and the water treatment plant in the City

Table 4.1. Existing Raw Water Supply Well Data.

Well No.	Date Installed.	Depth	Casing Dia.	Formation	Capacity When Drilled	Current Capacity
1	1970	422’	8”	Silurian	50 gpm	100 gpm(1)
2	1977	460’	NA	Silurian	167 gpm	165 gpm
3	1984	502’	8”	Silurian	130 gpm	165 gpm
4	1988	500’	8”	Silurian	165 gpm	165 gpm
5	1994	1717’	12”	Jordan	600 gpm	1,100 gpm
6	2001	1820’	18” OD	Jordan	700 gpm	1,500 gpm
7	2009	1156’	24”	Jordan	1,100 gpm	1,100 gpm

(1) Well 1 can only pump 50 gpm when operating with Jordan Well No. 5 in operation.
(2) It is assumed that Well 2 can only pump 80 gpm when operating with Jordan wells in operation.
(3) Well 7 is used an ASR well.

FOX engineering

WATER FACILITY MAP

WATER SYSTEM FACILITY PLAN
NORTH LIBERTY, IOWA

FIGURE:	4.1	
REVISION	NO.	DATE
DRAWN JBA	PROJECT NO. 3373-12A	DATE 12-21-12

Additional data regarding these various wells accumulated during this study are included the report Appendix A.

The Iowa Department of Natural Resources (IDNR) has adopted the Great Lakes Upper Mississippi River Board of State Public Health & Environmental Managers Recommended Standards for Water Works (commonly referred to as "10-States Standards"). The most current 2007 Edition of the 10-States Standards requires that the total developed source (total of all operating wells) be capable of providing the peak day demand with the largest producing well out of service (firm capacity). This rule has been revised from older versions which only required that the source provide a firm capacity equal to the average day demand. Based on the capacities listed in Table 4.1, the raw water supply wells have a firm capacity of **1560 gpm** with the largest well out of service (Well 6). This assumes Well 5 is pumping 1,100 gpm, Wells 1 and 2 are pumping 50 and 80 gpm and Wells 3 and 4 are pumping 165 gpm. This capacity is adequate to provide approximately **1.87 MGD** in a 20-hour period. With an ASR well in operation, the maximum day demand required from the raw water wells is actually reduced by the reliable capacity of the ASR Well, since the treatment plant size is typically reduced a corresponding amount. Another way to look at this is to simply consider that ASR well as a raw water supply well as discussed below.

If the ASR well is considered as a water supply well, the firm capacity is much greater. With Well 6 out of service, and including the ASR well, the firm water supply capacity is **2660 gpm**, or **3.19 MGD** in a 20-hr period.

With the ASR well considered in the firm capacity, the raw water capacity is anticipated to provide adequate water supply for the city for a population of approximately 21,000 (projected year 2019) with the existing treatment facility.

If the Silurian wells are separated from the Jordan wells, the firm capacity of the raw water supply wells may be increased (due to the increased pumping of the Silurian wells) to approximately 2,795 gpm with the largest well out of service (Well 6) and including the ASR. This capacity is adequate to provide a finished water supply of approximately 3.35 MGD and may extend the water supply capacity another year or so.

4.2 Raw Water Transmission Lines

Raw water currently arrives at the water treatment plant through two raw water transmission lines from the remote Wells 2, 3, 4 and 6. Wells 3, 4 and 6 feed into the same 8" line, which was upgraded in the 2001 project when Well 6 was constructed. Well 2 is connected to the water plant through a 4-inch raw water line. Well 1 and 5 are immediately adjacent to the water treatment plant. As previously discussed, the Silurian wells capacity is reduced when pumping in combination with the Jordan wells. It was discussed with the city that raw water piping modifications would be proposed where practical to allow the Silurian wells to pump closer to their maximum capacity when operating with the much larger Jordan wells. The Silurian wells flow is decreased due to increased operating pressure with the higher flowrates from the Jordan wells. The increased pressure falls outside of the well pumps design pressure and causes of the pump operation to slide backwards on its operating curve thereby lowering the pump output. When Well 1 is operating with Well 5 (Jordan) it can only pump approximately 50 gpm versus its rated 100 gpm capacity. It is assumed that the other Silurian wells have a similar issue, although this has not been specifically tested.

The only other option to increase the Silurian well flow would be to install larger pump motors and variable frequency drives to allow adjustment for conditions with and without the Jordan wells in operation. This option may require upsized electrical supplies at the individual well sites. An in-depth electrical analysis of the well sites to verify the extent of modifications to install larger pump motors was not conducted as part of this study.

4.3 Treatment Plant

The existing water treatment plant (Figure 4.3) is located on a cul-de-sac at the south end of Chestnut Street in the East Central part of the City. Constructed in 1977 and upgraded in 2001, the plant incorporates an induced draft aerator, a detention basin that also serves as a wet well for the high service pumps, three high service pumps, two 6-cell horizontal high rate pressure filters, two cation exchange water softeners, salt storage for softener regeneration, and chemical feed systems for gas chlorine and polyphosphate. The treatment building also houses a laboratory and office space for the water system personnel. Current space is very limited in both the existing water plant building and on the plant site with little or no room available for any additional expansion of the treatment capacity without acquisition of additional property. Figure 4.3.2 illustrates a site plan of the existing plant site.

Figure 4.3.1 Existing Water Treatment Plant

FIGURE: 4.3.2

REVISION	NO.	DATE

DRAWN	PROJECT NO.	DATE
JAK	3373-12A	03/25/13

EXISTING WATER TREATMENT PLANT

WATER SYSTEM FACILITY PLAN
NORTH LIBERTY, IOWA

FOX Engineering Associates, Inc.
414 South 17th Street, Suite 107
Ames, Iowa 50010
Phone: (515) 233-0000
FAX: (515) 233-0103

FOX engineering

PATH: K:\proj\3000\3373-12A – North Liberty\Drawings\ENGINEER SKETCHES\EXISTING 4.3.2.dwg

4.3.1 Aeration

The induced draft aerator is a General Filter Co. Model ID-204. It has an area of 7' x 7' and a sidewall height of 10 feet (Figure 4.3.1.1). Constructed of ¼" aluminum plate with an 8" inlet pipe and 10" outlet pipe, it was originally rated for a flow of 800 gpm. Based on typical sizing criteria of 25 gpm/sf the aerator should be able to handle at least 1,200 gpm. The aerator has been operated at flows up to 1500 gpm and there have been no reports of poor performance at these elevated flows. While it is likely at these elevated flows that the hydrogen sulfide stripping action of the aerator would not be as effective, the main purpose of the aerator is to introduce oxygen for iron oxidation.

The unit has a ½ hp, 460-volt, 3-phase blower with a capacity of 3,000 cfm at a head loss of 3/8" of water. Raw water from the various wells is delivered directly to this aerator. The aerator sets atop the detention tank and effluent from the aerator drops directly into the tank. The centerline of the eight-inch ductile iron pipe feeding the aerator is about 25 feet above ground level at the detention tank. The aerator appears to be in good operating condition, although it is over 30 years old and has internal redwood slat trays which are not recommended in modern designs. Redwood lumber is more prone to the proliferation of microbial slimes and certain strains of bacteria. It is recommended that the internals be replaced to remove the redwood slats in addition to adding or replacing the existing aerator for future increased flow options.

Figure 4.3.1.1 Existing Induced-Draft Aerator

4.3.2 Raw Water Detention Tank

The Raw Water Detention Tank is a 29,000 gallon welded steel tank that was installed with the original plant in the 1970s (Figure 4.3.2.1). The tank has a diameter of 26 feet and sidewall height of eight feet. The tank is elevated approximately 5 feet above the ground on a concrete foundation. Effluent from the detention tank feeds to the high service pumps through a ten inch ductile iron pipe.

Shive Hattery inspected the existing tank in May 2011 and recommended that the tank be taken out of service and the roof cap repaired in the next couple years due to the loss of steel material and corrosion on the interior of the tank.

At 29,000 gallon capacity, the tank provides 22 minutes of detention time at current peak plant capacity of 1,300 gpm based on the softeners operation as discussed later in this section. According to Ten States Standards (4.6.1.2) a minimum detention of 30 minutes shall be provided for iron and manganese oxidation following aeration. The minimum time may be reduced only where a pilot plant study has been conducted.


Figure 4.3.2.1 Existing Raw Water Detention Tank

At North Liberty, the current detention tank meets 75% of the Ten States Standards design requirement, but seems to provide adequate time for oxidation of iron and manganese under the current plant operating conditions. A pilot test could be conducted to determine the maximum capacity of the existing tank; however, based on an inspection by Shive-Hattery, the existing detention tank is in bad structural condition and should be replaced in the near future. The city would prefer a new detention tank with multiple compartments to allow for a single compartment to be taken out of service without stopping plant operations. The city would also like the ground storage tank to be moved from the current location. In order to accomplish this, the adjacent property to the east would need to be acquired if the city does not desire to utilize city park property.

4.3.3 High Service Pumps

The water system is supplied through the use of three horizontal split case pumps located in the water plant building (Figure 4.3.3). Two of the pumps were installed as part of the 2001 expansion and are provided with variable frequency drives to allow pumping rates to be set to match system demand or treatment capacity. The third pump was installed in 2007. The firm capacity of the pumps with two pumps running is approximately 1550 gpm. If the softeners are bypassed completely, the pumps can deliver 1670-1680 gpm to the system. The pumps take suction from the ten-inch suction header originating at the detention tank and discharge to the supply headers feeding the pressure filters.

Figure 4.3.3 Existing High Service Pumps

4.3.4 Pressure Filters

The iron filters are General Filter Corporation Multi-Cell model HPF six cell filters (Figure 4.3.4). One filter was originally installed during the initial plant construction in 1977 and replaced in 2006, while the second was installed as part of the 2001 expansion project. The filter installed in 2001 has had a pin-hole leak since start-up. Both units are eight feet in diameter and twenty-eight feet long with a total media surface area of 210 square feet each. The sand depth within the filters is 24 inches over a seventeen-inch layer of gravel setting on a 5/16" curved steel plat under-drains with stainless steel shroud baffles. The sand has a size range of 0.5 to 0.6 mm and a uniformity coefficient of 1.6 as designed.

Ten States standards (4.2.2.2) specify that the rate of filtration for rapid rate pressure filters shall not exceed 3 gpm/sf except where in plant testing as approved by the reviewing authority has demonstrated satisfactory results at higher rates. The standards also state that the minimum criteria related to rate of filtration, structural details, etc. provided for rapid rate gravity filters shall also apply where appropriate (4.2.2.1). A standard for rapid rate gravity filters which is assumed to apply is under 4.2.13 stating: "Where more than two filter units are provided, the filters shall be capable of meeting the plant design capacity at the approved filtration rate with one filter removed from service." Based on these standards, the combined filter capacity is 1,155 gpm with one of 12 filter cells out of service at 3 gpm/sf.

Although normal design practice is to design high rate sand filters at a filtration rate of 3 gpm/sf with one filter out of service, pushing the filtration rate beyond 3 gpm/sf, is common and possible. The potential of running the existing filters at an increased rate to increase the plant capacity was explored in 2006 based on a recommendation in the 2006 facility report. The filters were tested at 6 gpm/sf over a 3-day test period. During this time the filters were operated for a total of 12 hours and after that time the increase in pressure was already 2 psi, so the test was suspended and it was determined that operating at 6 gpm/sf for extended duration would result in frequent backwashing which would not be a practical operating scheme. The goal at the time was to see if the original filter could be abandoned rather than replaced. Since the test was not successful, the city proceeded with replacing the original filter in 2006.

Figure 4.3.4 Existing Horizontal Pressure Filters

The filters were not tested at filtering rates between 3 and 6 gpm/sf. Based on operating experience, city staff feels that the filters are able to operate up to approximately 1500 gpm without any detrimental effects. Table 4.3.4.1 presents a summary of the filter capacity based on IDNR standards and operating experience.

Table 4.3.4.1 Filters Capacity Summary

	Filter Rate w/ One Cell Out of Service (GPM/SF)	Filter Capacity w/ One Cell Out of Service (GPM)	Filter Capacity in 20 hrs (MGD)
Filters Original Design Capacity ('06)	3.0	1,155	1.39
Filters Current Maximum Operation	3.9	1,500	1.80

It may be possible to operate the existing filters at a higher filtering rate to increase the filter capacity; however, the concern would be how quickly the filter bed would become clogged and require backwashing. It currently takes 6 hours to backwash all filter cells or approximately one day per week. Backwashing all filter cells more than once per week would be very time consuming and difficult to maintain. Backwashing the filter cells more than once per week would not be a recommended design for North Liberty based on the number of filter cells and time commitment for backwashing. It may be possible that alternate media or a combination of media may allow the filter capacity to be increased while maintaining a similar performance; however, the frequency of backwashing would need to be evaluated as well. An on-site pilot could be conducted with several different media columns to determine if the existing filter units can be optimized at a higher flow.

Current operating practice is to backwash filters once per week except during peak production periods when more frequent backwash may be necessary to maintain flow capacity or treatment efficiency. There is not typically more than 1 psi headloss across the filter bed prior to a backwash period which indicates a fairly clean bed and good filter runs. A backwash trigger of 3.5 psi differential headloss is recommended in the operation and maintenance manual. The low differential pressure also suggests that the filtration rate could potentially be increased without exceeding reasonable filter capacity. We know that the existing filters are not able to operate for extended durations at 6 gpm/sf or above, but they could potentially be increased to 4.5-5 gpm/sf (1650 to 1900 gpm) to gain a small amount of filter capacity.

Filter backwash is rated at 15 gpm/sf. At a filtration rate of at least 525 gpm, one cell of the filter can be backwashed by the production of the other five cells, thus not requiring the filter to be backwashed from finished water storage in the system. The units have a six-inch inlet pipe connections and an eight-inch outlet pipe connections with four inch drain lines.

4.3.4.1 Filter Backwash Detention Tank

The filters are currently backwashed approximately once per week and backwash waste is sent to the Filter Backwash Detention Tank prior to pumping to the sanitary sewer. Plant staff are able to backwash all 12 filter cells in a single day and still keep up with water demands; however, they may have to wait for the Backwash Detention Tank to dewater before starting the next filter backwash. It takes approximately 6 hours to backwash all filter cells.

The Filter Backwash Detention Tank is a below-ground concrete tank that is approximately 7,200 gallons. With the current pumping capacity, the backwash tank can handle two complete filter backwash cycles before filling up and requiring a halt to backwash operations. The tank is a circular concrete tank laid on its side (6 feet diameter x 34 feet long). The tank is currently undersized to store a reasonable amount of flow from multiple backwash cycles, since it was originally designed for flows from a single horizontal pressure filter. The tank has two submersible backwash pumps that operate simultaneously to pump backwash water to the sanitary sewer. The backwash pumps are 2 hp each and rated for 260 gpm each at 16 feet total dynamic head (TDH).

The backwash water is currently pumped into the sanitary sewer behind Hickory Street. Occasionally, the city receives complaints during backwashing due to sanitary sewer back-ups. In the future, it may be possible to reroute backwash waste to the new sanitary sewer that was constructed through the park or construct additional backwash detention tank capacity to alleviate these concerns.

The Filter Backwash Detention Tank is adequate for the existing two pressure filters, except that it takes a little more time than would otherwise be required to backwash all filters in a single day when operators have to wait for the detention tank to dewater. If the Filter Backwash Detention Tank were large enough to handle all backwash flows from both pressure filters, it is estimated that it might only take 3.5-4 hours to backwash all filters in a single day versus the 6 hours currently required. It is probably not cost effective to increase the size of the detention tank based on the current flows, since it would only save 2-2.5 hours per week and the cost would be significant; however, if additional pressure filters are added in the future or if flows through the plant are increased, an increase in Filter Backwash Detention Tank capacity would be recommended.

4.3.5 Softeners

The cation exchange water softening system was added to the plant as part of the 2001 expansion. The softeners were intended to allow the system to meet primary drinking water standards for radiological parameters with Jordan Aquifer source water by removing the small quantities of radium and uranium contained in the raw water and to provide a higher quality, i.e. softened, water to the city's water customers. The softeners operate on a sodium cycle in which water passing through the exchange beds of the softeners exchange calcium and magnesium ions for sodium ions. The bed is then regenerated by feeding a high concentration of sodium chloride (salt) solution through the bed to flush off the hardness ions and re-substitute them with sodium ions.

Figure 4.3.4 Existing Zeolite Softeners

While the process serves adequately to remove hardness and certain other cations such as the radium from the water, it also increases the sodium in the finished water and chloride content in the wastewater. The high sodium content can be a detriment to horticulture when the water is used for irrigation and to persons on low sodium diets. The high level of chlorides is potentially a concern if elevated chloride levels in the regeneration water which is sent to the sanitary sewer cause violations in chloride effluent limits at the city's wastewater treatment facility. Home softeners also contribute significantly to elevated chloride levels in wastewater. The level of hardness removal and thus the hardness of the final water supply is controlled by the rate of water that is bypassed around the softeners since nearly all hardness is removed from water passing through the softeners.

There are two softening units each having 120" diameter and 108" vertical side wall height. The tanks are designed for a minimum working pressure of 100 psig and a hydrostatic test pressure of 150 psig. Each tank contains 380 cu. ft. of high capacity polystyrene-divinylbenzene cation exchange resin. The resin has an operating exchange value of 20,000 grains of hardness as CaCO3 per cubic foot when regenerated with 6 lbs. of salt per cubic foot or resin. This represents approximately 256,000 gallons of production capacity for each filter between regenerations.

The softeners are the current treatment limitation within the existing plant. Original shop drawings show a capacity of 418 gpm each softener with a bypass flow of 305 gpm and a total plant flow of 1140 gpm. At this condition, the softening rate is 5.3 gpm/sf. The original design was based on a raw water quality of 480 mg/l as CaCO3 (28 grains per gallon) total hardness and a finished water quality of approximately 120/ mg/l as CaCO3 total hardness CaCO3 (7.5 grains per gallon).

Current Ten States Standards (4.4.2.5) have a maximum allowable softening rate of 7 gpm/sf. During a softener regeneration, the total plant flow is automatically reduced to limit the flow through the softener in operation to improve the quality of finished water. Trying to increase the flow through the softening resin above the design rate in order to increase plant capacity has two negative affects including: (1) increased headloss and (2) decreased softening performance. The cation exchange resin has a hydraulic rating up to 24 gpm/sf; however, the treatment quality at this elevated flow is very low and the headloss is nearly five times higher that during normal operation. This is not a practical design point and is over three times higher than the IDNR standards.

It is not uncommon for a softening system to provide for adequate hardness removal at softening levels near the IDNR standards level of 7 gpm/sf; however, at North Liberty softening performance drops off at softening rates above approximately 5.5 gpm/sf or 435-440 gpm through each softener. One reason for this occurrence may be due to the elevated total dissolved solids (TDS) levels in the raw water. Raw waters with elevated TDS will experience a greater loss in performance with increased flow; however it is not typically that noticeable. Operating staff has indicated that the softening performance has always been reduced above 435-440 gpm per softener, so it does not appear to be a phenomenon of needing a resin replacement. Plant testing in January 2013 showed a small increased capacity (<5%) at up to 5.8 gpm/sf before significant softening performance was reduced. The plant superintendent thinks the capacity increase is a result of the recent softening valves replacement. As the valves age the performance will likely be reduced back to previous levels. For the purposes of planning in this report, a maximum softening capacity of 5.5 gpm/sf will be assumed.

The current method of softener operation has altered from the original design. Currently, the total plant production rate is higher than originally designed, but the softeners treated flow has changed very little. Instead, the quantity of softener bypass flow has increased and the finished hardness level of the blended water has increased 50% from a design level of 128 mg/l as $CaCO_3$ to an average finished water hardness level of 193 mg/l as $CaCO_3$ for 2012. The current average raw hardness (510 mg/l) is also higher than the design raw water hardness (480 mg/l), which also reduces the performance of the softeners from the original design.

Plant staff has indicated that the softeners currently operate very well at a total plant flow of approximately 1305 gpm. At this production rate, the softeners are treating 435 gpm each with 435 gpm bypass around the softeners. Above this flow rate, the hardness begins to increase. During days when it is necessary to increase the plant production rate above 1305 gpm, such as this past July, plant operation can be adjusted by either increasing flow rate through the softeners, increasing the bypass flow or both. All scenarios result in a lower quality finished water or a higher level of total hardness.

Softening plants are typically operated in the range of finished water total hardness of 80-120 mg/l as $CaCO_3$. The current finished water quality at North Liberty varies from approximately 136 – 342 mg/l with an average of 193 mg/l as $CaCO_3$ and is considered to be a fairly hard water. The increase in finished water hardness is due to current water demands exceeding the original softening design. In July 2012, the average finished water hardness was 220 mg/l as $CaCO_3$. For future planning throughout this report, a design total hardness level of 120 mg/l as $CaCO_3$, similar to the original design will be targeted for improved water quality more typical of a conventional softening plant.

Based on the current softening levels, the existing treatment plant has a maximum capacity of 1305 gpm which equates to approximately **1.56 MGD** in 20 hours. In 2012, there were 33 days when water demand exceeded 1.56 MGD. The capacity of the existing softening system is even lower if the desired total hardness level is less than currently being provided. In order to provide a softening level down to 120 mg/l total hardness, the plant capacity based on softeners is only approximately 1.37 MGD.

Table 4.3.5.1 summarizes the original softeners design and current operating capacity with current finished water quality.

Table 4.3.5.1 Softeners Capacity Summary

	Individual Softener Flow (GPM)	Bypass Flow (GPM)	Total Plant Flow (GPM)	Softeners Capacity in 20 hrs (MGD)	Finished Water Hardness
Softeners Original Design Capacity (2001)(1)	417	305	1140	1.37	128 mg/L as $CaCO_3$
Softeners Current Maximum Capacity with Current Water Quality(2)	435	435	1305	1.56	170 mg/l as $CaCO_3$

(1) Original softener design based on 480 mg/l raw water hardness.
(2) Estimated current typical operation based on 510 mg/l raw water hardness.

One method of increasing the exchange capacity of the existing softeners, but not necessarily the allowable softening rate, would be to increase the salt regeneration rate from 6 lbs per cubic foot of resin to 10 pounds per cubic foot of resin. This would increase the treatment capacity of each softener from approximately 20,000 grains/cf to approximately 26,000 grains/cf and extend treated water per softener from 256,000 gallons to 330,000 gallons. The downside of increasing the regeneration rate is that it is not very efficient. For example, a 70% increase in salt usage during regeneration only increases the treatment capacity by approximately 30%, so there is an increase in cost due to wasted salt and the rate through the softeners can not necessarily be increased (just the time between regenerations).

4.3.5.1 Salt Storage Tank

The existing salt storage tank is a fiberglass tank with a salt storage capacity of 43 tons (10 feet diameter x 15 ft straight sidewall height) which is located indoors (Figure 3.5.1). The tank is not able to accept full truck-load deliveries (50 tons) and drivers often dump excess salt at the city street department to prevent overfilling the tank at the water plant. The salt purchased by the water department is NSF certified solar salt from Cargill. Cargill currently requires 5 days advance notice of salt deliveries and the city requires salt deliveries every 6-9 days, so there is not a lot of leeway in scheduling deliveries.

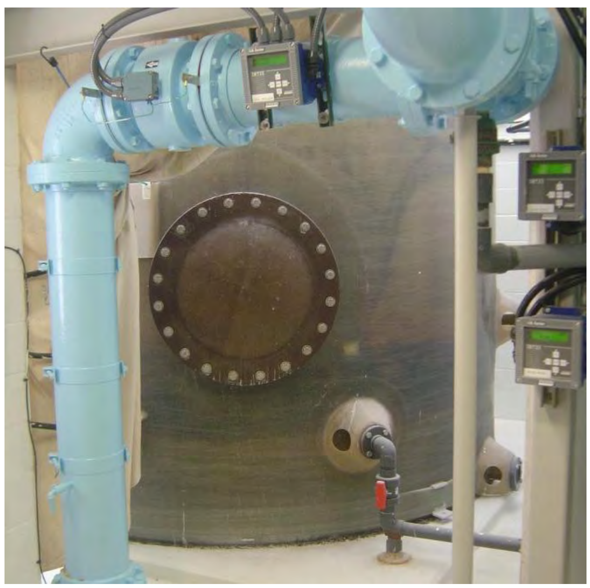

Figure 4.3.5.1 Existing Salt Storage Tank

Ten States Standards (4.4.2.14) states salt storage tanks "should" have sufficient capacity to store in excess of 1.5 truckloads of salt and provide for at least 30 days of operation. Based on 1.5 truckloads storage, the salt storage tank should be at least 75 tons. City staff has indicated that they would prefer at least 2-2.5 storage volumes (100-125 tons) in order to provide for adequate operation during holidays and weekends and to allow for the supplier's required 5-day delivery notice. The storage requirement based on 30 days of operation is also considered and is the controlling factor for North Liberty. At current softener capacity of 1300 gpm (1.56 MGD) 30 days of salt storage capacity would require about 139 tons. To provide 30-days salt storage during the maximum month at the 20-year design four (4) 100-ton salt storage tanks would be required. A 100-ton fiberglass tank is the largest prefabricated fiberglass tank available for salt storage. A 100-ton tank is 14-foot diameter x 23’-8” high. This would require a large amount of a space and a complicated piping network.

Table 4.3.5.1.1 provides a summary of salt storage tank requirements for the IDNR recommended 30-day storage period and a lower storage period of 21-days which may be allowed through an IDNR variance if a 30-day storage is thought to be impractical or unnecessary for future designs.

Table 4.3.5.1.1 Salt Storage Tank Requirements Summary

	Average Day Water Demand (MGD)	30-Day Salt Storage Requirement(1) (Tons)	21-Day Salt Storage Requirement(3) (Tons)
Existing (2012)	1.14	139(1)	97
Phase 1 – 2022 Design	1.9	250(2)	175
Phase 2 – 2032 Design	2.5	325(2)	225

(1) Based on peak plant capacity for 30 days.
(2) Based on max month water demand estimated at 1.5 times average day demand.
(3) Based on a 21-day delivery schedule during a maximum month.

The indoor location of the brine tank has created a corrosive environment due to salt residue which is very corrosive to concrete. The current indoor location requires a lot of maintenance to clean up and prevent corrosion. The existing salt storage tank should preferably be relocated to the outdoors.

4.3.6 Chemical Feed

The plant has the ability to feed chlorine for disinfection purposes, fluoride for tooth decay prevention, and phosphate compounds for improving water stability to protect water mains and plumbing systems from corrosion and depositions. Chlorine for disinfection is fed through two Advance gas chlorinators supplied from 150-pound chlorine gas cylinders. Since the plant improvements installed in 2001, the city has added automatic chlorine feed controls to the chemical feed system so that the chlorine feed rate is always proportional to finished water flow rates to the system. The city did not indicate any deficiencies with the existing chemical feed systems. For any options which consider upgrading and increasing the flow through the existing plant, the chemical feed storage tanks and pumping systems may need to be upsized as well.

4.4 Present Water Quality and Treatment Requirements

The treated water supply from the city's treatment plant can be characterized as a good quality water. The water contains no contamination that exceeds the maximum contaminant limits (MCLs) established by the US EPA and the Iowa Department of Natural Resources. The treated water has a relatively high hardness with an average hardness of around 190 mg/L as $CaCO_3$ in 2012. Table 4.4 summarizes typical water quality characteristics of the North Liberty finished water supply.

Table 4.4 Water Quality of North Liberty Water Supply.

Parameter	Units	Conc.	Parameter	Units	Conc.
pH	---	7.8	Total Hardness	mg/L	190
Total Dissolved Solids	mg/L	955	Free Chlorine	mg/L	0.5
Sulfate	mg/L	460	Sodium	mg/L	290
Chloride	mg/L	27	Iron	mg/L	0.01
Fluoride	mg/L	1	PO_4	mg/L	1.5
Nitrate (as N)	mg/L	1.4			

4.4.1 Biological Contaminants

In reviewing existing data provided by the utility, no information was found that suggested problems with biological contaminants in the finished water. The City provides chlorine disinfection of the finished water.

4.4.2 Organic Contaminants

A review of existing water quality data from North Liberty shows no organic contamination of the treated water. Test results gave no indication of contamination problems with volatile organic compounds (VOC), synthetic organic compounds (SOC), or pesticides.

4.4.3 Radiological Contaminants

As stated above, although the raw water from the Jordan Aquifer does contain radiological contaminants in excess of primary drinking water standards, the current softening system readily reduces the levels of these materials to safe levels.

4.4.4 Inorganic Contaminants

Comparison of MCLs for inorganic compounds and utility records indicates that there are no problems in complying with most mandated standards. Levels of metals such as arsenic, barium, cadmium, chromium, mercury, and selenium are well below

MCLs. The treated water is softened so that calcium and magnesium levels are significantly lower than raw water levels, but no drinking water standards require softened water.

4.5 Aquifer Storage & Recovery (ASR) Well

The City currently has one aquifer storage and recovery (ASR) well constructed in the Jordan aquifer. The ASR well is named Well 7. It was constructed in July 2009 and has been undergoing testing cycles for the past three years. In August-September 2012, ASR recovery water was first pumped to the system.

The ASR well is intended to provide a supplemental supply of water to meet peak day demands. The decision to build an ASR well was selected in 2006 as an alternate to expanding the treatment plant capacity because it offered significant cost savings. The concept of ASR wells is that treated water is stored in underground aquifers during periods of excess water production and then the treated water is pumped back out of the wells during periods of excess water demand. For typical systems where peak demands are often twice average demand rates, this may mean a water treatment plant sizing can be cut nearly in half as long as a back-up plan or additional redundancy is provided in the case of an ASR well failure. A single ASR wells does not provide system reliability, even with stand-by power. ASR wells are equipped with a single mechanical pump that is subject to failure. If the operation of an ASR well is critical to meeting peak day demands, then a source of redundancy in case of failure should be provided such as another ASR well or connection to a neighboring city's water supply. When the ASR well was originally proposed and constructed, the plan for providing a reliable backup was to connect to the City of Coralville. North Liberty currently has a 12-inch connection to the City of Coralville's water distribution system although an inter-city agreement is not in place. The city should explore the potential for obtaining an agreement with a neighboring city to provide a back-up supply in case of emergency.

ASR Well 7 was designed for an injection rate of 500 - 900 gpm and a withdrawal rate up to 1,200 gpm. ASR Well testing to date shows that the ASR well is able to operate at an injection rate of 800 gpm and a recovery rate of 1,100 gpm when pumping to the system. If we add a small amount of conservatism and assume a 20-hour available recovery period to allow downtime for maintenance or emergency repairs; this results in an available recovery volume of **1.32 MGD** from ASR Well 7. This capacity can be used to off-set peak day demands, provided an adequate volume of treated water can be injected during low demand periods.

Table 4.5.1 summarizes data from the ASR test cycles over the past three years. During the first test cycle in March 2010, 11.5 million gallons (Mgal) was pumped into the ASR and 125% was recovered and tested. Arsenic levels exceeded the drinking water limit (0.01 mg/l) after 50% recovery; however total dissolved solids (TDS) only increased slightly from approximately 990 mg/l to 1,000 mg/l after 125% recovery testing. During the second test cycle in late 2010, a similar volume of treated water was pumped in the ASR well and the arsenic levels stayed below drinking water standards until 90%

recovery was reached. During Test Cycle 3 in 2011, 34.6 Mgal of treated water was injected into the ASR for 36 days at 800 gpm and recovered over a period of 28 days at 1200 gpm. Arsenic levels reached the EPA standard after 12 days or 50% recovery (17.3 MGal). During Test Cycle 4 in 2012, the city injected 39.3 Mgal into the ASR well over 69 days in February through April and recovered 30% of the injected water (11 MGal) in August through September 2012 over approximately 15 days. The recovery period was halted by the city in order to maintain operation within the current IDNR operating permit which states that the recovery period is through September 30. It may be possible to extend this recovery period either by notifying the IDNR or obtaining a modified permit at some point in the future. The city of Ankeny has a similar ASR operating permit and is able to utilize ASR recovery water in the month of October if they notify the IDNR and take the additional required 4th quarter samples.

The recovery hardness levels for the fourth test cycle were lower than the first three test cycles, but were still well above the average injected hardness level of 195 mg/l as $CaCO_3$. This is evidence of intermixing of injected water with the natural aquifer, which was anticipated in the initial years of the ASR operation. For Recovery Cycle 4, hardness levels steadily increased with increased recovery. Total hardness levels throughout the ASR recovery averaged 303 mg/l as $CaCO_3$ which is a 55% increase in hardness levels from injection to recovery.

Table 4.5.1 Well 7 (ASR) Testing Data.

Test Cycle	ASR Injection Period	ASR Recovery Period	Injection / Recovery Volume (MGal)	Injection / Recovery Rate (gpm)	Notes
1	3/1/10 – 3/15/10	3/15/10 – 3/24/10	11.5 / 14.3	800± / 1200±	Arsenic >0.01 mg/l at 50% recovery/ TH: 290-513 mg/l as $CaCO_3$
2	11/11/10 – 11/25/10	12/6 – 12/14/11	11.5 / 10.4	800± / 1200±	Arsenic = 0.01 mg/l at 90% recovery: TH: 342-445 mg/l as $CaCO_3$
3	1/17/11 – 2/22/11	3/24/11 – 5/9/11	34.6 / 43.2	800± / 1200±	Arsenic >0.01 mg/l at 50% recovery: TH: 308-478 mg/l as $CaCO_3$
4(1)	2/1/12 – 4/9/12	8/28/12 – 9/29/12	39.3 / 11.5	800± / 1100±	Hardness levels: 274 – 342 mg/l as $CaCO_3$

(1) ASR recovery pumped to system.

Testing to date has shown promising results with respect to injection and recovery rates as planned and recovery water with little increase in total dissolved solids (TDS) after 100% recovery. The main concern with water recovery from the ASR wells is exceeding arsenic standards and the elevated hardness levels. The latest Arsenic Rule went into effect in 2006, thereby lowering the previous limit from 50 ppb to 10 ppb. Testing indicates that background arsenic levels in ASR Well 7 are around 16 to 18 ppb. During two testing events, arsenic levels increased above the drinking water standards after only 50% recovery indicating mixing with the natural aquifer. The best strategy to try to prevent withdrawal of recovered water with elevated arsenic and hardness is to develop a protective "bubble" of treated water surrounding the new ASR injections. The City will need to closely monitor arsenic levels during recovery periods and halt recovery operations if the arsenic levels begin to rise.

In evaluating the effective capacity of the ASR well as a component of the overall treatment system, there are two things to consider: (1) peak recovery flow and (2) available recovery volume. ASR Well 7 has a peak recovery flow of 1.32 MGD (1,100 gpm in 20 hrs) which is added to the peak production rate of the water treatment plant to determine the combined system peak production capacity. In addition to peak flow capacity, the ASR well must have sufficient storage volume in order to supply flow during an extended period of peak demands. The storage volume is provided from injection of the "excess" treatment plant capacity during the injection season. If the water plant does not have sufficient excess capacity to provide adequate injection waters to the ASR well, then the ASR well will not have an adequate storage supply and cannot be utilized to its full potential throughout the recovery season regardless of its peak flow capacity.

Table 4.5.2 presents the required production from the ASR based on various treatment plant capacities based on the requirement for meeting the maximum day demand needs. The gray shades illustrate conditions where the maximum capacity of ASR Well 7 (1.32 MGD) is not sufficient to meet peak day demands and increased treatment capacity is required. As illustrated in Table 4.5.2, with the existing plant production capacity of 1300 gpm (1.56 MGD) based on softening limitations, the ASR well has enough peak flow capacity for a population of about 18,000 (estimated 3 years based on population projections). If the softening capacity could be increased to match the assumed filtration capacity of 1500 gpm (1.80 MGD), then the ASR well would have enough peak flow capacity for a population of about 20,500 (estimated 5 years based on population projections). If the existing plant capacity is upgraded for a higher flow rate of approximately 2,085 gpm (**2.50 MGD**), then ASR Well 7 would have enough peak flow capacity for a population of about 25,000 (estimated 10-11 years based on population projections). To meet water demands for a population of 32,300 (projected 20 year population), a water plant capacity of **3.50** MGD in conjunction with the ASR well would be required. However, none of these scenarios consider the ability of the water treatment plant to adequately fill the ASR well during the injection season. The peak capacity of the ASR well cannot be realized if the necessary volume of treated water cannot be provided for storage.

Table 4.5.2 ASR Well No. 7 Peak Production Requirements with Different Treatment Plant Capacities

Year	Population	Peak Day Demand @ 150 gpcd (MGD)	ASR Peak Production Required (MGD): Existing WTP Capacity of 1.56 MGD (1,300 gpm)(1)	WTP Capacity of 1.80 MGD (1,500 gpm)(2)	Phase 1 WTP Capacity of 3.00 MGD (2,500 gpm)(3)	Phase 2 WTP Capacity of 4.20 MGD (3,500 gpm)(3)
2012	15,500	1.80	0.24			
2013	16,340	2.45	0.89	0.65		
2014	17,180	2.58	1.02	0.78		
2015	18,020	2.70	1.14	0.90		
2016	18,860	2.83	1.27	1.03		
2017	19,700	2.96	1.40	1.16		
2018	20,540	3.08	1.52	1.28	0.08	
2019	21,380	3.21	1.65	1.41	0.21	
2020	22,220	3.33	1.77	1.53	0.33	
2021	23,060	3.46	1.90	1.66	0.46	
2022	23,900	3.59	2.03	1.79	0.59	
2023	24,740	3.71	2.15	1.91	0.71	
2024	25,580	3.84	2.28	2.04	0.84	
2025	26,420	3.96	2.40	2.16	0.96	
2026	27,260	4.09	2.53	2.29	1.09	
Phase 1 2027	**28,100**	**4.22**	**2.66**	**2.42**	**1.22**	**0.02**
2028	28,940	4.34	2.78	2.54	1.34	0.14
2029	29,780	4.47	2.91	2.67	1.47	0.27
2030	30,620	4.59	3.03	2.79	1.59	0.39
2031	31,460	4.72	3.16	2.92	1.72	0.52
2032	32,300	4.85	3.29	3.05	1.85	0.65
2033	33,140	4.97	3.41	3.17	1.97	0.77
2034	33,980	5.10	3.54	3.30	2.10	0.90
2035	34,820	5.22	3.66	3.42	2.22	1.02
2036	35,660	5.35	3.79	3.55	2.35	1.15
Phase 2 2037	**36,500**	**5.48**	**3.92**	**3.68**	**2.48**	**1.28**

(1) Assumes 20 hours WTP operation. 1300 gpm is existing softener capacity.
(2) Assumes 20 hours WTP operation. 1500 gpm is existing filters capacity.
(3) Assumes 20 hours WTP operation.
(4) Gray shaded areas indicate ASR requirements in excess of current ASR Well 7 capacity. ASR Well 7 capacity is 1.32 MGD based on 1,100 GPM in 20 hours production.

The second criterion for evaluation of the ASR well is to look at the available storage volume based on the ability of the water treatment plant to adequately fill the ASR well during the injection season. This is based on the available treatment plant capacity, allowable injection rate and assumed time frame for filling the ASR well. In evaluating the ability of the treatment plant to the fill the ASR well, it was assumed that the injection period would be limited to 6 months from October through March at some point in the future and during that period, no recovery would occur. The city's current IDNR operating permit is based on a 6-month recovery period from April 1st through September 30th.

It was also conservatively assumed that water injection could only occur up to 75% of the time the plant was in operation, and only 75% of the injected water could be recovered. Similarly, the recovery period was assumed to span over a 6 month period from April through September and during that time, no injection would occur. This method of operation is based on an ideal operating scheme for the ASR well where plant operating staff manually adjust the valves as required twice a year and performs any required start-up/shut down operations at that time, but that the ASR well is not required to switch back and forth through injection and recovery modes on a daily basis year-round. Table 4.5.3 summarizes the ASR evaluation criteria which were used to develop a model to predict optimum treatment plant capacity to fill the ASR well.

Table 4.5.3 ASR Well 7 Criteria for Usable Extended-Period Capacity

Parameter	
Injection Period(1)	Oct. 1 – Mar. 31 (6 mos.)
Daily Injection Time	75% of plant operation
Recovery Period(2)	April 1- Sept. 30 (6 mos.)
% ASR Recovery	75%

(1) Potential future operation.
(2) Based on IDNR ASR operating permit.

The maximum population that can be served based on the quantity of excess water available for injection into the ASR at various treatment plant capacities are summarized in Table 4.5.4. Results show that the ability to fill the ASR controls the maximum population served for the 1.56 MGD plant capacity. Beyond that, the requirement based on ability to fill the ASR well and the peak production are the same or the peak production alone from the ASR well is the controlling factor. With a plant capacity of 1.56 MGD, the maximum population served is approximately 18,000 (estimated 3 years based on population projections). With a plant capacity of 1.80 MGD, the maximum population served is approximately 20,500 (estimated 5 years based on population projections). This is similar to the projections from the 2006 report. It should be noted that, while the plant is capable of producing 1.80 MGD, it is at a much lower water quality than desired.

Table 4.5.4 Maximum Population Served Based on Ability to Fill ASR Well No. 7

Plant Capacity (MGD)	Maximum Allowable Average Day Demand (MGD)	Maximum Population Served (3)
1.56	1.44(1)	18,000
1.80	1.66(1)	20,500
2.50	1.91(2)	23,900
3.50	2.58(2)	32,300

(1) Based on the ability of the water plant to provide adequate treated water volume to fill the ASR with a 4.5 month injection period (11/16 – 3/31) and a 7.5 month recovery period (4/1 – 11/15)
(2) Based on the ability of the water plant to provide adequate treated water volume to fill the ASR with a 6 month injection and recovery period.
(3) Based on the maximum allowable average day demand and 80 gpcpd.
(4) Gray shaded areas indicate the maximum population served is based on the peak production capacity of the ASR rather than the ability to fill the ASR.

Table 4.5.4 shows that the existing treatment plant capacity (1.56 MGD) provides enough excess capacity to adequately fill the ASR well for a population of about 18,000. This is based on providing a similar level of finished water quality to that currently provided **and potentially extending theASR well recovery operation from 6 to 7.5 months through mid-November to cover some peak days**, instead of at the end of September in accordance with the IDNR operating permit. The IDNR should not have any issues with allowing the City to extend the recovery period; however, an additional quarterly sampling would be required. It should be pointed out that the water quality produced in 2012 with respect to total hardness (193 mg/l as $CaCO_3$) is actually 50% higher than the original design value and not within the recommended range for a softening treatment plant. At this point, customers are used to the current hardness levels and continuing to operate at those levels in order to maintain the existing plant in operation is very cost effective. Softening is not a requirement of a municipal drinking water system and there are cities that do not provide softened water for their customers. The main impetus for softening at North Liberty originally was to reduce radiological parameters to below drinking water standards and reduce hardness levels which are extremely high in the Jordan aquifer.

Figures 4.5.1 through 4.4. illustrate the current and projected operating scenarios with the ASR well. The 2012 water demands in Figure 4.5.1 are based on monthly operating reports.

Figure 4.5.1 shows that there were several months in 2012 when the treatment plant was operated above its theoretical treatment capacity. This can be done on a temporary basis as long as the hydraulic capacity is available; however, the water quality will be lowered. Plant operations will also have to be altered such as delaying required softener regenerations due to insufficient flow capacity. The design capacity of the plant (1.56 MGD) allows for down time for softener regenerations and accounts for water lost in the production of water. The plant can be operated outside of its "ideal" operating range as a temporary stop-gap measure, but it is not a recommended long-term operating scheme.

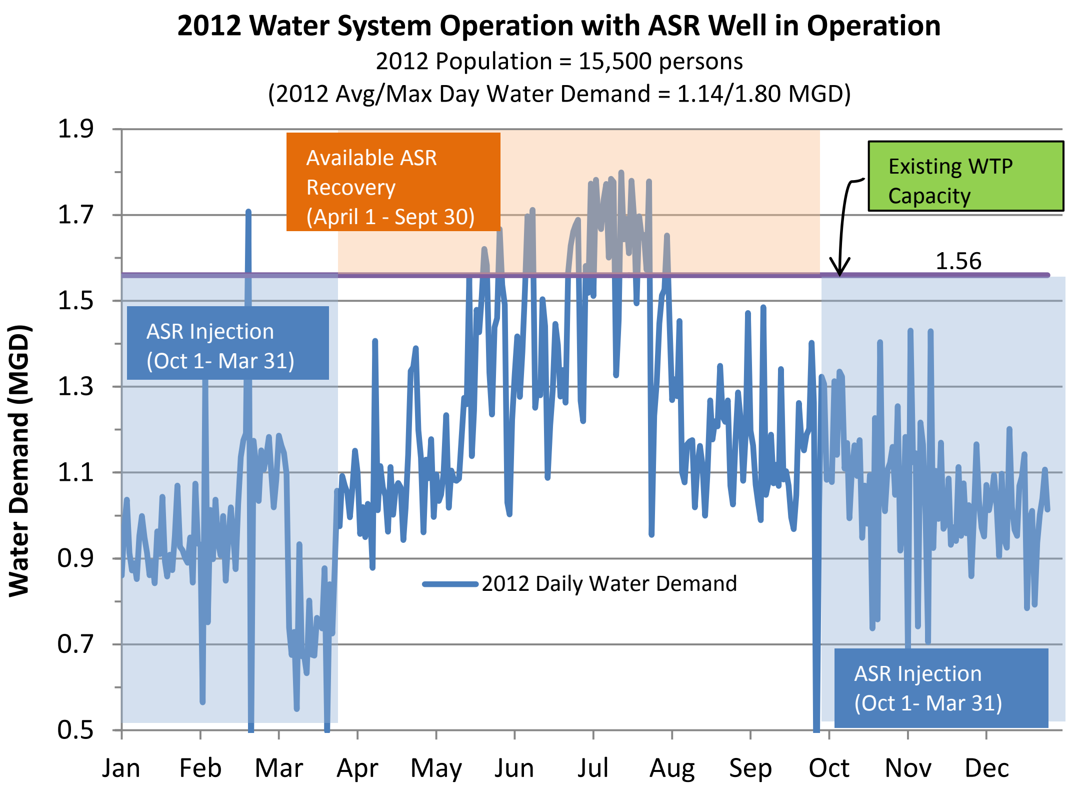

Figure 4.5.1. 2012 Water System Operation with ASR Well

The existing plant hydraulic capacity of about 1500 gpm (1.8 MGD) with two pumps running is the maximum allowable flow through the plant at the present time without completely bypassing the softening units and is also assumed to be the maximum desirable filtration rate. Figure 4.5.2 shows that at this flow rate, the existing WTP has enough capacity to fill the ASR well for approximately the next 5 years through 2018 with some alterations in plant operation from ideal conditions. Operation of the existing plant through 2018 would require extension of the ASR well recovery period into October and November which results in a reduction of the ASR well injection period from 6 to 4.5 months.

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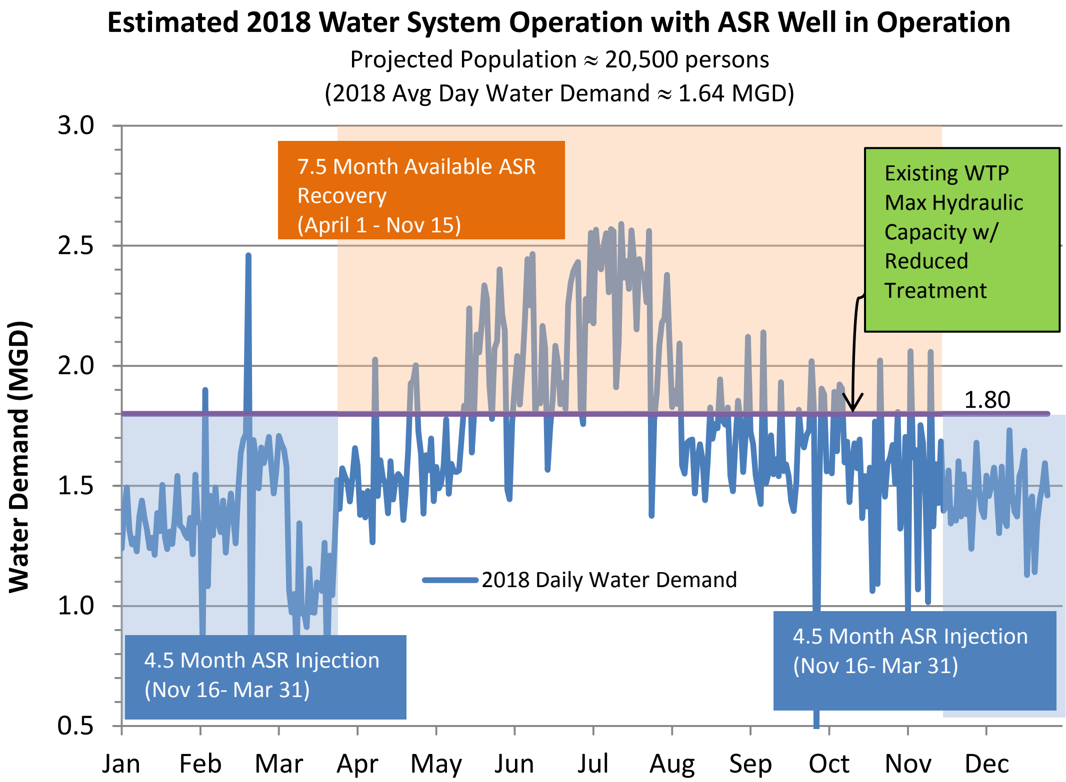

Figure 4.5.2 Estimated 2018 Water System Operation with ASR Well

The projected water demands in Figure 4.5.3 and 4.5.4 are based on a population of 28,100 and 36,500 persons based on peaking factors from 2012 daily records. Figure 4.5.3 shows that at a plant design flow of **3.0 MGD** in conjunction with ASR Well 7 allows sufficient capacity to operate meet peak day demands during the peak season. Figure 4.5.4 shows that at a plant design flow of **4.2MGD** in conjunction with ASR Well 7 allows sufficient capacity to operate meet peak day demands during the peak season.

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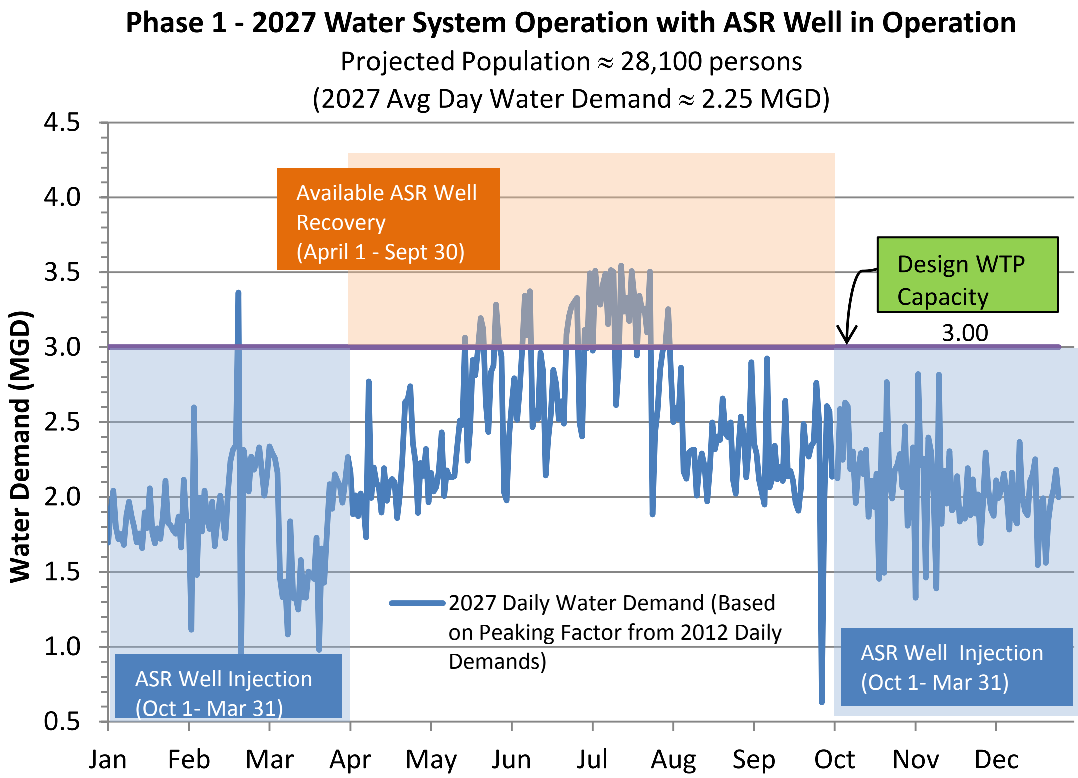

Figure 4.5.3 2027 Projected Water System Operation with ASR Well

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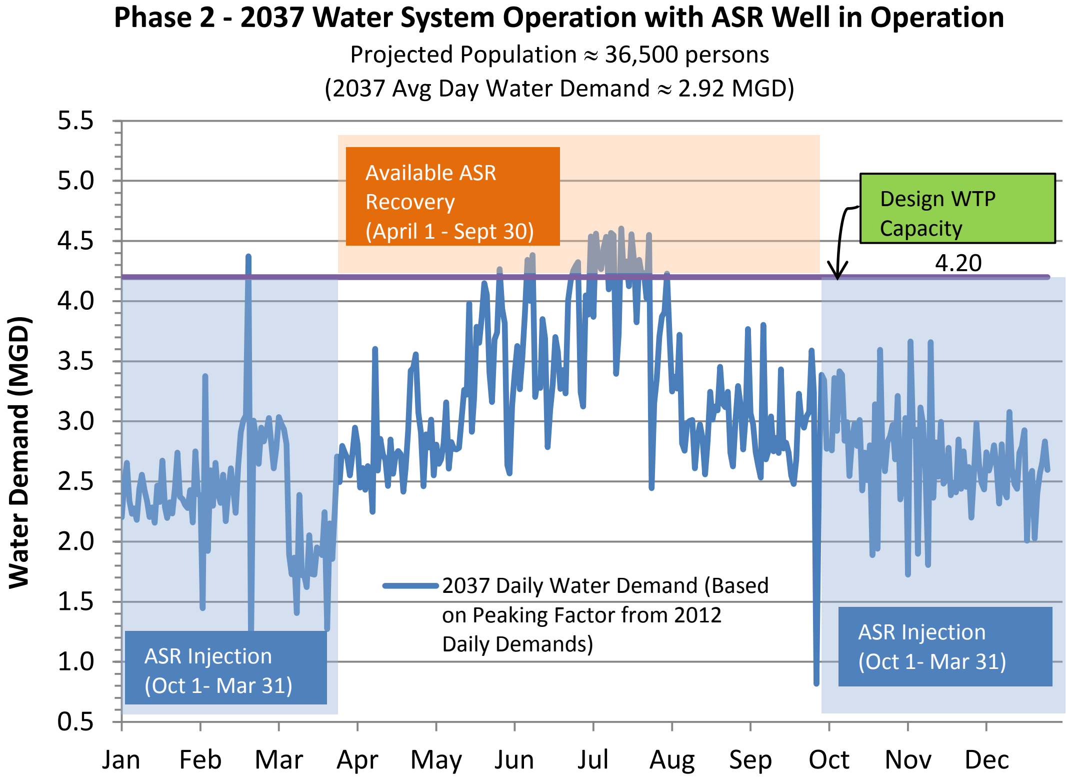

Figure 4.5.4 2037 Projected Water System Operation with ASR Well

The results of the ASR well evaluation with respect to peak flow capacity and total available storage volume provided by the treatment plant excess capacity are comparable to the 2006 planning report. In the 2006 report, it was estimated that if the plant was operated at 1,500 gpm, that a 1,100 gpm ASR well would provide sufficient peak day capacity for a population of about 22,000.

With the current treatment plant capacity of only 1300 gpm, based on the limitation of the softeners, the existing treatment plant in combination with the ASR can serve a population of about 18,000 (estimated ≈3 years). If the plant capacity is not increased in the near future, the plant will be required to operate at elevated flows above its actual treatment capacity and the quality of the finished water will decrease. However, ignoring finished water quality, the plant can hydraulically pass approximately 1,500 gpm through the plant without any upgrades, which it has done on occasion to meet peak day demands. If we consider only the hydraulic capacity of the existing plant, the existing facility in conjunction with the ASR can serve a population of about 20,500 (estimated ≈5 years) with an extension of the ASR well recovery period from the end of September to mid-November

4.6 Condition and Performance of Existing Facilities

4.6.1 Deficiencies In Existing System

Deficiencies in the existing system for the most part occur in the area of capacity. With the high rate of growth being experienced by the City of North Liberty, there appears a need to proceed in the near future to provide for additional water supply capacity for the community to meet average and peak day conditions. Other areas of deficiency are related to the physical condition of the raw water detention tank.

4.6.2 Wells

The existing wells have a firm capacity (largest well out of service) of **1560 gpm** or **1.87 MGD** (in 20 hours production). If the ASR well is considered as a water supply well, the firm capacity is **2660 gpm**, or **3.19 MGD** in a 20-hr period. This would be adequate for a population of approximately 21,000 (projected year 2019) with the current treatment technology and associated process losses in the production of water.

Beyond that time, an additional two (2) to (3) Jordan raw water wells should be constructed depending on the future upgrades by the city and treatment process selected.

If the Silurian wells are separated from the Jordan wells, the firm capacity of the raw water supply wells may be increased to approximately 2,795 gpm with the largest well out of service (Well 6) and including the ASR. This capacity is adequate to provide a finished water supply of approximately 3.35 MGD and may extend the water supply capacity another year or so.

When considering the reliability of a water supply emergency or secondary power sources should be considered. Ten-States Standards require that a secondary power source or emergency standby generator be provided so that water may be treated and/or pumped to the distribution system during power outages to meet the average day demand. The existing water plant has a standby generator that also serves Wells 1, 2, & 5. The ASR well also has a standby generator. This provides a capacity of 2330 gpm (2.80 MGD in 20 hours), which should be adequate for the foreseeable future. However, emergency standby power should be considered for new treatment facilities and/or water supply wells.

4.6.3 Raw Water Transmission Lines

The current raw water piping layout currently prevents the Silurian wells from operating at their maximum capacity when operating with a much larger Jordan well. Where practical, the Silurian wells should be disconnected from the Jordan well raw water mains and run in a separate main as close to the raw water detention tank as possible.

Another option to increase the Silurian well flow would be to install larger pump motors and variable frequency drives to allow adjustment for conditions with and without the Jordan wells in operation. This option would need to be explored further from an electrical standpoint and is not a proposed option in this report although it is a potential solution.

Construction of new raw water source facilities such as a new well will no doubt require construction of raw water transmission lines to transport the water to a treatment facility. If a new treatment facility is constructed to meet increased water demands, then there may also be a need to connect existing well supply to the new treatment plant site. In any case, raw water transmission lines will be a significant factor in comparing costs of various expansion alternatives. Also as discussed in this evaluation, separation of the Silurian wells from the Jordan well transmission mains may increase the well capacity by approximately 300 gpm.

4.6.4 Treatment Plant

The existing treatment plant building and site is very limited in space and any significant additions to treatment capacity will likely require the acquisition of adjacent property or construction of a second plant or replacement of the existing plant with a plant of larger capacity. Existing plant deficiencies will need to be addressed if the existing plant is to remain in operation for the remaining expected physical life of the facility.

4.6.4.1 Aeration

The existing aerator should handle flows up to 1200-1300 gpm with adequate performance in oxidation for iron and manganese. Above these flows, the aerator will result in lower efficiency. The unit has been operated up to 1,500 gpm without any known negative effects; however, flows above this could possibly exceed the hydraulic capacity of the piping and the aeration unit. The existing aerator also has redwood slats which should be replaced, since they can be prone to microbial proliferation. Based on a maximum assumed flow of 1,500 gpm, the aerator has enough capacity to meet the city's needs in combination with the existing ASR well for approximately the next 3 years. Beyond that period, the existing aerator should be replaced with a larger unit or a second unit should be installed.

4.6.4.2 Raw Water Detention Tank

The Raw Water Detention Tank is in poor condition structurally and should be replaced in the near future. The detention tank is also undersized based on Ten States Standards and would likely not have the ability to handle significant increased flows in the future without switching to chemicals to speed up the oxidation reactions of iron and manganese prior to filtration.

4.6.4.3 High Service Pumps

There are three high service pumps with a firm capacity of 1,550 gpm with two pumps in operation. Based on this flow, and assuming 20-hours plant operation, the high service pumps can provide approximately **1.86 MGD** of finished water to the system if the treatment capacity of the plant were increased. The high service pumps are currently able to deliver more flow to the filters and softeners than can currently be treated, so they are adequately sized for the current scenario, but may need to be upsized in the future if the plant capacity is increased significantly. It is estimated that the existing high service pumps will meet the city's needs in the conjunction with the ASR well for a population of about 21,000, approximately 5-6 years.

4.6.4.4 Filters

Based on operating experience, the filters have a current operating capacity of approximately 1,500 gpm, or 1.8 MGD in 20 hours of production, based on 3.9 gpm/sf with one filter cell out of service. This capacity is adequate to meet the projected water demands for a population of about 20,500, or approximately 5-6 years. Pilot testing the existing filters and various media options may increase the allowable filtration rate to allow the filters to be operated at a higher rate; however, the concern would be the increase in backwash frequency. Backwashing the filters more than once per week is not very practical at North Liberty due to the larger number of filter cells and time required to backwash. If the filtering rate can be increased to 4.5-5 gpm/sf, this increases the allowable production flow to 1,700 -1,900 gpm.

4.6.4.4.1 Filter Backwash Detention Tank

The existing Filter Backwash Detention Tank is undersized to handle all backwash flows from even a single horizontal pressure filter; however, plant staff has adapted their operating strategy and is able to backwash all 12 filter cells in a 6-hour period. Ideally, the tank would have a larger volume to allow for a faster backwash operation; however, it is not critical at this point. If an additional filter is added to increase the plant capacity or if the plant flow is increased to the point where more frequent backwashing is required, an additional Filter Backwash Detention Tank should be added as well to make sure that backwash operations for all filters do not take any longer than with the current plant operations.

4.6.4.5 Softeners

The softeners are the current treatment limitation of the existing plant and are undersized to handle existing plant flows and produce the original design quality water. The existing softeners are only rated for 1140 gpm (1.2 MGD) based on providing a softened water down to 128 mg/l as $CaCO_3$ total hardness. The flow through the softeners has been increased over the years to meet the increased water demands and the softeners are able to operate at 1300 gpm; however, the water quality provided is a lower quality product that is still considered to be a hard water. The average finished water hardness in 2012 was 193 mg/l as $CaCO_3$ total hardness which is 50% above the design quality. It should be noted that while this is lower quality water, it still meets all primary drinking water standards to protect public health. Additional softeners are required to meet current and future water demands and allow softening down to original design levels. Based on the operating experience of the existing plant, it does not appear possible to optimize the capacity of the existing softeners above the existing treatment rate of 1300 gpm. Even the 1300 gpm rate is not really a desirable treatment rate, since it results in a harder water.

Although it is not possible to increase the flow through the softeners and maintain adequate treatment, it is possible to increase the time required between regenerations by increasing the exchange capacity of the existing softeners. This can be accomplished by increasing the salt regeneration rate from 6 lbs per cubic foot of resin to 10 pounds per cubic foot of resin. The downside to this approach is that it is not an efficient method of gaining increased treatment capacity and has a premium treatment cost due to wasted salt. It would not be recommended as a normal operating practice, but may be utilized during extreme conditions to increase the exchange capacity approximately 30% if needed.

4.6.4.5.1 Salt Storage Tank

The existing salt storage tank (43 tons) does not have sufficient capacity to allow for adequate time between deliveries and should be upsized to handle at least 2-2.5 truckloads (100-125 tons) and at least a three week delivery period versus the 6-9 days currently experienced. Going with a minimum three week storage capacity is less than the 10 States Standards 30-day recommendation; however, IDNR would likely agree with the reduced storage volume if the city is comfortable with maintaining a 3-4 week delivery cycle year-round. Multiple storage tanks will be required to provide a minimum 3-week storage volume.

Also, the brine tank is located indoors and subject to accumulation of salt dust during filling operations. Salt residue creates a corrosive environment and requires a lot of maintenance to clean up and prevent corrosion. The existing salt storage tank should preferably be relocated to the outdoors or any new storage tanks should be installed outdoors.

4.6.4.6 Chemical Feed

Current chemical feed systems appear to be functioning properly and are not in need of repair or expansion.

4.6.5 Aquifer Storage & Recovery (ASR) Well

The ASR has been performing adequately in its first years of operation. As previously noted, 2012 was the first year the water was actually recovered and utilized. Tests have shown that there is some blending of the injected water with the native water, as was expected. One concern is due the higher levels of arsenic in the Jordan water. Careful monitoring of the recovered water will be necessary to ensure that the primary drinking water standard for arsenic is not violated. Continued use of the ASR should improve the quality of water recovered, as the “bubble” of treated water pushes back the native water. If ASR Well ultimately proves ineffective for water storage and recovery, it can be converted to a raw water supply well.

The other concern is ensuring that there is adequate redundancy for the ASR well in case of a failure. This could be provided through connection to an adjacent water system, as was originally planned. North Liberty currently has a 12-inch connection to the City of Coralville’s water distribution system although an inter-city agreement is not in place. The city should explore the potential for obtaining an agreement with a neighboring city to provide a back-up supply in case of emergency.

Alternatively, the redundant raw water supply well could serve as backup for the ASR. The raw water would need to be blended with the treated water, which would require modifications to the pumps and piping at the water plant or well. This would also result in reduced water quality, due to blending untreated water with treated water. This would not be an ideal situation, but may be acceptable during an emergency situation.

5

DISTRIBUTION AND STORAGE

5.1 Distribution System

The scope of this study did not include in-depth study or analysis of the distribution system. The distribution system consists mainly of 8-inch and 6-inch water mains, with some 12-inch mains. No known deficiencies have been reported in terms of pressure and flow. In the vicinity of the existing water plant, there are three 6-inch mains that distribute the treated water to the rest of the city. Previous pumping tests on the high service pumps indicated that there do not appear to be significant constraints in the distribution system in terms of limiting high service pumping rates within the pumping capacity of the high service pumps. If the plant flows are increased significantly beyond that, improvements to the distribution system in the vicinity of the plant may be necessary. Figure 5.1 shows the North Liberty water distribution system and tower locations.

5.2 Elevated Storage

The North Liberty water system currently has two (2) elevated storage tanks of 400,000 and 1 million gallon capacities for a total of 1.4 million gallons. The largest tank was placed in operation in 2007.

Ten State Standards (7.0.1) require that the minimum storage capacity shall be provided to meet domestic demands based on average daily consumption and fire flow demands, where fire protection is provided. This level of storage can be reduced if adequate source and treatment capacity is available to supplement peak demands of the system and if these units are provided with adequate emergency standby power. Excessive storage capacity should be avoided to prevent potential water quality deterioration problems.

The former IDNR drinking water standards specified that in order to provide adequate fire flows, the minimum storage provided shall be equal to the average day demand <u>or</u> the total of the design fire flow times the design duration of the fire flow plus eighty percent of the instantaneous peak flow rate for the water system times the design fire flow duration (Q_f x $T_f + 0.8Q_i$ x T_f). The instantaneous peak flow used in this report is based on the following formula in the Statewide Urban Design and Specifications (SUDAS).

$$Qi = ADD \times 7/P^{0.167}$$

Where, ADD = Average Day Demand (GPM)
P = Population in Thousands

FOX engineering

WATER MAIN MAP

WATER SYSTEM FACILITY PLAN
NORTH LIBERTY, IOWA

FIGURE:	5.1	
REVISION	NO.	DATE
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Based on the Insurance Services Office (ISO) 2008 Guide for Determination of Needed Fire Flow, the maximum needed fire flows for 1- and 2-family dwellings not exceeding 2 stories in height is 1500 gpm. For other types of habitational buildings, the maximum needed fire flow is listed at 3,500 gpm which would be applicable to apartments or commercial buildings. For example, for a 3-story, 5,000 square foot apartment building, the required fire flow varies depending on the method of calculation, but averages approximately 3,250 gpm for apartments without sprinklers and 2,000 gpm for apartments with sprinklers. According to ISO, flows up to 2,500 gpm require 2-hour duration and flows up to 3,500 gpm require 3-hours duration.

A city is not required to provide the highest level of fire protection for all types of development in a city. The Insurance Services Office (ISO) uses the Fire Suppression Rating Schedule (FSRS) to measure the major elements of a community's fire-suppression capability and develop a numerical grading called a Public Protection Classification (PPC) which is used in determining fire insurance rates. Three components of the PPC are based on (1) fire alarms, (2) fire department and (3) water supply which is 40% of the overall score. The water supply component is based upon fire-flow testing at representative locations and other various components.

For purposes of planning in this report, storage requirements for the calculated method will be evaluated based on three scenarios including: (1) the highest requirements for low density residential (1500 gpm for 2 hrs), (2) an in-between capacity for low and high-density residential/commercial (2,500 gpm for 2-hours), and (3) the highest listed requirement for high-density residential/commercial (3,500 gpm for 3 hrs). The storage requirements base on both the formula method and average day requirements are presented in Table 5.2.

Table 5.2 shows that the storage requirement based on the average day flow is controlling under all scenarios. Based on projected demands, existing storage capacity will be less than average day demand when the population reaches about 18,000 (projected year 2015). Planning for additional storage should be considered. For the Phase 1 population, a total of 2.25 million gallons (MGal) of elevated storage is recommended, which exceeds the current 1.4 MGal capacity. Similarly for the Phase 2 population, a total of 2.92 MGal of elevated storage is recommended.

Additional elevated storage should be provided for both the Phase 1 and 2 population projections to supplement the existing 1.4 MGal elevated storage capacity. An additional 0.85 million Mgal elevated storage tank should be provided for the Phase 1 population and 1.52 MGal for Phase 2.

Table 5.2. Storage Requirements under Various Fire Flows and Population Projections

	Phase 1 Design Population of 28,100 in 2027			Phase 2 Design Population of 36,500 in 2037		
Qave (gpm)	1,565 (2.25 mgd)	1,565	1,565	2,030 (2.92mgd)	2,030	2,030
Peak Factor	4.0	4.0	4.0	3.85	3.85	3.85
Qi (gpm)	6,260	6,260	6,260	7,815	7,815	7,815
Qf (gpm) =	1,500	2,500	3,500	1,500	2,500	3,500
Tf (min) =	120	120	180	120	120	180
Qf x Tf (gal)	180,000	300,000	630,000	180,000	300,000	630,000
0.8(Qi x Tf) (gal)	601,000	601,000	902,000	750,000	750,000	1,125,000
Total Storage by Formula (gal)(1)	781,000	901,000	1,532,000	930,000	1,050,000	1,755,000
Total Storage as Average Day (gal) (2)	2,250,000	2,250,000	2,250,000	2,920,000	2,920,000	2,920,000

(1) 0.8(Qi x Tf) + Qf x Tf (gal)

(2) Gray shading indicates storage requirement exceeds current elevated storage capacity (1.4 MGal) without considering the ASR well.

5.3 Summaries of Deficiencies in Distribution and Storage

5.3.1 Distribution System

The scope of this study did not include in-depth study or analysis of the distribution system. As previously noted, no known deficiencies have been reported in terms of pressure and flow. Increasing the existing plant flow significantly beyond its current capacity will likely require improvements to the distribution system in the vicinity of the plant. Other improvements in the distribution system will be related to growth in the community. Careful planning should be conducted to ensure that adequately sized water mains and loops are installed as the community grows. A water distribution model should be utilized for planning water main sizing, loops, and connections for future growth areas.

5.3.2 Elevated Storage

The existing elevated storage capacity of 1.4 million gallons (MGal) is adequate for existing average demands. If population growth is as projected, the average day demand will exceed the existing storage in the next 2-3 years. Ten States Standards recommends storage equal to the average day demand, but allows a reduction in storage volume where adequate backup power is provided, as is the case with North Liberty. As the City continues to grow additional storage will be required. Construction of additional storage should be considered in the next 5-10 years. The water distribution model previously discussed should be utilized to determine the best location of the tower(s).

For the Phase 1 population (28,100), a total of 2.25 MGal of elevated storage is recommended, which would require an additional 0.85 MGal of elevated storage. For the Phase 2 population (36,500), a total of 2.92 MGal of elevated storage is recommended, which would require an additional 1.52 MGal elevated storage volume. For purposes of this evaluation, it was assumed that two 0.75 MGal towers would be constructed. Additional analysis and modeling should be conducted to determine if it would be most beneficial to add one large tower versus two smaller towers.

6

WATER SUPPLY SOURCE INVESTIGATION

6.1 General Discussion

A detailed water supply source investigation was not conducted as part of this facility plan update. A detailed investigation was conducted as part of the 2006 report and the results are considered to be applicable to the current evaluation as well. For planning it is assumed that additional water supply source will be through the addition of multiple Jordan wells as required.

7 Water Supply and Treatment Alternatives

7.1 General Introduction

Water Supply and Treatment Alternatives presented in this report were selected in conjunction with the analysis and recommendations of the 2006 report. In preparation of the draft Water System Facility Plan, 13 options were developed in detail and discussed with the city. This list of options was then narrowed down to four (4) options for presentation in the final report to simplify the report and to eliminate those options that may not be viable solutions. For water supply, the 2006 report included a detailed evaluation which is considered to be applicable to this evaluation as well. Where additional water supply is required for future alternatives, it is assumed that additional Jordan wells will be added as required to meet the water supply needs in similar locations as presented in the previous evaluation.

For treatment alternatives, the 2006 report included lime softening, electrodialysis reversal (EDR), and Reverse Osmosis (RO). For this report, treatment options considered include expansion of the existing ion-exchange (IX) softening plant at the existing site in combination with construction of a new supplemental or replacement RO plant. Lime softening was not considered due to its non-favorable ranking in the 2006 report and EDR was not included since it is supplied by a single manufacturer which is a concern for capital costs and pricing for future replacement parts , although it would be a viable alternative. It is assumed that the overall costs for EDR would be similar to the RO options based on results from the 2006 report.

Although consideration of expanding the existing IX on the existing plant site was considered, construction of a new supplemental or replacement ion-exchange (IX) softening plant on a new site was not. These options were considered in preparation of a draft facility plan and discussed with the City. It was decided to eliminate the construction of a new IX softening plant at a new site as a viable option due to concerns with elevated chloride levels from salt used in the softening process which gets discharged to the wastewater treatment plant. Chloride is a regulated contaminant of concern at the wastewater facility and there is a concern that the elevated chloride in the drinking water may cause chloride violations at the wastewater plant. There is also a potential that the IDNR may not allow the construction of a new IX plant in cities such as North Liberty looking to implement a “major upgrade” where chlorides are a concern. The option of expanding the existing IX plant to the extent possible on the existing site was seen to be a potential cost effective alternative and was left in even while it is realized that the IDNR may not allow or look favorably on such expansion.

As discussed in Section 3, the City of North Liberty is currently experiencing rapid growth which continues to outpace even optimistic predictions from previous evaluations. With an uncertain future, planning for future water system needs for the City is difficult. Options considered should provide flexibility for expansion as the City grows. A planning period of twenty–five years has been selected, with construction of facilities being implemented in two phases. The first phase would provide capacity for a population of approximately 28,100 (year 2027 projected population) and a peak day demand of about 4.2 MGD or required treatment plant capacity of 3.0MGD in combination with the current ASR well. The second phase would provide capacity for a population of approximately 36,500 (year 2037 projected population) and a peak day demand of 45.5 MGD or required treatment plant capacity of 4.2 MGD in combination with the current ASR well. The supply and treatment alternatives selected for further evaluation for each of these planning periods is discussed in the following sections.

7.2 Water Supply and Treatment Alternatives

A summary of the required treatment plant design capacities used as the basis of planning with and without the ASR well in operation are summarized in Table 7.2.1 as previously discussed in Sections 3 and 5. Note that the daily design flows (MGD) have been rounded up to the nearest tenth for simplification and ease of discussion.

Table 7.2.1 Water Treatment Plant Required Design Capacities

		ADD @ 80 gpcd	MDD @ 150 gpcd	WTP Design Capacity with ASR Well 7		WTP Design Capacity without ASR Well 7	
Year	Population	(MGD)	(MGD)	(MGD)	(GPM) (1)	(MGD)	GPM) (1)
2027	28,100	2.3	4.2	3.0	2,500(1)	4.2	3,520(1)
2037	26,500	2.9	5.5	4.0	3,500(1)	5.5	4,570(1)

(1) Assumes 20 hours WTP operation with current treatment technology. Required flowrate will vary with different technologies.

The alternatives identified for further evaluation are listed in table 7.2.2 below. These alternatives are described in further detail in the following sections.

Table 7.2.3 Water Supply and Treatment Alternatives

No.	Description
1.	Optimize Capacity of Existing Water Treatment Plant (WTP)
2.	Upgrade Capacity of Existing WTP for Phase 1 (3.0 MGD) and New RO Softening Plant at New Site for Phase 2 (4.2 MGD) (Utilize ASR Well)
3.	New Reverse Osmosis Softening Plant at New Site to Supplement Existing WTP (1.5 MGD expandable to 2.7 MGD) (Utilize ASR Well)
4.	New Reverse Osmosis Softening Plant at New Site to Replace Existing WTP (3.0 MGD expandable to 4.2 MGD) (Utilize ASR Well)

7.3 Alternative 1: Optimize Existing WTP

The potential to optimize the plant capacity beyond the current capacity is not considered to be feasible at this time. This is mainly due to the limitations of the existing softeners and the City's desire to produce a higher quality finished water. This was discussed further in previous sections of this report.

7.4 Alternative 2: Upgrade Existing WTP and New RO Treatment Plant

Alternative 2 involves upgrading the existing water treatment plant at the existing site including acquisition of adjacent property for Phase 1 improvements. Phase 1 improvements are estimated to meet the city's water demand for a population of about 28,100, estimated to occur in the year 2027. For Phase 2 improvements, a new supplemental RO softening plant would be constructed at a new site currently owned by the city. The new site is along the west side of South Front Street and approximately 0.5 miles from the existing plant to the southwest. The new plant in conjunction with the existing plant would be sized to meet the city's water demand for a population of approximately 36,500 persons (estimated 20-year design). Both phases would be sized to meet the city's peak water demands in conjunction with the city's existing ASR Well 7. Utilizing the ASR well allows the required treatment capacity to be downsized.

Additional raw water supply would need to be developed for the Phase 1 improvements. It is assumed that a Jordan well would be added in a similar location to that recommended in the 2006 report. This alternative and the treatment processes are further described in the following sections. Figure 7.4 illustrates the proposed location of the new wells, water main, and plant.

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ALT. 2 - UPGRADE EXISTING WTP (PHASE 1) & NEW RO PLANT TO SUPPLEMENT EXISTING PLANT (PHASE 2)

WATER SYSTEM FACILITY PLAN
NORTH LIBERTY, IOWA

FIGURE:	7.4	
REVISION	NO.	DATE
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7.4.1 Water Supply

Alternative 2 utilizes the Jordan aquifer to meet additional raw water supply demands. The Phase 1 population of 28,100 persons (estimated to occur in 2027) would require the construction of one new Jordan well. This new well is estimated to be required when the population reaches around 21,000. Assuming a minimum capacity of 1,100 gpm for the new well along with the existing Silurian and Jordan wells, this would provide a raw water capacity of about 2,660 gpm *with the largest well out of service*. This provides 3.19 MGD in 20 hours of operation or 3.83 MGD in 24 hours of operation. The required firm raw water capacity is **3.0 MGD** in conjunction with ASR Well 7 for Phase 1. The addition of a single Jordan well should be adequate to meet the water demands for Phase 1 with firm capacity in 20-hours operation.

For Phase 2, increased water supply is needed for the new 1.2 MGD reverse osmosis (RO) treatment plant process due to the water loss in the production of water. The recovery rate for an RO treatment system is assumed to be 75%. In other words, there is 25% water loss in the production of water through the RO system, so a higher water supply is required for a given water demand. In order to meet a target softening level of approximately 120 mg/l total hardness, it is assumed that 23% of the raw water flow will be bypassed around the RO system and 25% of the water through the RO system will be lost, which equals approximately a roughly 23% water loss in the production of water. For Phase 2, a required raw water supply flow of **4.6 MGD** is required. An additional Jordan well would be required for Phase 2 in order to maintain redundancy and allow for continued softening.

In addition to the wells, new raw water main would need to be constructed to connect the existing wells and new wells, and to transport the raw water to the new plant. For Phase 1 it is estimated that up to an additional mile of 12-inch raw water main would be required for tie-in of the new Jordan well. For Phase 2, it is estimated that approximately 1,800 feet of 16-inch water main would be required for tie-in to the new water treatment plant in addition to raw water main for the second Jordan well.

7.4.2 Treatment Plant

Alternative 2 includes upgrading the existing ion-exchange softening plant by increasing the treatable flow approximately 90% to a maximum flow of 2,500 gpm or 3.0 MGD in 20 hours operation for Phase 1 improvements. The existing treatment plant site is space limited. In order to accomplish this alternative additional property would be required. The plant capacity would be expanded by an addition on the south side of the existing water plant and installation of three (3) additional ion-exchange softeners similar to the existing units. The filter rate would be increased through the existing horizontal pressure filters and no additional filters would be installed due to space constraints. Since the Jordan wells are relatively low in iron and make up the majority of the plant flow, it is assumed that the plant would be able to bypass a portion of plant flow and still meet secondary limits with respect to iron and manganese. The feasibility of this option would need to be explored through on site filtration testing to confirm iron levels in various wells and allowable filtration rates. A filtration test was conducted in 2006 which showed that filtering near 6 gpm/sf would require increased backwash events. A similar test should be rerun with various combinations of wells in operation and alternate media could be explored as well.

The existing raw water detention tank and stand-by generator would have to be relocated. In order to prevent utilizing the adjacent park property, it is proposed under this alternative that the city acquire some property adjacent to the existing water treatment plant for location of a new raw water detention tank, stand-by generator, three 100-ton brine storage tanks and an additional filter backwash detention tank. Under this alternative, it is also proposed that a maintenance garage be installed on the expanded property.

For Phase 2, a new 1.2 MGD reverse osmosis softening plant would be constructed on a new site along South Front Street. The new plant would be used in conjunction with the existing plant to meet the Phase 2 demand of 4.2 MGD in conjunction with the operation of ASR Well 7. Two RO skids (400 GPM each) would provide the required flow in 20 hours operation with 50% redundancy in the event that one RO skid was out of operation. The RO skids would be equipped with a cartridge filter and an RO pump. The RO units would operate at over 160 psi. RO manufacturers prefer a minimum supply pressure of 40 psi into the cartridge filter upstream of the RO booster pump to provide enough driving head to accommodate the headlosses through the cartridge filter and provide a minimum of approximately 17-20 psi on the suction side of the booster pumps. In-line booster pumps would be provided on the raw water supply lines from the wells to boost the supply pressure to meet this requirement.

With an RO treatment system, additional high service pumps (in addition to the RO pumps) would also be included to pump treated water into the distribution system.

The advantage of this alternative is that it offers a lower Phase 1 cost which may allow a lower rate increase initially. The disadvantage of this alternative is that it requires the operation of two plants down the road which has increased operating and maintenance requirements. It also continues to use the ion-exchange softening process, with the associated issues with sodium and chlorides as previously noted.

7.4.2.1 Treatment Process Description – Ion-Exchange

For Alternative 2, ion-exchange (IX) softening is the process that will be continued for Phase 1 with upgrade of the existing plant.

IX softening is the technology that in addition to being used at the existing water treatment plant is used in most home water softeners. In general, the process involves passing the untreated water through a vessel packed with ion-exchange resin beads.

During the time that the water is in contact with the IX resin, hardness minerals (calcium and magnesium molecules) are transferred to the resin in exchange for sodium molecules. Most resins that work for softening water will also have a high affinity for radium and will remove this as well.

With the high raw water hardness and TDS that North Liberty has to treat, it would be necessary to treat approximately 80-85% of the raw water to meet the total hardness goal. By not treating all of the flow, the IX equipment is slightly smaller, which results in a lower capital cost relative to treatment technologies that must treat all the raw water flow. Blending is also needed to reduce the corrosiveness of water. Multiple units would be necessary in order to maintain the finished water quality needed when one unit is offline for regeneration and backwash.

The primary disadvantage of the IX technology is the addition of sodium to the drinking water and chloride to the wastewater. With regards to sodium, the targeted ions being removed are replaced with sodium. The US EPA guidance level for sodium is 30 to 60 mg/L and this is based on taste. Some arguments could be made that too much sodium in water may increase health risks to people with high blood pressure, diabetes, or kidney disease. However, the vast majority of sodium ingested by people would likely come from the food they eat. In 2008, the American Heart Association suggested that the adult maximum daily intake of sodium should be 2300 mg. Chloride added to the wastewater during softener regeneration leads to elevated chloride levels which may cause violation of wastewater effluent standards. A wastewater evaluation is currently being conducted which will confirm if continued use of IX technology is acceptable based on chloride levels.

The 2006 engineering study did not include IX softening as a treatment technology for future plant expansion, since it is not typically suited for larger installations and has the downside of elevated chloride levels which may be a concern at the wastewater treatment plant. The IX technology is considered within this report, since the city is familiar with this technology and it was desired to see if the existing technology could be expanded on the existing site as a cost-effective solution. The potential for expansion on new sites was added as well as a thorough consideration of the potential for this technology compared to reverse osmosis.

Another downside to the IX technology is the high cost of the salt needed to regenerate the resin and the difficulties of residuals disposal. Given the finished flow rate for North Liberty and its treatment goal of a total hardness of 120 mg/L as $CaCO_3$, preliminary estimates indicate that it would currently cost around $200,000 per year to purchase the salt and meet target hardness levels. Over the next twenty years, the present worth value of this annual expenditure is $3.6M (assuming a rate of inflation at 3%).

7.4.2.2 Treatment Process General Description – Reverse Osmosis

For Alternative 2, ion-exchange softening would be continued for Phase 1 with upgrade of the existing plant (Section 7.4.2.1). For Phase 2, a new RO treatment plant would be constructed to supplement the existing plant.

7.4.2.1.1 RO General Description

Reverse osmosis (RO) is a process in which water is forced through a semi-permeable membrane under high pressure while salts and other dissolved solids are retained. RO systems are most typically used for desalination of sea water or brackish water, but have also been used for dissolved solids removal in freshwater sources. RO membranes typically remove contaminants down to 0.0001 μ in size (25,400 μ = 1 inch). Typical operating pressures and recoveries range from 300 psi and 75% to 85% for brackish waters to 1200 psi and 40% to 50% for sea water. For freshwater sources operating pressures may be as low as 70 psi with recoveries from 75% to 90%. In softening applications, since the RO process removes the majority of dissolved solids, a portion of the feed water would be bypassed around the RO unit and blended with the permeate to produce the desired finished water quality.

RO effectively removes most contaminants, including cysts, bacteria, viruses, dissolved solids (salts, hardness), natural organic material, synthetic organic material, and pesticides. While RO is effective at removing dissolved solids, suspended solids tend to foul the membranes and cause operational problems. To remove suspended solids, fairly intense pretreatment is required.

7.4.2.1.2 RO Major Components

Pretreatment: Pretreatment is required for RO systems to minimize fouling and damage to the membranes, and maximize cleaning intervals and membrane life. The amount of pretreatment required depends on the source water, treated water quality goals, membrane type, and membrane design criteria (flux, recovery). Pretreatment may range from a simple cartridge filter up to coagulation, flocculation, sedimentation, and filtration, or even lime softening. For the Jordan aquifer, it was again assumed that pretreatment for iron removal would not be required. This would also need to be confirmed by pilot testing. Assumed pretreatment requirements include acid and antiscalant addition, followed by a cartridge filter for membrane protection (typically provided by the membrane manufacturer).

RO Membrane System: The RO membrane elements are contained in pressure vessels and skid mounted. The units come pre-plumbed from the factory in standard size modules. Feed pumps are used to pressurize the flow and force it through the membranes. Other auxiliary equipment includes a clean-in-place (CIP) system which use chemicals to clean the membranes, piping, and acid/antiscalant feed systems and cartridge filters for pretreatment.

Post Treatment and Chemical Addition: Post treatment after a RO system typically consists of stabilization and/or chemical addition. Caustic soda (or other chemical) may also be used for pH adjustment. Disinfection to destroy pathogens is often accomplished by the addition of chlorine gas. Fluoride addition would also be needed.

Ground Storage Reservoir and High Service Pumping: The ground storage reservoir and high service pumping requirements will be similar to Alternative 2 Phase 2, except that finished water storage would be provided in lieu of a raw water detention tank.

Chemical Storage and Handling
In general, chemical requirements will be similar for RO and ion-exchange softening, except that RO would not require brine. Chlorine and fluoride storage and handling will be similar. Pretreatment chemicals (chemical oxidants, acid, antiscalants) would be additional, and should be stored in a separate room. Adequate ventilation should be provided along with the use of non-corrosive materials, such as PVC and FRP.

Building and Lab Space
The building and lab space requirements for a RO plant would be similar to the ion-exchange softening plant.

7.4.3 Water Storage

For Alternative 2, it is assumed that an additional 0.75 million gallons of elevated storage would be required as part of both Phase 1 and Phase 2 (two total) system improvements. However, the towers could be constructed at a later date as growth and demands dictate the need. The City should consider constructing the first tower in the next 5 – 10 years, based on current population projections. Prior to constructing a new tower, additional analysis and modeling should be conducted to determine the best location for the tower(s) and if it would be most beneficial to add one large tower versus two smaller towers in separate phases.

7.5 Alternative 3: RO Treatment to Supplement Existing Plant

Alternative 3 involves construction of a new RO treatment to supplement the existing plant and ASR Well 7. Phase 1 would involve maintaining the existing water plant in operation and building a new 1.5 MGD supplemental RO softening plant In Phase 2 the RO plant would be expanded to 2.7 MGD. The existing plant and ASR Well 7 would remain in service for Phase 2 as well. The new plant would be constructed at a new site currently owned by the city along the west side of South Front Street similar to the previous alternatives.

Additional raw water supply would need to be developed for the Phase 1 improvements. It is assumed that a Jordan well would be added in a similar location to that recommended in the 2006 report. This alternative and the treatment process is further described in the following sections. Figure 7.4 previously presented for Alternative 2 illustrates the proposed location of the new wells, water main, and plant for Alternative 3.

7.5.1 Water Supply

Alternatives 3 utilizes the Jordan aquifer to meet additional raw water supply demands and involves supplementing the existing water plant with a 1.5 MGD RO plant for Phase 1 that is expanded to 2.7 MGD for Phase 2. The raw water demand based on the previous assumptions for RO treatment supply requirements as outlined under Alternative 2 is **3.5 MGD** for Phase 1 and **5.0 MGD** for Phase 2. For this alternative, the addition of a single Jordan well with a minimum flow of 1,100 GPM would provide a firm capacity of 2660 GPM and is able to produce 3.19 MGD in 20 hours of operation or 3.83 MGD in 24 hours of operation. One new Jordan well would meet the Phase 1 water demands. The well would not be needed until the population reaches approximately 21,000 (projected year 2019).

For Phase 2 a second new Jordan well with a minimum flow of 1,100 GPM would be required to boost the firm capacity to approximately 3,760 GPM to produce 4.33 MGD in 20 hours of operation or 5.41 MGD in 24 hours of operation.

7.5.2 Treatment

7.5.2.1 Alternative 3 Treatment

Alternative 3 utilizes the existing IX technology at the existing plant and the RO technology previously discussed under Alternative 2 for the new plant. For Phase 1, the existing water plant will remain in operation at the current operating capacity (1.56 MGD) and a new supplemental RO plant (1.5 MGD) will be constructed at a new site currently owned by the city along South Front Street. The property to the east of the existing plant would need to be acquired to allow for relocation of the existing raw water detention tank onto this site. Also, two (2) 75-ton brine storage tanks would be located on this property.

At the new RO plant site for Phase 1, two RO skids (600 GPM each) would provide the required flow and provide greater than 50% redundancy in the event that one RO skid was out of operation. The RO skids would be equipped with a cartridge filter and an RO pump. The RO units would operate at over 200 psi. In-line booster pumps would be provided on the raw water supply lines from the wells to boost the supply pressure to meet the RO skid requirements. For Phase 2, a third skid with a similar capacity would be added to the facility.

With an RO treatment system, additional high service pumps (in addition to the RO pumps) would also be included to pump treated water into the distribution system.

7.5.3 Water Storage

Water storage needs for Alternative 3 would be the same as the previous alternatives, an additional 0.75 million gallons of elevated storage for both Phase 1 and Phase 2.

7.6 Alternative 4: RO Treatment

Alternative 4 involves construction of a new RO treatment plant to replace the existing water plant. The ASR well would continue to be utilized for this alternative. A new 3.0 MGD RO softening plant would be constructed for Phase 1. The plant would be expanded to 4.2 MGD for Phase 2. The new plant would be constructed at a new site currently owned by the city along the west side of South Front Street similar to the previous alternatives.

Additional raw water supply would need to be developed for the Phase 1 improvements. It is assumed that a Jordan well would be added in a similar location to that recommended in the 2006 report. This alternative and the treatment process is further described in the following sections. Figure 7.6 presents the proposed location of the new wells, water main, and plant for Alternative 4.

7.6.1 Water Supply

Alternative 4 involves abandoning the existing Silurian wells to prevent the need for pre-filtration for iron removal and replacing the existing water plant with a 3.0 MGD RO plant for Phase 1 that is expandable to 4.2 MGD for Phase 2. For Alternative 4, the raw water demand based on the previous assumptions for RO treatment supply requirements as outlined under Alternative 2 is **3.9 MGD** for Phase 1 and **5.2 MGD** for Phase 2. For Phase 1, the addition of two Jordan wells with a minimum flow of 1,100 GPM each and abandonment of the existing Silurian wells would provide firm capacity of 3,300 GPM , or about 3.80 MGD in 20 hours of operation and 4.75 MGD in 24 hours of operation. For Phase 2 a third new Jordan well with a minimum flow of 1,100 GPM would be required to boost the firm capacity to 4,400 GPM to produce 6.34 MGD in 20 hours of operation or 5.07 MGD in 24 hours of operation.

In addition to the wells, new raw water main would need to be constructed to connect the existing wells and new wells, and to transport the raw water to the new plant. For each added Jordan well it is estimated that up to an additional mile of 12-inch raw water main would be required for tie-in. It is estimated that approximately 1,800 feet of 16-inch water main would be required for tie-in to the new water treatment plant.

FOX engineering

ALT. 4 - NEW RO PLANT TO REPLACE EXISTING PLANT

WATER SYSTEM FACILITY PLAN
NORTH LIBERTY, IOWA

FIGURE:	7.6	
REVISION	NO.	DATE
DRAWN JBA	PROJECT NO. 3373-12A	DATE 03-25-13

7.6.2 Treatment

Alternative 4 is similar to Alternative 3 except that the existing water plant would be abandoned and a new RO plant would be constructed at a new site in replacement of the existing plant. For Phase 1, the new RO plant would be sized for 3.0 MGD with two roughly 1.1 MGD permeate RO skids. For Phase 2, the plant would be upsized to 4.2 MGD with the additional of a fourth RO skid. For this alternative, three RO skids (660 GPM each) would provide the required flow and greater than 50% redundancy in the event that one RO skid was out of operation. The RO skids would be equipped with a cartridge filter and an RO pump. The RO units would operate at over 200 psi. In-line booster pumps would be provided on the raw water supply lines from the wells to boost the supply pressure to meet the RO skid requirements. For Phase 2, a fourth skid with a similar capacity would be added to the facility.

7.6.3 Water Storage

Water storage needs for Alternative 4 would be the same as the previous alternatives, an additional 0.75 million gallons of elevated storage for both Phase 1 and Phase 2.

8

EVALUATION OF SUPPLY AND TREATMENT ALTERNATIVES

8.1 Basis of Evaluation

Each of the alternatives outlined in the previous section were evaluated in order to determine which is the most feasible and beneficial. The evaluation was based on both economic, including capital and operation and maintenance costs, and non-economic factors. These evaluations are discussed in the following sections. For ease of reference, the alternatives considered are numbered and listed in Table 8.1 below.

Table 8.1. Alternatives Considered.

Water System Alternatives
1. Optimize Existing Water Treatment Plant (WTP)
2. Upgrade Existing WTP to 3.0 MGD (Phase 1) and Construct a New Reverse Osmosis (RO) WTP at a New Site to 4.2 MGD (Phase 2)
3. New RO WTP at New Site to Supplement Existing WTP (1.5 MGD expandable to 2.7 MGD)
4. New RO WTP at New Site to Replace Existing WTP (3.0 MGD expandable to 4.2MGD)

8.2 Capital Costs

Capital cost opinions are presented here as a general aid in evaluating the proposed alternatives. The estimates involve a significant amount of judgment at this early stage of planning and should be considered approximate in nature. Generally, planning level estimates are considered to have an accuracy of ± 20 to 30 percent. More refined estimates will be possible during subsequent design phases of any resulting project.

Capital cost opinions are total project costs and include: contractors' costs for construction, including overhead and profit, engineering fees for design engineering, field exploration, and construction-phase engineering services, legal and contract administration fees, land costs where applicable, and contingencies. All capital costs are based on current construction costs, with no allowance for inflation.

Costs were derived using previous FOX Engineering cost data, supplier quotations, and published cost data. Costs for purchasing water capacity from Iowa City or Coralville as an emergency back-up to the ASR well are unknown at this point in time. The capital cost opinions are based on 2012 cost data. Adjustment for inflation may be necessary if these estimates are referred to in the future.

8.3 Capital Costs of Alternatives

8.3.1 Alternative 1: Optimize Plant Capacity

The potential to optimize the plant capacity beyond the current capacity is not considered to be feasible at this time and no cost is presented for this alternative.

8.3.2 Alternative 2: Upgrade Existing Plant

A preliminary design of an upgraded treatment facility and future reverse osmosis plant was done in order to get a planning level estimate of the costs involved. The preliminary design was used to estimate material quantities, equipment and space requirements.

The opinion of probable cost for Alternative 2 is presented in Table 8.3.2. Costs include all anticipated capital costs such as construction of the water supply well(s), construction of the new treatment facility, construction of raw water mains, construction of elevated storage and engineering, legal, and administration fees. The costs are presented for the two phases of construction identified in Section 7. For Alternative 2, Phase 1 capacity is 3.0 MGD and Phase 2 is 4.2 MGD. Table 8.3.2 shows that Alternative 2 has a capital cost opinion of $27.5 million.

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Table 8.3.2 Opinion of Capital Cost: Alternative 2 – Upgrade Existing Plant for Phase 1 & New RO Softening Plant at New Site for Phase 2

	Engineer's Opinion of Probable Cost
Phase I	
Water Supply	$2,400,000
Raw Water Main	$790,000
Upgrade Existing WTP	$6,200,000
RO Treatment Plant	----
Elevated Storage	$2,800,000
Subtotal Phase I	$12,190,000
Phase II	
Water Supply	$2,400,000
Raw Water Main	$1,290,000
Upgrade Existing WTP	----
RO Treatment Plant	$8,800,000
Elevated Storage	$2,800,000
Subtotal Phase II	$15,290,000
Total	**$27,480,000**

8.3.3 Alternatives 3 & 4: RO Softening

A preliminary design of a supplemental and replacement RO softening plant was done in order to get a planning level estimate of the costs involved. The preliminary design was used to estimate material quantities, equipment and space requirements.

The opinion of probable cost for Alternatives 3 and 4 are presented in Table 8.3.3. These include all anticipated capital costs such as construction of the water supply well(s), construction of the new treatment facility, construction of raw water mains, construction of elevated storage and engineering, legal, and administration fees. The costs are presented for the two phases of construction identified in Section 7. For Alternatives 3 and 4, Phase 1 capacity = 3.0 MGD and Phase 2 = 4.2 MGD. Of these alternatives, Alternative 3 has the lowest capital cost opinion of $26.4 million.

Table 8.3.3 Opinion of Capital Cost: Alternatives 3 & 4 – New RO Softening Plant with ASR Well

	New RO Plant & Supplement Existing WTP Alt 3	New RO Plant & Replace Existing WTP (Abandon Silurians) Alt 4
Phase I		
Water Supply	$2,400,000	$4,800,000
Raw Water Main	$1,290,000	$2,080,000
Upgrade Existing WTP	$0	$0
RO Treatment Plant	$11,100,000	$9,500,000
Elevated Storage	$2,800,000	$2,800,000
Subtotal Phase I	$17,590,000	$19,180,000
Phase II		
Water Supply	$2,400,000	$2,400,000
Raw Water Main	$790,000	$790,000
Upgrade Existing WTP	----	----
RO Treatment Plant	$1,800,000	$2,000,000
Elevated Storage	$2,800,000	$2,800,000
Subtotal Phase II	$7,790,000	$7,990,000
Total	**$25,380,000**	**$27,170,000**

Note: Engineering, legal & administrative costs and contingencies have been included in the costs.

8.4 Summary of Capital Costs

For comparison purposes, the estimated capital cost for each alternative is summarized in Table 8.4. Of the new treatment plant alternatives, Alternative 3 has the lowest total capital cost opinion at $25.4 million. Alternatives 2, 3 and 4 are all within 10% of each other and are considered equivalent based on the planning level estimates. Alternative 2 has the lowest Phase 1 capital cost opinion of $12.2 million.

The capital costs for the new treatment plants presented above are based on a moderately sized building/support facilities (see Figure 8.1). If one of the new treatment plant options is selected, particularly one of the membrane options, there would be ways of reducing the capital cost such as eliminating the maintenance garage, break room, and reducing the size of the office, laboratory, chemical storage, and process areas; however the cost savings will not be that significant and will increase the per unit cost of the facility.

Table 8.4 Summary of Opinion of Probable Capital Cost.

Description		Phase 1 (mil $)	Phase 2 (mil $)	Total (mil $)	Rank
1	Optimize Existing WTP	Not Feasible			
2	Upgrade WTP within Site for Ph. 1 & New RO Plant - Utilize ASR (3.0 to 4.2 MGD)	$12.2	$15.3	$27.5	1
3(1)	RO WTP & Utilize ASR - Supplement Existing WTP (1.5 to 2.7 MGD)	$17.6	$7.8	$25.4	1
4(2)	RO WTP & Utilize ASR - Replace Existing WTP (3.0 to 4.2 MGD)	$19.2	$8.0	$27.2	1

(1) Silurian wells piped to existing WTP only.

(2) Abandon Silurian Wells

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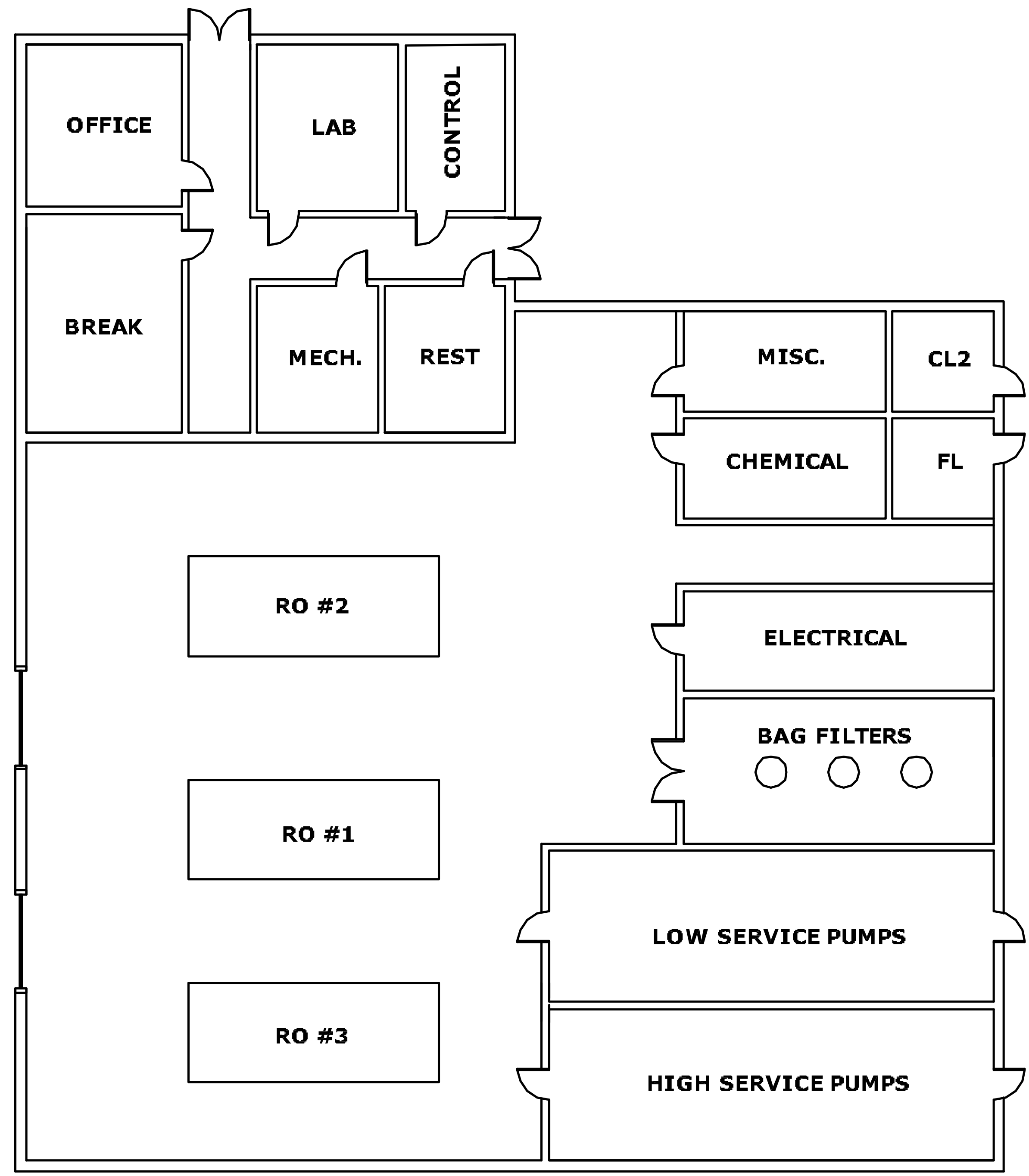

CONCEPTUAL FLOOR PLAN

SCALE: 1/16" = 1' - 0"

FOX engineering

NEW RO PLANT (ALT. 6B)

WATER SYSTEM FACILITY PLAN
NORTH LIBERTY, IA

FIGURE: 8.1		
REVISION	NO.	DATE
DRAWN	PROJECT NO.	DATE
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8.5 Operation and Maintenance Costs

A significant portion of the total annual cost of water treatment is for operation and maintenance costs. Operation and maintenance (O&M) costs include energy, chemicals, labor, and maintenance materials used to keep equipment and other facilities functioning. Actual O&M costs from the City's current facility, O&M cost data from similar facilities, quotes from chemical suppliers, and published data were used to develop O&M cost opinions for the alternatives evaluated. Because chemical costs and power make up a significant portion of the O&M costs, and use of power and chemicals is directly proportional to the quantity of water treated, O&M costs tend to increase over time as production increases. However, there are also fixed costs associated with O&M.

The city's annual O&M expenditures for 2011 were approximately $1.1 million subtracting out the costs for capital reserve, debt service and accounting. This results in a total cost of production of approximately $3.09/1000 gallon based on water pumped to the system. Of this cost, it was estimated that approximately $0.39/1000 gallon is currently expended on the purchase of salt based on current softening levels. If the facility were softening down to design levels on the order of 120 mg/l total hardness, the cost of salt would increase to $0.45/1000 gallon which would be 15% of the O&M cost.

For purposes of planning in this report, a cost of $0.45/1000 gallon was used for ion exchange softening. For RO treatment, a cost of $0.23/1000 gallon was used for the cost of pumping through the RO membranes, chemical addition and five-year membrane replacement costs. Where two types of treatment systems were proposed for a particular alternative, the costs were proportioned as necessary. Where two treatment plants were required to be operated and maintained a cost of $0.15/1000 gallons was added to the O&M costs. An additional fixed O&M cost of $2.70/1000 gal was added to the cost of each treatment alternative to match the City's existing O&M expenditures for energy, labor, maintenance and other operational requirements. This equates to a total O&M cost of $3.15/1000 gallons for a 100% ion-exchange treatment system or $2.93/1000 gallons for a 100% RO treatment system.

A summary of the total O&M costs over the 20-year planning period are included in the following section.

8.6 Present Worth Evaluation

In order to evaluate the alternatives on the same basis, the present worth of each was calculated. The following assumptions were used in preparing the life cycle costs.

- Interest rate equals 3.0 percent, based on past SRF loan rates.
- Salvage values and equipment replacement were considered.

- Service lives of structures and equipment were assumed to be:

Structures & Mains	40 years
Process Equipment	20 years
Pumps	10 years

- Salvage values were based on straight-line depreciation.

The above assumptions were used to calculate the present worth of each alternative which are summarized in Table 8.6. The O&M cost shown in the table is the present worth of all of the O&M costs throughout the planning period. Again, inflation was not considered.

Considering the accuracy of the analysis, those alternatives with a present worth cost within approximately 10 percent of each other may be considered equivalent. As can be seen in Table 8.6, Alternatives 2, 3 and 4 are all within 10% and considered to be equivalent for purposes of this evaluation.

Table 8.6 Present Worth Evaluation.

Alternative		Present Worth				
		Capital Cost (Mil $)	Salvage (Mil $)	O&M Cost (Mil $)	**Total (Mil $)**	Rank
1	Optimize Existing WTP	Not Feasible				
2	Upgrade WTP within Site for Ph. 1 & New RO Plant - Utilize ASR (3.0 to 4.2 MGD)	$27.5	($5.6)	$24.9	$46.8	1
3(1)	RO WTP & Utilize ASR - Supplement Existing WTP (1.5 to 2.0 MGD)	$25.4	($5.0)	$25.2	$45.5	1
4(2)	RO WTP & Utilize ASR - Replace Existing WTP (3.0 to 4.2 MGD)	$27.2	($5.9)	$23.2	$44.4	1

(1) Silurian wells piped to existing WTP only.
(2) Abandon Silurian Wells

8.7 Non-Economic Evaluation

The water treatment and supply options were evaluated on a noneconomic basis considering the following factors:

1. Land requirements.
2. System Control: Consideration was given to North Liberty's ability to make decisions and implement them, based upon City needs. Lower ratings were

given to systems with little local input and higher ratings were given to those with more local input.

3. Operational requirements: Consideration was given to operational complexity, operator attention requirements, and operator familiarity. Lower ratings were given to processes which require more operator attention and with multiple stages of operation.
4. Reliability: Alternatives were rated based on their ability to continuously process water. Consideration was given to external factors such as materials availability, weather conditions, and market availability.
5. Flexibility: Alternatives were rated based on their ability to adapt to changing process conditions or external factors. Higher ratings were given to alternatives with multiple processing options.
6. Ease of Expansion: Alternatives were rated on their ability to be easily expanded as system demands increase with community growth.
7. Monitoring requirements: Monitoring requirements were assessed based on general process monitoring requirements and requirements for documenting the process parameters.
8. Finished water quality.

The noneconomic comparison of alternatives is presented in Table 8.7. Numerical ratings from 1 to 5 were assigned to each factor for each alternative. A rating of 1 is poor (worst) and a rating of 5 is excellent (best). The highest (best) possible rating is 40. In general, those alternatives requiring the purchase of land not currently owned by the city were given lower ratings. Alternatives requiring operation of two plants had a lower rating. Alternatives using two treatment plants were given a higher rating for flexibility. Membrane systems offer a great deal of flexibility in operation, and were thus given the highest rating. Because of their modular design, membrane systems were given the highest rating for ease of expansion. Alternatives that utilize two treatment plants (existing and new), or membrane systems requiring iron removal for pretreatment received lower ratings for monitoring requirements. Finally, those options using the existing water plant were given the lowest water quality rating because of the ion exchange softening process.

Based on the noneconomic factors considered, Alternative 4 received the highest rating total of 30. This alternative is a RO treatment plant sized to replace the existing plant.

Table 8.7 Evaluation of Non-Economic Factors

Description		Land Requirements	System Control	Operation Requirements	Reliability	Ease of Expansion	Monitoring Requirements	Water Quality	**TOTAL**(1)	Rank(2)
1	Optimize Existing WTP	Not Feasible								
2	Upgrade WTP for Ph. 1 & New RO Plant - Utilize ASR	2	2	2	3	4	2	3	**18**	3
3	RO Plant & Utilize ASR - Supplement Existing WTP	4	2	2	3	4	2	3	**20**	2
4	RO Plant & Utilize ASR - Replace Existing WTP	3	4	4	5	5	4	5	**30**	1

(1) Rating Factor, 1 = worse; 5 = best.

(2) Lower Ranking is most Favorable

8.8 Project Plan Selection

A summary of the combined analysis described in the previous sections for monetary and non-monetary factors is shown in Table 8.8. For the combined analysis, monetary factors are given a 75% weighting and non-monetary factors are weighted at 25%. In comparing the alternatives, Alternative 4 has the best overall ranking. Alternative 4 – New RO Plant to Replace the Existing WTP is shaded in Table 8.8 as the recommended alternative.

Table 8.8 Summary of Analysis of Alternatives.

	Description	Total Net Present Worth (NPW) Rank	Weighting Factor (%)	(1) Total NPW Weighted Rank	Non-Monetary Factors Rank	Weighting Factor (%)	(2) Total Non-Monetary Factors Weighted Rank	Total Combined Weighted Rank (1)+(2)	Overall Rank
1	Optimize Existing WTP	Not Feasible							
2	Upgrade WTP for Ph. 1 & New RO Plant - Utilize ASR	1	75%	0.8	3	25%	0.8	1.5	**3**
3	RO Plant & Utilize ASR - Supplement Existing WTP	1	75%	0.8	2	25%	0.5	1.3	**2**
4	RO Plant & Utilize ASR - Replace Existing WTP	1	75%	0.8	1	25%	0.3	1.0	**1**

(1) Rating Factor, 1 = worse; 5 = best.
(2) Lower Ranking is most Favorable
(3) Shaded areas are two recommended options.

Financial, technical, operational, and administrative considerations all must be weighed in the final decision-making process. Once a decision is reached, then discussions can proceed on various preliminary design aspects associated with the selected plan. Some of the recommendations and analyses discussed in this report may merit more detailed examination. During the design development stage, numerous decision points will arise regarding specific features of the proposed project. The City can then decide which of the recommendations to include in its selected plan and which deviations to make from the concepts proposed by this analysis.

8.9 Financing Options

The options available to finance the proposed capital improvements are relatively limited. Major municipal capital improvements are normally funded by selling bonds. The bonds can be General Obligation (G.O.) Bonds that are normally repaid through a property tax levy or Revenue Bonds that are normally repaid through user fees paid by the customers. Each has some advantages and disadvantages that should be considered. G.O. bonds usually attract about a 1% better interest rate than Revenue Bonds since they are supported by the community’s ability to levy taxes. There is, however, a limit on the amount of G.O. Bond indebtedness a community can have (5% of property value) and this is the only source of capital for a number of community improvements. Considering the growth of the community and other likely demands of GO bonding ability in supporting this growth, use of GO bonds is not considered the best source of funding for water system improvements.

Revenue bonds are supported by the user fees generated in the utility system. The City can set those fees at any rate required to generate funds to repay the debt. The interest paid on these bonds, as with the G.O. bonds, is tax free to the investor and therefore is lower than an open market loan. The investors typically require that the water utility have a rate structure in place that generates from 25% to 40% higher revenue than anticipated expenses in order to attract the best interest rate. This is a viable financing option and should be considered.

The State Revolving Fund (SRF) loan program is another viable option to finance improvements to community water systems. Recent reductions in the interest rate make this an even more attractive option. The loans have an effective interest rate of 2% and a repayment period of up to twenty (20) years for most communities. There are some additional regulations, applications, and permits required for this process, but it should be considered as a very viable source of funds for this Project.

The financial impact on the North Liberty water system users will be significant regardless of the expansion alternative selected. The implementation of any of the alternatives under consideration will no doubt be the driving force for a significant water rate increase.

9
RECOMMENDATIONS

9.1 Recommendations for Implementation

Based on the facility planning efforts summarized in this report, the choices available to the City for moving forward to insure adequate water supply for the anticipated future growth fall into three main categories including: (1) upgrading the existing water treatment plant to the extent possible prior to building a new RO treatment plant at another location, (2) maintaining the existing plant at its current capacity and building a new supplemental RO treatment facility at another location, or (3) replacing the existing water treatment plant with a new RO treatment facility at another location.

Based on a 20-year present worth-analysis, the option to maintain the existing plant and build a new supplement RO plant down the road (Alternative 3) is equivalent to the option for replacing the existing plant with a new RO plant at a new site (Alternative 4), since they are within 10% based on a 20-year present worth evaluation. Between Alternatives 3 and 4, the preferable option would be Alternative 4 to build a new replacement RO plant, since it eliminates the requirement to operate and maintain two separate treatment facilities which may require increased staff and concerns with mixing of two varying water qualities. Alternative 4 is also the number one ranked alternative based on combination of present-worth analysis (75% weighting) and non-monetary factors (25% weighting) out of all of the alternatives considered.

In summary, the recommended option for increasing the water system capacity to keep up with the future water demands due to the city's growing population is Alternative 4 as listed below.

Recommended Alternative for Treatment System Improvements:

Alternative 4: Construct a new RO Softening Plant on a New Site to Replace the Existing WTP (3.0 MGD for Phase 1 expanded to 4.2 MGD for Phase 2) (Utilize ASR)

The advantages of Alternative 4 are that it utilizes a membrane technology that provides a higher quality water and does not require the addition of salt as part of the softening process. Salt (sodium chloride) increases the amount of sodium uptake in a daily diet and also increases the chloride levels to the wastewater plant. Membrane technology can also be relatively easily expanded in modular units. The disadvantages of Alternative 4 are that membrane technologies require more raw water, since they have a 20-25% loss of water during water production as compared with 5-10% loss for cation exchange softening.

The opinion of probable capital cost for the 20 year planning period for Alt.4 is $27.2 million. This is based on the population growing to 36,500 by year 2037. Because of the rapid population growth in North Liberty, actual population served will have the greatest impact on when the improvements will be required. Table 9.1 shows the improvements and the corresponding population when the improvement(s) will be needed. The table also shows the estimated year, based on population projections.

While the City is moving forward with the planning of water system improvements, it is also recommended that the City have Shive Hattery re-inspect the interior of the Raw Water Detention Tank at the plant and plan for routine inspections over the next few years until the new plant is built and the tank can ultimately be abandoned. The existing Raw Water Detention Tank is a 29,000 gallon welded steel tank that was installed with the original plant in the 1970s. Shive Hattery inspected the existing tank in May 2011 and recommended that the tank be taken out of service and the roof cap repaired in the next couple years. Repairing the tank is a major undertaking which would require the tank to be out of service for an extended period with special accommodations in place to operate the existing plant without the tank. If the new plant is constructed within the next few years, repair of the tank may not be necessary.

Table 9.1. Phased Implementation of Recommended Improvements

ALTERNATIVE 4 – New RO WTP at New Site to Replace Existing WTP				
	Capital Cost ($ Million)	Population[1]	Estimated Year[2]	Total By Population (or Est. Year)
Phase 1				
Phase 1A:				
Treatment Plant	$9.5	19,700	2017	$13.2
Raw Water Main	$1.3			
New Jordan Well	$2.4			
Phase 1B:				
Elevated Storage	$2.8	22,000	2020	$2.8
Phase 1C:				
Raw Water Main	$0.8			
New Jordan Well	$2.4	27,500	2023	$3.2
Total Phases 1A – 1C	**$19.2**			
Phase 2				
Phase 2A:				
Treatment Plant	$2.0	29,000	2028	$2.0
Phase 2B:				
Elevated Storage	$2.8	31,000	2030	$2.8
Phase 2C:				
Raw Water Main	$0.8			
New Jordan Well	$2.4	33,000	2033	$3.2
Total Phases 2A-2C	**$8.0**			

Notes: (1) Population when improvement is recommended to be in place.
(2) Estimated year when improvement is recommended, base on population projections.

Given these considerations, the following recommendations are offered:

1. The concepts presented in this Facility Plan should be reviewed and discussed. Next, decisions should be made regarding the specific features and components to be included in the selected plan. The City should concur with the proposed components as presented or direct that revised analysis be made.

2. The final facility plan should be submitted to the IDNR for review and approval.

3. Following acceptance of the facility plan by the City and comment by IDNR, pilot testing of the membrane systems should begin in approximately one year. The pilot test will be used to confirm or update analysis presented here, and help assure the most cost effective system is selected. One major component of the pilot testing would be to determine if preliminary treatment for iron removal is required, as well as required chemicals and doses for membrane cleaning.

4. The preliminary design phase should be initiated, as appropriate. The City should anticipate and allow for a three year time period required for design and construction of any improvements.

5. Actions should be taken as soon as possible for the City to obtain a formalized agreement with the City of Coralville or Iowa City for 1.3 MGD of water during an emergency condition if the ASR well were to fail during a peak day demand. If a formalized agreement cannot be obtained for sufficient capacity to match the ASR well peak day capacity, then additional considerations should be explored for emergency conditions when the ASR well is down, including: (1) increased treatment capacity (converting the ASR to a supply well),(2) blending raw water with finished water as long as blended water can meet drinking water standards and (3) an additional ASR well for redundancy.

6. Finally, as recommended in the 2006 report, the City should begin or continue discussions within their own community among interest groups regarding water conservation options. Although Iowa is not typically considered to be a water scarce region, the availability of adequate and economical sources of potable water within specific small areas is becoming more of a challenge here as elsewhere. Use of conservation measures, either voluntarily or by ordinance, may be required at some point in many Iowa communities and beginning discussions of such measures before they become required may help pave the way for acceptance of such measures. Such measures can take the form of surcharges or premiums paid for water use during peak demand periods, water rationing options for such uses as irrigation, voluntary or required alternate day irrigation, specific design and operating requirements for irrigation systems to optimize such uses and level demand rates, and education programs regarding use of low water use landscaping. The lowered peak day per capita water usage from the 2006 report is evidence that water conservation measures are already being implemented throughout the city and should be encouraged to continue.

9.2 Impact on User Rates

Based on the proposed improvements, the City Administrator and financial advisors performed a rate analysis to determine the impact on user rates. Refer to Appendix B for details of the rate analysis and proposed increases. The rate projections were prepared based on projected revenues and expenditures through fiscal year 2025. The projections were based on estimated revenue increase of 2% per year. The projections show that rate increases will be necessary through fiscal year 2021 to fund the needed improvements. Rate increases will vary from year to year, but will range between 3% to 15%.

9.3 Proposed Schedule

A proposed implementation schedule of the recommended alternative is provided below for consideration.

Table 1.5. Proposed Implementation Schedule

Approve Facility Plan and Submit to IDNR	June 2013
Submit SRF Intended Use Plan Application	June 2013
R.O. Pilot Testing	May - Aug. 2014
Design Engineering	Oct. 2014 - June 2015
Bidding	August 2015
Construction/New WTP Start-Up	Sept. 2015 – June 2017
Construction/New WTP Final Acceptance	September 2017

APPENDIX A

RAW WATER DATA

State Hygienic Laboratory

The University of Iowa

Accession Number	68166
Date Sample Finalized	2012-12-05 16:09
Date Received	2012-11-08 09:20
Sample Source	Drinking Water
Project	
Date Collected	2012-11-08 08:18
Collection Site	well #5
Collection Town	NORTH LIBERTY
Sample Description	
Client Reference	fox engineering
Collector	pentecost michael
Phone	319/560-7261

NORTH LIBERTY WATER SUPPLY
3 QUAIL CREEK CIR
PO BOX 77
NORTH LIBERTY, IA 52317

Note: Upon arrival, sample met container and preservation requirements for the analysis requested. Please review carefully your sample results for additional analyte comments or method exceptions.

Results of Analyses

Uranium, EPA 200.8

Units	ug/L	Analyzed In	Ankeny
Date Analyzed	2012-11-13 12:14	Date Verified	2012-11-15 14:37
Analyst	SGB	Verifier	TAB
Analysis Prep	Turbidity screen for drinking water metals		

Analyte	Result	Quant Limit	MCL
Uranium	<1.0	1	30

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Gross Alpha (ineluding Uranium), EPA 00-02

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-11-16 16:44	Date Verified	2012-11-19 10:22
Analyst	MAM	Verifier	WB
Analysis Prep	Sample Check-In		

Analyte	Result	Uncertainty	Quant Limit
Gross Alpha including Uranium	16.6	2.4	1.0

Note: This method was performed to eliminate interference from dissolved solids that prevent the use of the screening method, EPA 900.0. This EPA 00-02 method is more labor intensive, thus the cost is higher.

If the "Gross Alpha including Uranium" result is less than 15 pCi/L, for Iowa water supplies, the result will be considered the same as "Gross Alpha excluding Uranium". If the result is greater than 15 pCi/L then a "Gross Alpha excluding Uranium" test is also performed. The EPA has designated a maximum contaminant level (MCL) of 15 pCi/L for "Gross Alpha excluding Uranium" for public drinking water supplies. The MCL (is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Gross Alpha (excluding Uranium), EPA 00-02/200.8

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-11-19 13:57	Date Verified	2012-11-20 14:58
Analyst	MAM	Verifier	WB

Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
2220 S. Ankeny Blvd
Ankeny, IA 50023
515/725-1600 Fax: 515/725-1642

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The University of Iowa

Accession Number | 68166

Analyte	Result	Uncertainty	Quant Limit	MCL
Gross Alpha excluding Uranium	16.6	2.4	1.0	15

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Radium 226, EPA 903.0

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-12-05 08:54	Date Verified	2012-12-05 16:09
Analyst	DMM, MAM	Verifier	DMM
Analysis Prep	Sample Check-In		

Analyte	Result	Uncertainty	Quant Limit
Radium-226	3.8	0.5	0.8

Radium 228, EPA 904.0

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-11-21 12:51	Date Verified	2012-11-26 08:59
Analyst	DMM, MAM	Verifier	DMM
Analysis Prep	Sample Check-In		

Analyte	Result	Uncertainty	Quant Limit
Radium-228	2.3	0.7	0.6

Combined Radium 226 and 228, EPA 903.0/904.0

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-12-05 08:54	Date Verified	2012-12-05 16:09
Analyst	DMM, MAM	Verifier	DMM

Analyte	Result	Quant Limit	MCL
Combined Radiums	6.1	1.0	5.0

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Description of Units used within this report

pCi/L = PicoCuries per Liter
ug/L = Micrograms per Liter

The result(s) of this report relate only to the items analyzed. This report shall not be reproduced except in full without the written approval of the laboratory.

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If you have any questions, please call Client Services at 800/421-IOWA (4692) or 319/335-4500. Thank you.

Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
2220 S. Ankeny Blvd
Ankeny, IA 50023
515/725-1600 Fax: 515/725-1642

State Hygienic Laboratory

The University of Iowa

Accession Number	68167
Date Sample Finalized	2012-12-05 16:09
Date Received	2012-11-08 09:20
Sample Source	Drinking Water
Project	
Date Collected	2012-11-08 08:50
Collection Site	hydrant
Collection Town	NORTH LIBERTY
Sample Description	well #6
Client Reference	fox engineering proj
Collector	petecost michael
Phone	319/560-7261

NORTH LIBERTY WATER SUPPLY
3 QUAIL CREEK CIR
PO BOX 77
NORTH LIBERTY, IA 52317

Note: Upon arrival, sample met container and preservation requirements for the analysis requested. Please review carefully your sample results for additional analyte comments or method exceptions.

Results of Analyses

Uranium, EPA 200.8

Units	ug/L	Analyzed In	Ankeny
Date Analyzed	2012-11-13 12:14	Date Verified	2012-11-15 14:37
Analyst	SGB	Verifier	TAB
Analysis Prep	Turbidity screen for drinking water metals		

Analyte	Result	Quant Limit	MCL
Uranium	<1.0	1	30

Note: *The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).*

Gross Alpha (including Uranium), EPA 00-02

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-11-16 16:44	Date Verified	2012-11-19 10:22
Analyst	MAM	Verifier	WB
Analysis Prep	Sample Check-In		

Analyte	Result	Uncertainty	Quant Limit
Gross Alpha including Uranium	17.8	2.5	0.9

Note: *This method was performed to eliminate interference from dissolved solids that prevent the use of the screening method, EPA 900.0. This EPA 00-02 method is more labor intensive, thus the cost is higher.*

If the "Gross Alpha including Uranium" result is less than 15 pCi/L, for Iowa water supplies, the result will be considered the same as "Gross Alpha excluding Uranium". If the result is greater than 15 pCi/L then a "Gross Alpha excluding Uranium" test is also performed. The EPA has designated a maximum contaminant level (MCL) of 15 pCi/L for "Gross Alpha excluding Uranium" for public drinking water supplies. The MCL (is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Gross Alpha (excluding Uranium), EPA 00-02/200.8

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-11-19 13:57	Date Verified	2012-11-20 14:59
Analyst	MAM	Verifier	WB

Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
2220 S. Ankeny Blvd
Ankeny, IA 50023
515/725-1600 Fax: 515/725-1642

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Accession Number | 68167

Analyte	Result	Uncertainty	Quant Limit	MCL
Gross Alpha excluding Uranium	17.8	2.5	0.9	15

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Radium 226, EPA 903.0

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-12-05 08:54	Date Verified	2012-12-05 16:09
Analyst	DMM, MAM	Verifier	DMM
Analysis Prep	Sample Check-In		

Analyte	Result	Uncertainty	Quant Limit
Radium-226	5.5	0.5	0.4

Radium 228, EPA 904.0

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-11-21 12:51	Date Verified	2012-11-26 08:59
Analyst	DMM, MAM	Verifier	DMM
Analysis Prep	Sample Check-In		

Analyte	Result	Uncertainty	Quant Limit
Radium-228	2.0	0.7	0.6

Combined Radium 226 and 228, EPA 903.0/904.0

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-12-05 08:54	Date Verified	2012-12-05 16:09
Analyst	DMM, MAM	Verifier	DMM

Analyte	Result	Quant Limit	MCL
Combined Radiums	7.5	1.0	5.0

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Description of Units used within this report
pCi/L = PicoCuries per Liter
ug/L = Micrograms per Liter

The result(s) of this report relate only to the items analyzed. This report shall not be reproduced except in full without the written approval of the laboratory.
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Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
2220 S. Ankeny Blvd
Ankeny, IA 50023
515/725-1600 Fax: 515/725-1642

State Hygienic Laboratory

The University of Iowa

Accession Number	14257
Date Sample Finalized	2012-03-02 16:45
Date Received	2012-02-01 14:00
Sample Source	Drinking Water
Project	
Date Collected	2012-02-01 13:30
Collection Site	well hydrant 290 s chest
Collection Town	NORTH LIBERTY
Sample Description	well #5-raw
Client Reference	asr-injection
Collector	metternich greg
Phone	319/560-7261

NORTH LIBERTY WATER SUPPLY
3 QUAIL CREEK CIR
PO BOX 77
NORTH LIBERTY, IA 52317

Note: Upon arrival, sample met container and preservation requirements for the analysis requested. Please review carefully your sample results for additional analyte comments or method exceptions.

Results of Analyses

Anions, EPA 300.0

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-15 10:18	Date Verified	2012-02-17 12:41
Analyst	BW	Verifier	SB

Analyte	Result	Quant Limit	MCL
Chloride	30	1	
Sulfate	480	1	

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Anions, EPA 300.0

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-02 14:29	Date Verified	2012-02-07 14:48
Analyst	BW	Verifier	TAB

Analyte	Result	Quant Limit	MCL
Nitrate nitrogen as N	< 1.0	1.0	10
Nitrite nitrogen as N	< 0.1	0.1	1.0

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Alkalinity as CaCO3, SM 2320 B 18th

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-08 15:56	Date Verified	2012-02-10 08:50
Analyst	AB	Verifier	SB

Analyte	Result	Quant Limit
Total Alkalinity	250	1.0

Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
2220 S. Ankeny Blvd
Ankeny, IA 50023
515/725-1600 Fax: 515/725-1642

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Fluoride, SM 4500-F C 18th

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-10 15:30	Date Verified	2012-02-13 11:23
Analyst	BR	Verifier	DS

Analyte	Result	Quant Limit	MCL
Fluoride	1.4	0.1	4.0

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Total Hardness as CaCO3, SM 2340 C 18th

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-07 13:42	Date Verified	2012-02-07 16:24
Analyst	AB	Verifier	TAB

Analyte	Result	Quant Limit
Total Hardness	460	1.0

Laboratory pH, SM 4500 H+ B 18th

Units	pH	Analyzed In	Ankeny
Date Analyzed	2012-02-08 15:51	Date Verified	2012-02-21 16:30
Analyst	AB	Verifier	DS

Analyte	Result
Laboratory pH	7.5

Total Dissolved Solids (Dried at 180 degrees C), SM 2540

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-02 00:00	Date Verified	2012-02-06 16:23
Analyst	LD, MP	Verifier	DS

Analyte	Result	Quant Limit
Total Dissolved Solids	1000	1

Mercury, EPA 200.8

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-08 11:53	Date Verified	2012-02-10 08:31
Analyst	SB	Verifier	BW
Analysis Prep	Turbidity screen for drinking water metals		

Analyte	Result	Quant Limit
Mercury	<0.0002	0.0002

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act

Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
2220 S. Ankeny Blvd
Ankeny, IA 50023
515/725-1600 Fax: 515/725-1642

Uranium, EPA 200.8

Units	ug/L	Analyzed In	Ankeny
Date Analyzed	2012-02-02 16:01	Date Verified	2012-02-06 15:44
Analyst	SB	Verifier	DS
Analysis Prep	Turbidity screen for drinking water metals		

Analyte	Result	Quant Limit	MCL
Uranium	<1.0	1.0	30

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Metals, EPA 200.8

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-02 16:01	Date Verified	2012-02-06 15:44
Analyst	SB	Verifier	DS
Analysis Prep	Turbidity screen for drinking water metals		

Analyte	Result	Quant Limit	MCL
Antimony	<0.005	0.005	0.006
Arsenic	<0.001	0.001	0.010
Barium	<0.05	0.05	2
Cadmium	<0.001	0.001	0.005
Chromium	<0.01	0.01	0.1
Selenium	<0.01	0.01	0.05
Thallium	<0.001	0.001	0.002

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Metals, EPA 200.7

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-08 09:59	Date Verified	2012-02-10 09:29
Analyst	MC	Verifier	BW
Analysis Prep	Turbidity screen for drinking water metals		

Analyte	Result	Quant Limit	Quant Limit
Calcium	95	1.0	0.02
Magnesium	52	0.5	0.02
Sodium	160	0.5	0.02

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
2220 S. Ankeny Blvd
Ankeny, IA 50023
515/725-1600 Fax: 515/725-1642

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Dissolved Metals, EPA 200.7

Units	mg/L	Analyzed In	Ankeny
Date Analyzed	2012-02-08 09:58	Date Verified	2012-02-10 09:27
Analyst	MC	Verifier	BW
Analysis Prep	Turbidity screen for drinking water metals		

Analyte	Result	Quant Limit
Dissolved Iron	<0.02	0.02
Dissolved Manganese	<0.02	0.02

GCMS Volatiles, EPA 524.2

Units	mg/L	Analyzed In	Iowa City
Date Analyzed	2012-02-01 16:59	Date Verified	2012-02-05 10:53
Analyst	LL	Verifier	JM

Analyte	Result	Quant Limit	MCL
Vinyl chloride	<0.0005	0.0005	0.002
1,1-Dichloroethene	<0.0005	0.0005	0.007
Methylene chloride	<0.0010	0.0010	0.005
trans-1,2-Dichloroethylene	<0.0005	0.0005	0.1
cis-1,2-Dichloroethylene	<0.0005	0.0005	0.07
1,2-Dichloroethane	<0.0005	0.0005	0.005
1,1,1-Trichloroethane	<0.0005	0.0005	0.2
Carbon tetrachloride	<0.0005	0.0005	0.005
Benzene	<0.0005	0.0005	0.005
1,2-Dichloropropane	<0.0005	0.0005	0.005
Trichloroethene	<0.0005	0.0005	0.005
1,1,2-Trichloroethane	<0.0005	0.0005	0.005
Toluene	<0.0005	0.0005	1
Tetrachloroethylene	<0.0005	0.0005	0.005
Chlorobenzene	<0.0005	0.0005	0.1
Ethylbenzene	<0.0005	0.0005	0.7
Styrene	<0.0005	0.0005	0.1
Total Xylenes	<0.0005	0.0005	10
p-Dichlorobenzene	<0.0005	0.0005	0.075
o-Dichlorobenzene	<0.0005	0.0005	0.6
1,2,4-Trichlorobenzene	<0.0005	0.0005	0.07

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

GCMS Semivolatiles, EPA 525.2

Units	mg/L	Analyzed In	Iowa City
Date Analyzed	2012-02-07 14:07	Date Verified	2012-02-13 13:32
Analyst	SES, TC	Verifier	TC
Analysis Prep	Prep by Solid Phase Extraction, EPA 525.2		

Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
2220 S. Ankeny Blvd
Ankeny, IA 50023
515/725-1600 Fax: 515/725-1642

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Analyte	Result	Quant Limit	MCL
Simazine	<0.0001	0.0001	0.004
Atrazine	<0.0001	0.0001	0.003
Alachlor	<0.0001	0.0001	0.002
bis(2-Ethylhexyl)adipate	<0.0006	0.0006	0.400
bis(2-Ethylhexyl)phthalate	<0.0006	0.0006	0.006
Benzo(a)pyrene	<0.0001	0.0001	0.0002

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Prep by Solid Phase Extraction, EPA 525.2

Units	mg/L	Analyzed In	Iowa City
Date Analyzed	2012-02-06 07:00	Date Verified	2012-02-06 11:52
Analyst	PM	Verifier	GJ

Acid Herbicides, EPA 515.3

Units	mg/L	Analyzed In	Iowa City
Date Analyzed	2012-02-09 22:05	Date Verified	2012-02-16 20:22
Analyst	MM	Verifier	SM
Analysis Prep	Prep by Liquid-Liquid Microextraction, EPA 515.3		

Analyte	Result	Quant Limit	MCL
2,4-D	<0.0005	0.0005	0.07
Dinoseb	<0.0005	0.0005	0.007
Pentachlorophenol	<0.0002	0.0002	0.001
Silvex	<0.0002	0.0002	0.05
Dalapon	<0.0005	0.0005	0.2
Picloram	<0.0005	0.0005	0.5

Note: The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Prep by Liquid-Liquid Microextraction, EPA 515.3

Units	mg/L	Analyzed In	Iowa City
Date Analyzed	2012-02-07 08:00	Date Verified	2012-02-08 07:34
Analyst	MS	Verifier	GJ

Gross Alpha (including Uranium), EPA 00-02

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-02-11 10:50	Date Verified	2012-02-14 16:38
Analyst	MM, AC	Verifier	MM
Analysis Prep	Sample Check-In		

Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
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515/725-1600 Fax: 515/725-1642

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Accession Number | 14257

Analyte	Result	Uncertainty	Quant Limit
Gross Alpha including Uranium	13.3	2	0.8

Note: *This method was performed to eliminate interference from dissolved solids that prevent the use of the screening method, EPA 900.0. This EPA 00-02 method is more labor intensive, thus the cost is higher.*

If the "Gross Alpha including Uranium" result is less than 15 pCi/L, for Iowa water supplies, the result will be considered the same as "Gross Alpha excluding Uranium". If the result is greater than 15 pCi/L then a "Gross Alpha excluding Uranium" test is also performed. The EPA has designated a maximum contaminant level (MCL) of 15 pCi/L for "Gross Alpha excluding Uranium" for public drinking water supplies. The MCL (is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).

Gross Alpha (excluding Uranium), EPA 00-02/200.8

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-02-28 13:52	Date Verified	2012-02-28 16:54
Analyst	MM	Verifier	DM

Analyte	Result	Uncertainty	Quant Limit	MCL
Gross Alpha excluding Uranium	13.3	2	0.8	15

Note: *The MCL (maximum contaminant level) is only applicable to compliance monitoring samples under the Safe Drinking Water Act (SDWA).*

Radium 226, EPA 903.0

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2013-03-02 11:27	Date Verified	2012-03-02 16:45
Analyst	MM, DM	Verifier	DM
Analysis Prep	Sample Check-In		

Analyte	Result	Uncertainty	Quant Limit
Radium-226	4.4	0.5	0.6

Radium 228, EPA 904.0

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2012-02-17 12:56	Date Verified	2012-02-20 09:55
Analyst	MM, DM	Verifier	MM
Analysis Prep	Sample Check-In		

Analyte	Result	Uncertainty	Quant Limit
Radium-228	1.5	0.6	0.5

Combined Radium 226 and 228, EPA 903.0/904.0

Units	pCi/L	Analyzed In	Iowa City
Date Analyzed	2013-03-02 11:27	Date Verified	2012-03-02 16:45
Analyst	MM, DM	Verifier	DM

Analyte	Result	Quant Limit	MCL
Combined Radiums	5.9	1	5.0

Michael D. Wichman, Ph.D.
Michael A. Pentella, Ph.D.
Associate Directors
http://www.shl.uiowa.edu

University of Iowa Research Park
2490 Crosspark Road
Coralville, IA 52241
319/335-4500 Fax: 319/335-4555

Lakeside Laboratory
1838 Highway 86
Milford, IA 51351
712/337-3669 ext. 6 Fax: 712/337-0227

Iowa Laboratories Complex
2220 S. Ankeny Blvd
Ankeny, IA 50023
515/725-1600 Fax: 515/725-1642

APPENDIX B

RATE AND BUDGET PROJECTIONS

Water Utility, 2013 Facility Plan Improvements; Rate & Budget Projections

	FY 12 Audited	FY 13 Budget	FY 14 Budget	FY 15 Estimated	FY 16 Estimated	FY 17 Estimated	FY 18 Estimated	FY 19 Estimated	FY 20 Estimated	FY 21 Estimated	FY 22 Estimated	FY 23 Estimated	FY 24 Estimated	FY 25 Estimated
Budget Inflation Rate		2.28%	2.00%	2.00%	2.00%	2.00%	2.00%	2.00%	2.00%	2.00%	2.00%	2.00%	2.00%	2.00%
Number of Accounts	6,666	6,818	6,954	7,093	7,235	7,380	7,528	7,678	7,832	7,988	8,148	8,311	8,477	8,647
Gallons Sold	307,750,000	327,713,520	334,267,790	340,953,146	347,772,209	354,727,653	361,822,206	369,058,651	376,439,824	383,968,620	391,647,992	399,480,952	407,470,571	415,619,983
Proposed Rate Increase	**0%**	**12%**	**8%**	**8%**	**8%**	**15%**	**8%**	**5%**	**5%**	**3%**	**0%**	**0%**	**0%**	**0%**
Base Rate	$11.44	$12.81	$12.81	$13.83	$14.94	$17.18	$18.56	$19.49	$19.49	$19.49	$19.49	$19.49	$19.49	$19.49
Rate/1000 Gallons	$4.28	$4.79	$5.17	$5.59	$6.03	$6.94	$7.49	$7.87	$8.26	$8.51	$8.51	$8.51	$8.51	$8.51
Revenues														
Water Sales	$2,004,270	$2,225,912	$2,366,543	$2,606,984	$2,871,853	$3,368,684	$3,710,942	$3,974,419	$4,165,480	$4,320,204	$4,406,608	$4,494,740	$4,584,635	$4,676,327
Sales Tax	$130,264	$123,812	$126,300	$130,349	$143,593	$168,434	$185,547	$198,721	$208,274	$216,010	$220,330	$224,737	$229,232	$233,816
Connection Fees/Permits	$110,565	$82,000	$95,750	$54,500	$54,500	$54,500	$54,500	$45,000	$45,000	$45,000	$45,000	$45,000	$45,000	$45,000
Use of Money	$3,627	$800	$800	$2,000	$2,000	$2,000	$2,000	$2,000	$2,000	$2,000	$2,000	$2,000	$2,000	$2,000
Miscellaneous	$14,248	$1,473	$1,500	$15,700	$15,700	$15,700	$15,700	$15,700	$15,700	$15,700	$15,700	$15,700	$15,700	$15,700
Transfers	$153,364	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
Accounts Receivable/Payable	($165,560)	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
Total Water Utility Revenues	**$2,250,778**	**$2,433,997**	**$2,590,893**	**$2,809,533**	**$3,087,646**	**$3,609,318**	**$3,968,689**	**$4,235,840**	**$4,436,454**	**$4,598,914**	**$4,689,638**	**$4,782,177**	**$4,876,566**	**$4,972,844**
Expenditures														
Budget Inflation Rate		5.54%	5.21%	5.00%	5.00%	15.00%	5.00%	5.00%	5.00%	5.00%	5.00%	5.00%	5.00%	5.00%
Personnel Services	$371,731	$401,198	$398,855	$418,798	$439,738	$505,698	$530,983	$557,532	$585,409	$614,679	$645,413	$677,684	$711,568	$747,147
Services & Commodities	$733,012	$869,654	$955,050	$1,002,803	$1,052,943	$1,210,884	$1,271,428	$1,335,000	$1,401,750	$1,471,837	$1,545,429	$1,622,700	$1,703,835	$1,789,027
Capital	$702	$0	$500	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
Transfers														
Equipment Revolving	$25,000	$0	$12,000	$122,500	$42,500	$127,000	$49,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000
Capital Reserve	$54,500	$0	$50,000	$55,000	$100,000	$125,000	$125,000	$125,000	$125,000	$125,000	$125,000	$125,000	$125,000	$125,000
Debt	$721,885	$725,289	$686,873	$694,543	$687,303	$694,240	$699,766	$703,823	$706,442	$633,444	$443,231	$355,823	$131,150	$131,119
Billing & Accounting	$233,063	$262,287	$272,708	$280,889	$289,316	$297,995	$306,935	$319,213	$331,981	$345,260	$359,071	$373,434	$388,371	$403,906
Upcoming Projects														
(1) Repaint Water Tower #2/Water Main Loop Projects				$143,414	$159,637	$157,324	$154,737	$156,824	$158,392	$159,518	$155,122	$155,432	$155,400	
(2) Phase 1a - Construct New Water Plant						$750,457	$818,560	$818,320	$817,860	$818,180	$818,260	$818,100	$818,700	$818,040
(3) Phase 1b - Construct Water Tower									$301,913	$331,270	$331,830	$331,759	$330,934	$329,202
(4) Phase 1c - Well & Main Improvements												$344,033	$378,994	$378,907
Total Water Utility Expenditures	**$2,139,893**	**$2,258,428**	**$2,375,986**	**$2,717,946**	**$2,771,436**	**$3,868,599**	**$3,956,410**	**$4,065,712**	**$4,478,747**	**$4,549,189**	**$4,473,356**	**$4,853,965**	**$4,793,953**	**$4,772,348**
Net Change in Fund Balance	**$110,885**	**$175,569**	**$214,907**	**$91,586**	**$316,210**	**($259,281)**	**$12,279**	**$170,128**	**($42,293)**	**$49,725**	**$216,282**	**($71,788)**	**$82,614**	**$200,496**
Beginning Fund Balance	$103,289	$214,174	$389,743	$604,650	$696,236	$1,012,446	$753,165	$765,444	$935,572	$893,280	$943,005	$1,159,287	$1,087,498	$1,170,112
Ending Fund Balance	$214,174	$389,743	$604,650	$696,236	$1,012,446	$753,165	$765,444	$935,572	$893,280	$943,005	$1,159,287	$1,087,498	$1,170,112	$1,370,608
% Reserved	10.01%	17.26%	25.45%	25.62%	36.53%	19.47%	19.35%	23.01%	19.94%	20.73%	25.92%	22.40%	24.41%	28.72%
Total Personnel Costs	$371,731	$401,198	$398,855	$418,798	$439,738	$505,698	$530,983	$557,532	$585,409	$614,679	$645,413	$677,684	$711,568	$747,147
% of Water Utility Expenditures	17.37%	17.76%	16.79%	15.41%	15.87%	13.07%	13.42%	13.71%	13.07%	13.51%	14.43%	13.96%	14.84%	15.66%
Debt Service Coverage (Net Revenue/All Debt)														
Actual	1.59	1.60	1.80	1.66	1.88	1.18	1.29	1.40	1.23	1.29	1.43	1.24	1.36	1.47
Required	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
Difference	0.39	0.40	0.60	0.46	0.68	(0.02)	0.09	0.20	0.03	0.09	0.23	0.04	0.16	0.27

Increase on consumption rate only

Updated: June 14, 2013

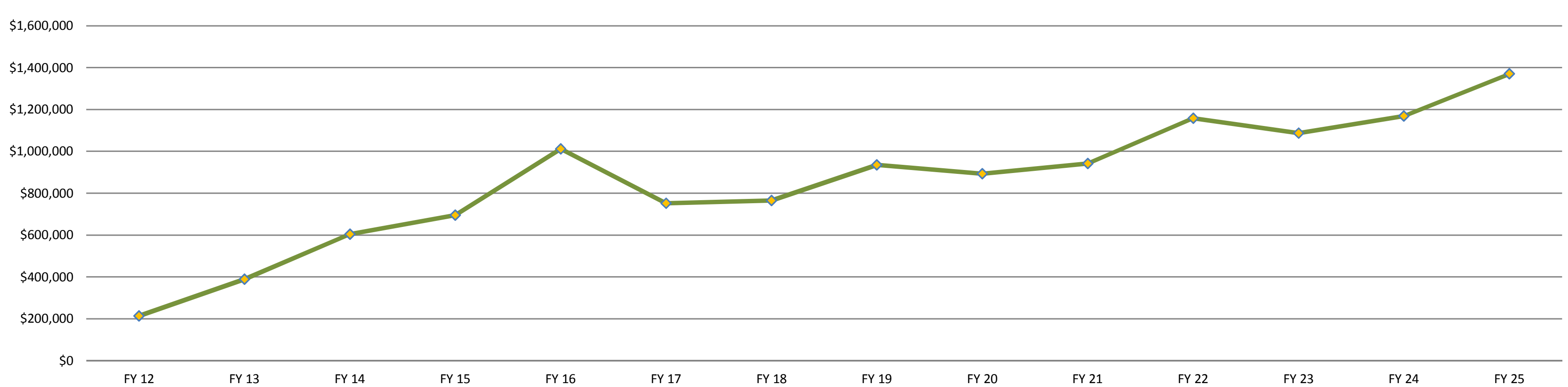
Water Utility, 2013 Facility Plan Improvements; Rate & Budget Projections

Water Rate Increase Analysis

Monthly Water Costs Based on Usage

Consumption in Gallons		FY 12	FY 13	FY 14	FY 15	FY 16	FY 17	FY 18	FY 19	FY 20	FY 21	FY 22	FY 23	FY 24	FY 25
3,000		$20.00	$22.39	$23.16	$25.01	$27.01	$31.06	$33.55	$35.22	$36.01	$36.51	$36.51	$36.51	$36.51	$36.51
5,000		$28.55	$31.97	$33.50	$36.18	$39.08	$44.94	$48.53	$50.96	$52.54	$53.53	$53.53	$53.53	$53.53	$53.53
8,000		$41.39	$46.34	$49.02	$52.94	$57.18	$65.76	$71.02	$74.57	$77.33	$79.06	$79.06	$79.06	$79.06	$79.06
11,000		$54.22	$60.71	$64.54	$69.71	$75.28	$86.57	$93.50	$98.17	$102.11	$104.59	$104.59	$104.59	$104.59	$104.59
15,000		$71.33	$79.87	$85.23	$92.05	$99.42	$114.33	$123.48	$129.65	$135.16	$138.63	$138.63	$138.63	$138.63	$138.63
3,000	Additional Water Cost/Month		$2.39	$0.77	$1.85	$2.00	$4.05	$2.48	$1.68	$0.79	$0.50	$0.00	$0.00	$0.00	$0.00
5,000	Additional Water Cost/Month		$3.42	$1.53	$2.68	$2.89	$5.86	$3.60	$2.43	$1.58	$0.99	$0.00	$0.00	$0.00	$0.00
8,000	Additional Water Cost/Month		$4.95	$2.68	$3.92	$4.24	$8.58	$5.26	$3.55	$2.76	$1.74	$0.00	$0.00	$0.00	$0.00
11,000	Additional Water Cost/Month		$6.49	$3.83	$5.16	$5.58	$11.29	$6.93	$4.67	$3.94	$2.48	$0.00	$0.00	$0.00	$0.00
15,000	Additional Water Cost/Month		$8.54	$5.36	$6.82	$7.36	$14.91	$9.15	$6.17	$5.51	$3.47	$0.00	$0.00	$0.00	$0.00
3,000	Additional Water Cost/Year		$28.73	$9.20	$22.23	$24.01	$48.62	$29.82	$20.13	$9.50	$5.95	$0.00	$0.00	$0.00	$0.00
5,000	Additional Water Cost/Year		$41.02	$18.39	$32.16	$34.74	$70.34	$43.14	$29.12	$18.94	$11.90	$0.00	$0.00	$0.00	$0.00
8,000	Additional Water Cost/Year		$59.45	$32.19	$47.06	$50.83	$102.92	$63.13	$42.61	$33.11	$20.82	$0.00	$0.00	$0.00	$0.00
11,000	Additional Water Cost/Year		$77.88	$45.98	$61.96	$66.92	$135.51	$83.11	$56.10	$47.27	$29.74	$0.00	$0.00	$0.00	$0.00
15,000	Additional Water Cost/Year		$102.46	$64.38	$81.83	$88.37	$178.95	$109.76	$74.09	$66.16	$41.64	$0.00	$0.00	$0.00	$0.00

Fund Balance Projection



-Summary of Projects-

(1) Water Main Loop Project: Extend water main on St. Andrews Drive and on 240th Street in order to loop water system and improve water quality and pressure; replace aging water main on Hickory Street; repaint Water Tower #2 (water tower work scheduled for FY 14, borrow money in FY 15); total cost estimated at $1.135mil.

(2) Phase 1a-Construct New Water Plant: Construct new RO water plant at Maintenance Facility Campus on Front Street as per Facility Plan; total cost estimated at $13.2 million.

(3) Phase 1b-Construct Water Tower: Construct water tower as per Facility Plan; total cost estimated at $2.8 million.

(4) **Phase 1c-Well and Main Improvements:** Construct a new Jordan well and install a new raw water main as per Facility Plan; total cost estimated at $3.2 million.

Updated: June 14, 2013