CITY OF LOWELL

LANE COUNTY, OREGON

WATER SYSTEM MASTER PLAN UPDATE

December 2006

HBH
Consulting
Engineers

City of Lowell
Lane County, Oregon

Water System Master Plan Update

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Table of Contents

Section 1

Executive Summary

1.1 Background

Lowell is located in central Lane County Oregon, about 20 miles southeast of Eugene/Springfield, and is situated along State Highway 58 adjacent to Dexter Lake (see Figure 2.1-1, “Regional Location Map”). The City of Lowell was incorporated in 1954 upon the site of an abandoned town that originally housed workers from the U.S. Army Corps of Engineers for the construction of Dexter and Lookout Point Reservoirs.

The City of Lowell water supply system is classified by the Oregon Department of Human Services (DHS) Drinking Water Program (DWP) as a “community” water system, identified within the public water system inventory by Public Water System (PWS) identification number OR4100492. At present, the DWP database lists the service population for this water system as 1,075 persons with 350 service connections. The city owns, operates, and maintains this water system.

The primary elements of the water system include an intake from Dexter Lake which is located near the covered bridge on the county causeway, a conventional water treatment plant (WTP), a 500,000 gallon primary storage tank, a distribution pump station (at the WTP), a booster-pump station for a 2,500 gallon high-elevation storage tank, and approximately 5½ miles of piping (mostly 6-in asbestos cement pipe).

Anticipated growth in the service population, existing storage deficiency concerns, pumping and filtration rates at near maximums during summer months, and projected shortfall in raw water supply have led to the need for an updated water system master plan. The planning period for this updated plan is 25 years, ending in 2031.

1.2 Population, EDUs, and Growth

Current Population and EDUs

The 2000 U.S. Census report originally listed the population of Lowell as 857 with 315 occupied housing units (342 total), yielding an average of 2.72 people per household. However, that population figure was later revised to 880, yielding an average of 2.79 people per household, which is the value adopted for this updated water master plan.

According to Lowell accounting records, 324 single-family dwellings, 22 apartment units, and 7 mobile homes are currently being serviced by the city water system. An average of 2.79 people per single-family dwelling is utilized. For apartment units and mobile homes, the population densities are 1.53 (apartment duplex units), 3.84 (apartment complex units), and 1.91 (mobile homes).

Based upon water usage records from July 2005 through June 2006, the average water usage per single family dwelling 5,188 gallons per month. The 324 single-family dwellings correspond to 324.0 EDUs. The 22 apartment units correspond to 25.3 EDUs, and the 7 mobile homes correspond to 4.8 EDUs. The total number of residential EDUs is therefore 354.1. With a population density of 2.79 people per EDU, the current residential population serviced by the city water system is estimated to be 988. As described in Section 2.2, about 75 people rely upon private wells. As a result, the current residential population for Lowell is estimated to be 1,063. This estimate is believed to be more reliable than that provided by the

U.S. Census Bureau.

However, for purposes of assessing the future water demands placed upon the system, only the population of 988 will be projected, for reasons described in detail in Section 2.4.

Finally, based upon the same 12-month period of water usage mentioned above, the non-residential water consumers (businesses, industries, and institutions) accounted for an amount of water usage equal to 22.7 single-family dwellings (22.7 EDUs). The total number of EDUs for the system is therefore 376.8. As a result, the current equivalent service population (ESP) is 1,051.

Projected Population and EDUs

Keeping in step with previous County-wide planning efforts, a growth rate of 3.30% was selected by the city to project population growth and establish estimated future water demands.

Table 1.2-1 is provided below which summarizes the existing and projected population and EDU projections for the planning period. Additional information on this subject can be found in Section 2.4.

Table 1.2-1 – Projected Population and EDU-Values (3.30% AAGR)

Year	Residential Population	Residential EDUs	Non-Res. EDUs	Local Grocer EDUs	Total System EDUs	Equiv. Serv. Population
2006	988	354.1	18.7	4.0	376.8	1,051
2007	1,021	365.8	19.3	4.1	389.2	1,086
2008	1,054	377.9	20.0	4.3	402.1	1,122
2009	1,089	390.3	20.6	4.4	415.3	1,159
2010	1,125	403.2	21.3	4.6	429.1	1,197
2011	1,162	416.5	22.0	4.7	443.2	1,237
2012	1,200	430.3	22.7	4.9	457.8	1,277
2013	1,240	444.5	23.5	5.0	472.9	1,320
2014	1,281	459.1	24.2	5.2	488.6	1,363
2015	1,323	474.3	25.0	5.4	504.7	1,408
2016	1,367	489.9	25.9	5.5	521.3	1,455
2017	1,412	506.1	26.7	5.7	538.5	1,503
2018	1,459	522.8	27.6	5.9	556.3	1,552
2019	1,507	540.0	28.5	6.1	574.7	1,603
2020	1,557	557.9	29.5	6.3	593.6	1,656
2021	1,608	576.3	30.4	6.5	613.2	1,711
2022	1,661	595.3	31.4	6.7	633.5	1,767
2023	1,716	614.9	32.5	6.9	654.4	1,826
2024	1,772	635.2	33.5	7.2	676.0	1,886
2025	1,831	656.2	34.7	7.4	698.3	1,948
2026	1,891	677.8	35.8	7.7	721.3	2,012
2027	1,954	700.2	37.0	7.9	745.1	2,079
2028	2,018	723.3	38.2	8.2	769.7	2,147
2029	2,085	747.2	39.5	8.4	795.1	2,218
2030	2,154	771.9	40.8	8.7	821.3	2,292
2031	2,225	797.3	42.1	9.0	848.4	2,367

1.3 Existing Water System

The scope of work for this Master Plan Update is to study the city's water supplies, treatment system, and storage reservoirs. Detailed information on each of these parts of the city's system is provided in Section 4 of this plan. A brief summary of the three major components is provided below.

Existing Water Supplies

Water supplies are defined as the city's available raw water resources or water rights. The city holds water rights on both Dexter Reservoir and several groundwater rights for wells located in different parts of the community.

Due to water quality problems, the city's wells have not been utilized for some time. Therefore, the city's principal water rights are those available to them from Dexter Reservoir.

A summary of the city's existing water rights is provided below in Table 1.3-1.

Table 1.3-1 – Existing Water Rights – City of Lowell

Source Type	Applic. No.	Permit No.	Certif. No.	Max. Flow Rate (cfs)	Priority Date
Ground (Well #1)	G05520	G05408	46884	0.45	05/19/1971
Ground (Well #2)	G08999	G08386	Not Issued	0.45	11/06/1978
Ground (Well #3)	G14204	G13499	Not Issued	0.45	11/20/1995
Surface (Dexter Lake)	S30077	S23705	23721	1.00	06/20/1955

Source: Oregon Water Resources Department – Ground and Surface Water Rights Records

Note: The water right to Well #2 was cancelled, effective 03/01/1983, as recorded by Special Order of the Oregon WRD Director (volume 37, pages 85–87).

Water Treatment Facilities

The existing treatment plant was originally constructed around 1969 though its use was intermittent for several decades due to problems associated with taste and odor in the surface water and the fact that the city could fall back on the use of their well sources.

In the 1990s, increases in water demand and decreases in well water quality resulted in discussions to revitalize the original surface water treatment plant. Efforts to obtain funding and upgrade the plant culminated in the 2001 water treatment plant upgrades. The existing treatment facilities are the results of this latest upgrade effort.

The existing water treatment facilities utilize the following major processes:

- Chemical coagulation
- Rapid mixing
- Flocculation
- Solids-contact clarification
- Dual media filtration
- Disinfection

The configuration and programming of the existing plant allow a production rate of around 160 gpm (230,400 gpd over 24 hours).

While the plant is generally in good condition, it is not currently capable of producing the volume of water required to meet the current demands of the community without extremely long run times. It is only through the exceptional efforts of the operations staff that the plant is able to provide the amount of water it does during peak demand conditions.

Distribution System

Review and analysis of the distribution system was beyond the scope of this project. However, some information and a basic system map is provided in Section 4.5 of the plan.

Finished Water Storage System

The city currently utilizes a single concrete reservoir for the storage of treated potable water. The reservoir was constructed in 1992 with a high water surface elevation of approximately 953 feet. The tank is about 32 feet tall and has a diameter of 54 feet. The reinforced concrete walls have a thickness of about 16 inches.

For more information on the existing reservoir, see Section 4.5 of this plan.

1.4 Water Demands

Section 5 of the plan provides analysis and a summary of the existing and projected water demands for the City of Lowell. Of particular interest is the following three water demand parameters:

- Average Annual Demand (AAD) – corresponds to the average water demand over a full year.

- Maximum Monthly Demand (MMD) – corresponds to the maximum water demand month. Describes the maximum water usage over the summer on a monthly basis. Useful to determine the level of performance and capacity that the plant must be able to sustain for a relatively long period of time (a full month).

- Maximum Daily Demand (MDD) – corresponds to the maximum demand experienced in one day in a given year. This high level of demand typically corresponds to a holiday, festival, or just a day when water usage patterns create the highest demand for water production in a year. This quantity is important for determining water rights requirements, treatment capacity requirements, and storage requirements.

Several years of production and consumption data were analyzed to quantify the existing water demand characteristics for the system. When determined, those demand characteristics were utilized to calculate the per capita water demands. Once the per capita water demand values were quantified, projected water demands were developed based on the population growth rates established in Section 2 of the plan.

Table 1.4-1 below summarizes the current and projected water demand criteria for the City of Lowell for the planning period.

Table 1.4-1 – Existing and Projected Water Demand

Year	Residential Population	Total System EDUs	ADD (gpd)	MMD (gpd)	MDD (gpd)
2006	988	376.8	112,000	173,600	308,000
2007	1,021	389.2	115,696	179,329	318,164
2008	1,054	402.1	119,514	185,247	328,663
2009	1,089	415.3	123,458	191,360	339,509
2010	1,125	429.1	127,532	197,675	350,713
2011	1,162	443.2	131,741	204,198	362,287
2012	1,200	457.8	136,088	210,936	374,242
2013	1,240	472.9	140,579	217,897	386,592
2014	1,281	488.6	145,218	225,088	399,350
2015	1,323	504.7	150,010	232,516	412,528
2016	1,367	521.3	154,961	240,189	426,142
2017	1,412	538.5	160,074	248,115	440,204
2018	1,459	556.3	165,357	256,303	454,731
2019	1,507	574.7	170,814	264,761	469,737
2020	1,557	593.6	176,450	273,498	485,238
2021	1,608	613.2	182,273	282,523	501,251
2022	1,661	633.5	188,288	291,847	517,793
2023	1,716	654.4	194,502	301,478	534,880
2024	1,772	676.0	200,920	311,426	552,531
2025	1,831	698.3	207,551	321,704	570,764
2026	1,891	721.3	214,400	332,320	589,600
2027	1,954	745.1	221,475	343,286	609,056
2028	2,018	769.7	228,784	354,615	629,155
2029	2,085	795.1	236,334	366,317	649,917
2030	2,154	821.3	244,133	378,405	671,365
2031	2,225	848.4	252,189	390,893	693,520

1.5 Alternatives

Based on the design criteria established in Section 6 of the plan, several alternatives were considered for the areas of study undertaken in this Master Plan Update. These areas include water supplies, treatment, and finished water storage. Detailed information on the alternatives considered is provided in Section 7 of this plan.

A brief summary of the various alternatives is provided below:

Water Supply Alternatives

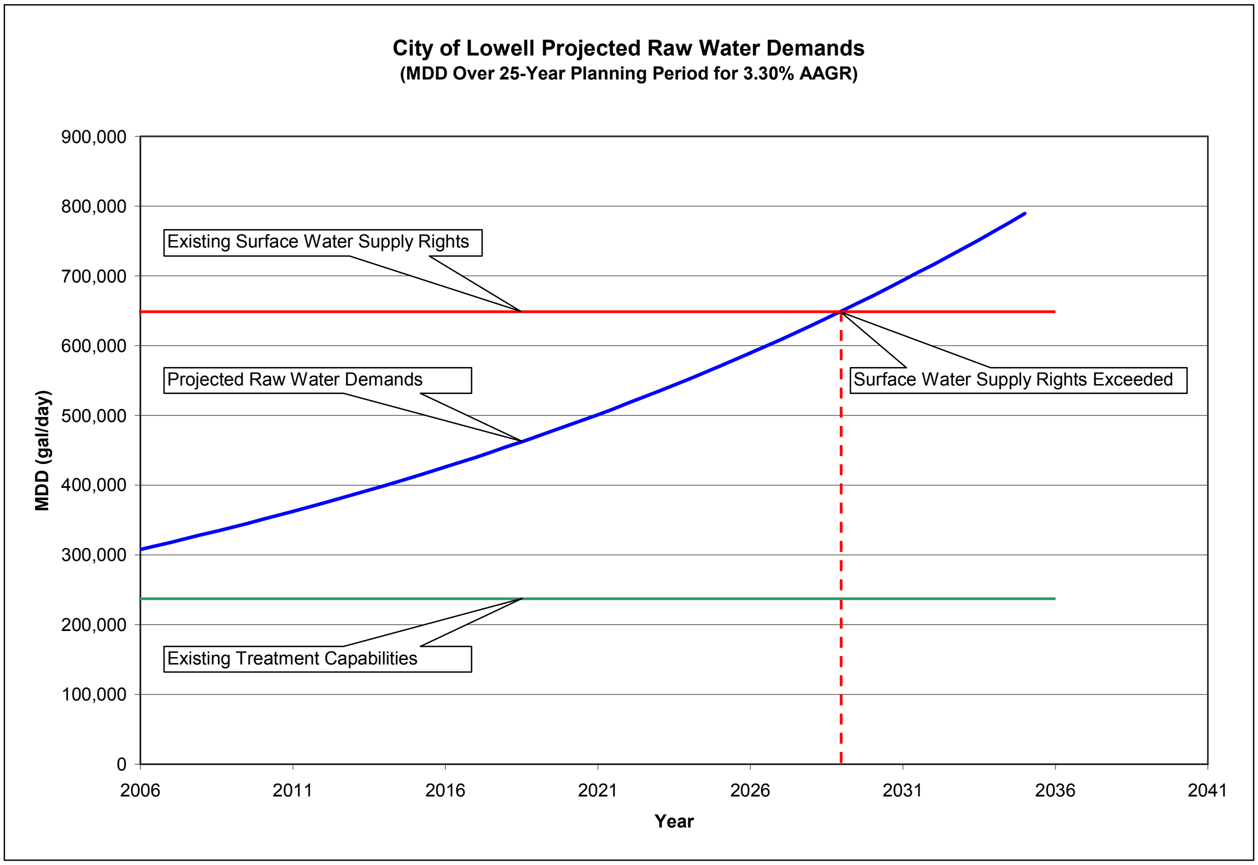
The two main alternatives for the city to consider with regard to water supplies include surface and groundwater alternatives.

The existing wells, while providing several years of good service, have degraded so that they present both quality and quantity (capacity) problems. Use of the existing wells would result in a relatively low sustained output (estimated at less than 100 gpm combined) and would require expensive treatment to remove arsenic from the well water.

According to water demand projections, the city will require additional surface water rights before the end of the planning period. Figure 1.5-1 below shows the projected water demands and the available surface water rights. It is estimated that the city will run out of surface water supplies by the year 2029. Therefore, steps must be taken to acquire additional water rights before that time.

The U.S. Army Corps of Engineers was contacted to discuss the potential for providing the city with additional water from Dexter Reservoir. While the COE would not commit, they did say that there is a process for the city to follow that would likely result in additional water rights being provided to the city.

Figure 1.5-1 – Projected Water Demands and Available Surface Water Rights



While the cost of obtaining additional water rights is unknown, the city should enter into discussions with the COE and begin the process of obtaining additional water rights from Dexter Reservoir. Additional information on this issue is available in Section 7.2 of this plan.

Water Treatment Facilities Alternatives

In Section 6, it was established that the city should develop treatment facilities capable of producing the projected MDD within a 22-hour operating period. This flow rate was determined to be around 525 gpm or 757,000 gpd.

While many alternatives were considered, three complete treatment alternatives were developed as the most appropriate alternatives for the city to consider for the expansion of their water treatment capabilities. A summary of the three complete treatment alternatives is provided below. See Section 7.3 for detailed descriptions and discussions of these alternatives.

Alternative 1: Conventional Treatment Expansion. Within the first alternative, the city would expand the capacity of the existing plant in order to meet the projected demands for the 25-year planning period. This will be accomplished through the construction of the following major components:

- New raw and finished water pumping systems, valves, fittings, chemical injection, and rapid mixing.
- New concrete sedimentation basin.

- Construction of third filter bay and upgrading process to allow simultaneous operation of all three filters.
- Piping, valve, meter, and fitting upgrades as required for a complete system.
- Electrical and control upgrades.
- On-site chlorine generation equipment.
- New concrete clearwell to provide for additional chlorine contact time.

Table 1.5-1 below provides a detailed preliminary cost estimate for Alternative 1.

Table 1.5-1 – Alternative 1 Cost Estimate- Conventional Filtration

Alternative 1: Complete Treatment Alternative - Conventional Filtration					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$100,000.00	$100,000.00
2	Construction Facilities/Temporary Systems	ls	1	$75,000.00	$75,000.00
3	Addition of third filter	ls	1	$15,000.00	$15,000.00
4	Conversion of plant to air scour backwash & blower	ls	1	$50,000.00	$50,000.00
5	Construction of conventional sedimentation basin	ls	1	$535,000.00	$535,000.00
6	Piping Improvements in plant and on site	ls	1	$80,000.00	$80,000.00
7	Controls & Instrumentation upgrades	ls	1	$50,000.00	$50,000.00
8	Installation of a GAC Cap in filters	ls	1	$5,000.00	$5,000.00
9	Construction of a new concrete clearwell	ls	1	$198,250.00	$198,250.00
10	New raw water & finished water pumping upgrades	ls	1	$35,000.00	$35,000.00
11	On-site chlorine disinfection system	ls	1	$38,200.00	$38,200.00
12	Flow metering and valve upgrades	ls	1	$50,000.00	$50,000.00
13	Electrical Improvements	ls	1	$100,000.00	$100,000.00
14	Small building over new clearwell for misc. equipment	lf	500	$150.00	$75,000.00
		Construction Total			$1,406,450.00
		Contingency (20%)			$281,290.00
		Subtotal			$1,687,740.00
		Engineering (18%)			$303,793.20
		Administrative costs (3%)			$50,632.20
		Total Project Costs			**$2,042,165.40**

<u>The advantages of Alternative 1 include:</u>

- Similar treatment technology and process to the existing plant.
- Makes use of most of the existing facilities.
- Reliable conventional technology.

<u>The disadvantages of Alternative 1 include:</u>

- Older technology. Not the best available technology.

- Large concrete tank results in higher construction costs. Highest cost alternative.
- Many of the existing system components are too small and must be replaced.

Alternative 2: Membrane Treatment. Within the second alternative, the city would utilize much of the existing treatment facilities but replace the existing filter process with a membrane treatment system. Under the second alternative, the following major improvements would be required:

- New raw and finished water pumping systems, valves, fittings, chemical injection, and rapid mixing.
- Use the existing clarifier for higher flow rate quiescent zone.
- Construct a packaged membrane treatment system. The system would be provided in a skid-mounted configuration for ease of installation.
- Piping, valve, meter, and fitting upgrades as required for a complete system.
- Electrical and control upgrades.
- On-site chlorine generation equipment.
- New concrete clearwell to provide for additional chlorine contact time.

Table 1.5-2 below provides the preliminary cost estimate for Alternative 2.

Table 1.5-2 – Alternative 2 Cost Estimate- Membrane Treatment

Alternatives No. 2 - Membrane Treatment w/ GAC Pressure Filters					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$75,000.00	$75,000.00
2	Construction Facilities/Temporary Systems	ls	1	$50,000.00	$50,000.00
3	Constructing of new concrete clearwell	ls	1	$198,250.00	$198,250.00
4	New Building to house membrane and other equipment	sf	1000	$150.00	$150,000.00
5	New raw and finished water pumping equipment	ls	1	$35,000.00	$35,000.00
6	New pre-filtration equipment (Amiad or similar)	ls	1	$40,000.00	$40,000.00
7	Membrane packaged treatment equipment	ls	1	$500,000.00	$500,000.00
8	Piping improvements in plant and on site	ls	1	$60,000.00	$60,000.00
9	Controls and instrumentation upgrades	ls	1	$35,000.00	$35,000.00
10	GAC pressure filters after membrane	ls	1	$40,000.00	$40,000.00
11	Onsite chlorine generation equipment	ls	1	$38,200.00	$38,200.00
12	Flow metering and valve upgrades	ls	1	$20,000.00	$20,000.00
13	Electrical improvements	ls	1	$100,000.00	$100,000.00
		Construction Total			$1,341,450.00
		Contingency (20%)			$268,290.00
		Subtotal			$1,609,740.00
		Engineering (18%)			$289,753.20
		Administrative costs (3%)			$48,292.20
		Total Project Costs			**$1,947,785.40**

The advantages of Alternative 2 include:

- Best available technology would be utilized for this long-term upgrade.
- Makes use of much of the existing facilities.
- Ease of operation and higher level of treatment security due to physical membrane barrier.

The disadvantages of Alternative 2 include:

- Discontinued use of the existing conventional filters.
- Relatively high construction costs, though slightly less than the conventional option.

Alternative 3: Interim Improvements. In the third alternative, the city would make minor improvements in comparison to the first two alternatives with the goal of increasing plant output in order to extend the life of the plant and postpone a major improvement for many years. This alternative, however, will not provide the capacity required for the demands that are projected for the planning period.

Improvements included within the Alternative 3 project include:

- New raw and finished water pumping systems.
- On site chlorine generation equipment.
- Construction of the third filter along with piping, programming, and fittings changes as required to operate all three filters simultaneously.
- New baffling and other improvements to increase contact time in the clearwell.
- Electrical and control improvements.

Table 1.5-3 below summarizes the preliminary cost estimate for the Alternative 3 project.

Table 1.5-3 – Alternative 3 Cost Estimate – Interim Improvements

Alternative No. 3 - Interim Treatment Measures					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$35,000.00	$35,000.00
2	Construction Facilities/Temporary Systems	ls	1	$20,000.00	$20,000.00
3	Construction of third filter w/ surface wash	ls	1	$20,000.00	$20,000.00
4	Addition of GAC cap	ls	1	$5,000.00	$5,000.00
5	On-site chlorine generation equipment	ls	1	$38,200.00	$38,200.00
6	Clearwell upgrades	ls	1	$30,000.00	$30,000.00
7	Piping, valve, actuator, and metering improvements	ls	1	$60,000.00	$60,000.00
8	Controls & Instrumentation upgrades	ls	1	$40,000.00	$40,000.00
9	Electrical upgrades	ls	1	$35,000.00	$35,000.00
10	New raw and finished water pumping & related equip.	ls	1	$35,000.00	$35,000.00
		Construction Total			$318,200.00
		Contingency (20%)			$63,640.00
		Subtotal			$381,840.00
		Engineering (18%)			$68,731.20
		Administrative costs (3%)			$11,455.20
		Total Project Costs			**$462,026.40**

<u>The advantages of Alternative 3 include:</u>

- Lower cost option.
- Maximize the use of existing facilities without building new facilities.
- Operation of system to remain essentially the same as the existing operation.

<u>The disadvantages of Alternative 3 include:</u>

- Uncertain performance of the solids contact clarifier at higher flows.
- This alternative will not provide adequate capacity for the entire planning period. Another major upgrade will be required in the future. As prices increase, the future upgrade will cost more.
- Some existing operational challenges will persist.

Alternative 4: Packaged Conventional Treatment. Much like Alternative 2, this fourth and final alternative, would also utilize much of the existing treatment facilities but replace the existing filter process with a packaged conventional treatment system rather than a membrane process. Under the the fourth alternative, the following major improvements would be required:

- New raw and finished water pumping systems, valves, fittings, chemical injection, and rapid mixing.
- Use the existing clarifier for higher flow rate quiescent zone.
- Construct a packaged conventional treatment system. The system would be provided in a skid-

mounted configuration for ease of installation.

- Piping, valve, meter, and fitting upgrades as required for a complete system.
- Electrical and control upgrades.
- On-site chlorine generation equipment.
- New concrete clearwell to provide for additional chlorine contact time.

Table 1.5-4 below provides the preliminary cost estimate for Alternative 4.

Table 1.5-4 – Alternative 4 Cost Estimate – Packaged Conventional Treatment

Alternatives No. 4 - Packaged Conventional Treatment Process					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$80,000.00	$80,000.00
2	Construction Facilities/Temporary Systems	ls	1	$55,000.00	$55,000.00
3	Constructing of new concrete clearwell	ls	1	$198,250.00	$198,250.00
4	New Building to house new treatment equipment	sf	1200	$150.00	$180,000.00
5	New raw and finished water pumping equipment	ls	1	$35,000.00	$35,000.00
6	New pre-filtration equipment (Amiad or similar)	ls	1	$40,000.00	$40,000.00
7	Conventional Packaged Treatment Equipment	ls	1	$400,000.00	$400,000.00
8	Piping improvements in plant and on site	ls	1	$60,000.00	$60,000.00
9	Controls and instrumentation upgrades	ls	1	$35,000.00	$35,000.00
10	GAC pressure filters for taste and odor	ls	1	$40,000.00	$40,000.00
11	Onsite chlorine generation equipment	ls	1	$38,200.00	$38,200.00
12	Flow metering and valve upgrades	ls	1	$20,000.00	$20,000.00
13	Electrical improvements	ls	1	$100,000.00	$100,000.00
		Construction Total			$1,281,450.00
		Contingency (20%)			$256,290.00
		Subtotal			$1,537,740.00
		Engineering (18%)			$276,793.20
		Administrative costs (3%)			$46,132.20
		Total Project Costs			**$1,860,665.40**

<u>The advantages of Alternative 4 include:</u>

- Makes use of much of the existing facilities.
- Familiar technology to existing operations staff.

<u>The disadvantages of Alternative 4 include:</u>

- Discontinued use of the existing conventional filters.
- Relatively high construction costs, though slightly less than the other alternatives.
- Not the best available technology (application of older technology)

Alternative No. 5: Do Nothing. If the City elects to do nothing, they will continue to struggle with the issue of being able to make an adequate amount of water for their growing community. If the City elects to do nothing to upgrade their treatment capabilities, they may consider the following measures to mitigate the deficiency:

1. Declare a development moratorium to freeze growth in the community. This will also require a specific plan and approach be prepared for DLCD to describe the steps the community will take to lift the moratorium within a short period of time (typically 6 months). A moratorium cannot be indefinite as a City cannot prevent land developers from making a living within their land assets located within the community.

2. Develop a strict and regimented water curtailment program to reduce the peak water use days during the summer months. This may include special water rates for high use during peak periods, various voluntary conservation and curtailment measures to reduce water consumption, mandatory conservation and curtailment laws complete with penalties for non-compliance, water reuse opportunities, and various other measures designed to reduce overall water use, especially during critical seasons.

3. While not necessarily a do-nothing approach, construction of more treated water reserves would help offset the affect of inadequate treatment plant production.

A do nothing approach is not recommended at this time as the city is growing and must make some accommodations for their increasing water needs.

Treated Water Storage Alternatives

In Section 6.2, it was determined that the city would require a total reserve capacity of around 1.23 million gallons. A summary of this calculation is provided below in Table 1.5-5.

Table 1.5-5 – Treated Water Reserves – Projected Need Calculation

Reserve Type	Description	Reserve Amount (gal)
System Equalization	0.25 × 25-Year MDD	173,380
Emergency Protection	1.00 × 25-Year MDD	693,520
Fire Suppression	2,000 gpm for 3 Hours	360,000
Total		1,226,900

With the existing 500,000 gallon reservoir, the city must add another 727,000 gallons of storage to the system to meet the projected treated water reserve requirements.

In a previous reservoir siting study effort, the city selected a location for a new reservoir known as Site C (see Figure 3.3 from the Siting Study reproduced on page 28 of Section 7 of this plan). While the scope of work for this plan did not include a siting study, alternative locations and sizing for the city's reservoir needs were considered. A summary of this analysis is provided below.

Large reservoir at Site C (upper pressure level). A large reservoir located at Site C presents several problems. Mainly, Site C is located in the upper pressure level and would provide service to this upper level where currently there is only a small number of customers. This situation would require increased pumping costs to raise the water to the new reservoir yet necessitate construction of additional facilities to reduce the water pressure for servicing the lower and main pressure levels. This approach is not efficient.

Large reservoir in lower pressure level (Seneca property in northwest part of city) and a smaller reservoir at Site C. A preferred option is to locate a larger reservoir in the lower or main pressure level and site a smaller reservoir at Site C to service development as it is required. The existing pumping effort could lift water into the new reservoir which should be located at the same elevation as the existing reservoir with a second pumping station at that reservoir to lift water to the Site C reservoir. This option would reduce the costs of lifting water to the upper level only to have it flow back into the lower level as well as eliminate the need for high-maintenance pressure reducing systems.

Reservoirs can be constructed of concrete, steel, or variations of coated steel. For this analysis, cost estimates were developed for concrete and glass-fused-to-steel (GFS) reservoir options. Concrete reservoirs are believed to typically provide longer life, though GFS reservoirs, if properly maintained, can provide a long and reliable service life.

As concrete reservoirs are generally much more expensive, this analysis focused on the use of GFS reservoirs, though costs for both are provided in Section 7.4 of this plan.

Tables 1.5-6 and 1.5-7 below summarize the estimated project costs for the construction of a 550,000-gallon reservoir in the lower or main pressure level and a 180,000-gallon reservoir to be constructed in the upper pressure level at Site C. It should be noted that neither of these cost estimates include specific land acquisition costs. The city must either negotiate the transfer of land from developers or allocate more funds for the purchase of property needed for the construction of the new reservoirs.

It also should be noted that reservoir improvements can be phased by constructing either one of the tanks as soon as possible and then adding the other tank later in the planning period. The decision of which tank should be built first greatly depends upon the development pressures in the system and whether the upper pressure level requires additional reserves sooner or later.

Table 1.5-6 – Cost Estimate for GFS Reservoir in Lower Pressure Level (550,000-gallon)

Glass Fused to Steel Reservoir - 550,000 gal					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$60,000.00	$60,000.00
2	Construction Facilities/Temporary Systems	ls	1	$35,000.00	$35,000.00
3	Site Preparation & Excavation	ls	1	$50,000.00	$50,000.00
4	Site Piping & Appurtenances	ls	1	$25,000.00	$25,000.00
5	Glass-Fused-to-Steel Reservoir (0.55 MG)	ls	1	$350,000.00	$350,000.00
6	Fencing	ls	1	$15,000.00	$15,000.00
7	Telemetry	ls	1	$10,000.00	$10,000.00
8	10-inch piping to reservoir	lf	2400	$65.00	$156,000.00
9	Roadway & site improvements (crushed rock)	ls	1	$15,000.00	$15,000.00
		Construction Total			$716,000.00
		Contingency (20%)			$143,200.00
		Subtotal			$859,200.00
		Engineering (18%)			$154,656.00
		Administrative costs (3%)			$25,776.00
		Total Project Costs			**$1,039,632.00**

Table 1.5-7 – Cost Estimate for GFS Reservoir in Upper Pressure Zone (180,000-gallon)

Glass Fused to Steel Reservoir & Pump Station - 180,000 gal					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$45,000.00	$45,000.00
2	Construction Facilities/Temporary Systems	ls	1	$25,000.00	$25,000.00
3	Site Preparation & Excavation	ls	1	$30,000.00	$30,000.00
4	Site Piping & Appurtenances	ls	1	$20,000.00	$20,000.00
5	Glass-Fused-to-Steel Reservoir (0.18 MG)	ls	1	$175,000.00	$175,000.00
6	Fencing	ls	1	$15,000.00	$15,000.00
7	Telemetry	ls	1	$7,500.00	$7,500.00
8	10-inch piping to reservoir	lf	1900	$65.00	$123,500.00
9	Booster Pump Station, Electrical & Appurtenances	ls	1	$100,000.00	$100,000.00
10	Roadway & site improvements (crushed rock)	ls	1	$10,000.00	$10,000.00
		Construction Total			$551,000.00
		Contingency (20%)			$110,200.00
		Subtotal			$661,200.00
		Engineering (18%)			$119,016.00
		Administrative costs (3%)			$19,836.00
		Total Project Costs			**$800,052.00**

In addition to developing alternatives for constructing new reservoirs, considerations were made for the rehabilitation of the existing concrete reservoir. The existing reservoir, which was constructed in 1992, exhibits leakage and wear due to improper design and/or construction issues. Several of the “cold joints” in the reservoir exhibit active leakage. Over the years, the leakage has stained the surface of the reservoir and encouraged the growth of algae and other debris on the exterior of the reservoir.

New techniques for sealing the leaks in concrete reservoirs and rehabilitating the surfaces of the reservoir have proven to be effective on similar projects. Section 7.4 includes photos of a similar reservoir in Rockaway Beach, Oregon which was rehabilitated using special injection and coating techniques.

Table 1.5-8 below provides a preliminary budget estimate for the rehabilitation of the existing reservoir. This project is best undertaken once the new 550,000-gallon reservoir has been constructed so that the existing reservoir can be taken off-line for repair.

Table 1.5-8 – Cost Estimate for Rehabilitation of the Existing Reservoir

Existing Reservoir Rehabilitation					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$5,500.00	$5,500.00
2	Construction Facilities/Temporary Systems	ls	1	$8,000.00	$8,000.00
3	Surface Preparation	ls	1	$5,000.00	$5,000.00
4	Foam Injection	lf	500	$65.00	$32,500.00
5	Epoxy Injection	lf	100	$65.00	$6,500.00
6	Exterior Cleaning & Coating	sf	5440	$8.50	$46,240.00
		Construction Total			$103,740.00
		Contingency (20%)			$20,748.00
		Subtotal			$124,488.00
		Engineering (18%)			$22,407.84
		Administrative costs (3%)			$3,734.64
		Total Project Costs			**$150,630.48**

1.6 Recommendations

Section 8 of this plan summarizes the recommendations for the city based on the alternatives discussed in Section 7. The recommendations for each system component analyzed within this plan follows:

Water Supply Plan

The City of Lowell will have adequate surface water supplies for the majority of the planning period. However, water demand projections suggest that additional water rights will eventually be required. According to the U.S. Army Corps of Engineers (COE), a process exists whereby additional water rights can be obtained on Dexter Reservoir. The COE was careful to not commit but suggested that the city begin the process of requesting these additional rights. Additional information on this process and the contact at the COE is available in Section 7.2 of this plan.

While it is unknown what the final cost of obtaining the water rights will be, it is recommended that the city budget around $100,000 for this effort. Additional funds may be required once the city progresses far enough into the "water supply reallocation process" to determine the final costs.

Water Treatment Facilities

The interim water treatment improvements will increase the capacity of the treatment facilities at a significantly lower cost. However, the capacity of the plant will not be adequate for the planning period, or perhaps not even 10 years of the planning period. Therefore, it is not the recommended alternative for the city to develop long term water treatment capabilities.

Alternative 2, which utilizes membrane treatment technology, is the preferred and recommended alternative for this planning effort. This is due to the fact that it has a lower estimated project cost than the conventional alternative (Alternative No. 1), it makes use out of much of the existing facilities, and it provides the best available technology to the city for a long term improvement.

The estimated project cost for the Alternative 2 complete treatment improvements is $1,947,785.

Treated Water Storage Facilities

This plan recommends that two reservoirs be constructed in the system to satisfy future treated water storage requirements. Specifically, it is recommended that a single 550,000-gallon reservoir be constructed within the lower or main pressure level and a second 180,000-gallon reservoir be constructed in the upper pressure level on Site C as developed in the city's previous siting study. The cost for the larger reservoir is estimated at $1,039,632 while the smaller reservoir is estimated at $800,052, including a booster-pump station to fill the smaller tank.

It is recommended that the larger tank be constructed first and the smaller tank in the upper pressure level be constructed when development pressures in the upper pressure level require it.

In addition to constructing the new reservoirs, it is recommended that the existing reservoir be rehabilitated when money is available and when the new 550,000-gallon reservoir is online so that the old reservoir can be taken offline. The estimated project cost to rehabilitate the existing reservoir is $150,630.

A summary of all recommended project costs is provided below in Table 1.6-1. This table makes up what amounts to the city's Capital Improvement Plan (CIP) for the water system.

Table 1.6-1 – City of Lowell Capital Improvement Plan

Project No.	Project Name and Description	Project Cost
1	Acquisition of 1.0 cfs Surface Water Rights	$100,000
2	Water Treatment Facilities Upgrades (based on membrane alternative)	$1,947,785
3A	New 550,000-gallon Reservoir Project	$1,039,632
3B	New 180,000-gallon Reservoir & Pump Station Project	$800,052
3C	Rehabilitation of Existing Reservoir	$150,630
	Total Project Costs	**$4,038,100**

1.7 Financing Strategy

The projects presented above in the CIP were grouped into three separate priority categories. The total project costs for each priority are summarizes as follows:

- Priority 1 – $2,987,417
- Priority 2 – $250,630
- Priority 3 – $800,052

The priority ratings are defined as follows:

- Priority 1 – Priority 1 projects should be undertaken immediately and as soon as the city has available funding. Priority 1 projects will correct existing deficiencies and provide capacity for the planning period. In this plan, projects 2 and 3A should be classified as Priority 1 projects.

- Priority 2 – Priority 2 projects should be undertaken when funding becomes available, but are not necessarily considered critical to address existing deficiencies. Priority 2 projects include important maintenance projects. In this plan, projects 1 and 3C should be considered as Priority 2 projects.

- Priority 3 – Priority 3 projects should be undertaken based on need and development pressures. These projects should be considered optional until development pressures require the project to be undertaken. In this plan, project 3B should be considered as a Priority 3 project.

For the calculations indicated below, it is assumed that the entire cost for each improvement project will be funded with a single 25-year loan at 4% interest so as to separately reveal the rate hike associated with each project. However, the rate hike is based upon the current system EDU-value (377 EDUs) in order to meet the payment schedule during the early stages of amortization.

If the Priority 1 improvements are adopted (project cost: $2,987,417), then an immediate increase in water system revenue of $15,769 per month is required, or about $41.83 per EDU.

If the Priority 2 improvements are adopted (project cost: $250,630), then an immediate increase in water system revenue of $1,323 per month is required, or about $3.51 per EDU.

If the Priority 3 improvements are adopted (project cost: $800,052), then an immediate increase in water system revenue of $4,223 per month is required, or about $11.20 per EDU.

Each priority category represents a significant investment in the city's water system. Therefore, the city must seek to obtain adequate funding to undertake the improvement projects. Various funding resources were discussed in Section 9 of this plan. A brief summary of each is provided below.

Systems Development Charges (SDCs)

SDCs are utilized to collect funds from development and growth resources in order to offset the cost of developing infrastructure that is capable of supporting growth and development. While a complete SDC methodology would be appropriate for the City of Lowell, an effort was made to quantify an appropriate SDC based upon the recommended improvements in this plan. A summary of the SDC calculation is provided below in Table 1.7-1. Additional information on SDCs can be found in Section 9.2 of this plan.

Table 1.7-1 – Potential SDC Calculation Summary

Project No.	Project Name and Description	Project Cost	Percent SDC Eligible	SDC Eligible Costs
1	Acquisition of 1.0 cfs Surface Water Rights	$100,000	93%	$93,000
2	Water Treatment Facilities Upgrades	$1,947,785	55%	$1,071,282
3A	New 550,000-gallon Reservoir Project	$1,039,632	55%	$571,798
3B	New 180,000-gallon Reservoir & Pump Station Project	$800,052	100%	$800,052
3C	Rehabilitation of Existing Reservoir	$150,630	0%	$0
	Total Project Costs	**$4,038,100**		**$2,536,132**
		Existing EDUs		377
		Future EDUs		848
		EDUs added over planning period		471
		Estimated SDC Charge per EDU		**$5,384.57**

Grants and Loans and Other Funding Sources

Several grant and loan programs are available through state and federal sources. Each program has specific requirements for qualifying which the city must satisfy. Many of the requirements have to do with local financial conditions, mean household income, disadvantaged communities, and other requirements. In many cases, preference is given to small communities.

A summary of many of the grant and loan programs is provided in Section 9.3 of this plan.

In addition to grants and loans, the city may consider various local funding alternatives such as bonds, taxes, improvement districts, and others.

Recommended Financing Plan

Based on the recommendations and information provided in this plan, this section is intended to provide a general financing plan for the city to follow in order to pursue and obtain the necessary funding to undertake the selected improvement project(s).

The recommended financing plan is as follows:

1. Immediately update the city's SDC methodology and assessment to reflect the new CIP. Begin collecting SDC funds that can be contributed to the project.

2. Schedule a one-stop meeting where all potential and participating funding agencies can attend to discuss and potentially offer funding packages for the city's improvement projects.

3. Begin the process of raising user rates in anticipation of a new loan. It is not necessary to make a very large increase, though the city should consider the likelihood of the funding package they

will receive and develop a schedule of rate increases that can be implemented over a couple of years.

4. When the project costs and funding package become clear, the city should raise rates as required to meet their obligations for the improvements.

Study Area & Population

Section 2

2.1 Location and Area Description

Lowell is located in central Lane County Oregon, about 20 miles southeast of Eugene/Springfield, and is situated along State Highway 58 adjacent to Dexter Lake (see Figure 2.1-1, "Regional Location Map"). The City of Lowell was incorporated in 1954 upon the site of an abandoned town that originally housed workers from the U.S. Army Corps of Engineers for the construction of Dexter and Lookout Point Reservoirs.

The city limits and urban growth boundary (UGB) for Lowell are virtually identical at present, with an area of approximately 762 acres (1.19 square miles), of which about 286 acres (38%) are undeveloped.

The city lies in Township 19 South, Range 01 West, W.M. The City of Lowell and its UGB are depicted in Figure 2.1-2, "Service Area Map". This study is limited in scope to this service area.

2.2 Historical and Existing Population

The U.S. Census Bureau collects and reports data on population and demographics every decade. In the year 2000, the City of Lowell was reported to have a population of 880 and a total of 342 housing units, of which 315 units were actually occupied, yielding an average of 2.79 persons per occupied household. Population data from earlier decennial reports is provided in Table 2.2-1.

Table 2.2-1 – County and City Historical Population

Year	Lane County Population	City of Lowell Population
1960	162,890	503
1970	215,401	567
1980	275,226	661
1990	282,912	785
2000	322,959	880

Source: U.S. Census Bureau – Decennial Population and Housing Reports

For the intervening years between the decennial census reports, the Population Research Center (PRC) of Portland State University (PSU) estimates population for Oregon. Population estimates for years 2000 to 2005 are provided in Table 2.2-2. **Note:** Initially, the U.S. Census report in 2000 had determined the population of Lowell to be 857. That figure was disputed and later changed to 880.

Table 2.2-2 – County and City Population Estimates

Year	Lane County Population	City of Lowell Population
2000	323,950	880
2001	325,900	880
2002	328,150	880
2003	329,400	890
2004	333,350	900
2005	336,085	920

Source: PSU Population Research Center – Annual Oregon Population Reports

The data contained in Tables 2.2-1 and 2.2-2 will be considered for population projections in an effort to assess future demands for treated water production by the City of Lowell. The actual population trends for the county and city are displayed in Figures 2.2-1 and 2.2-2.

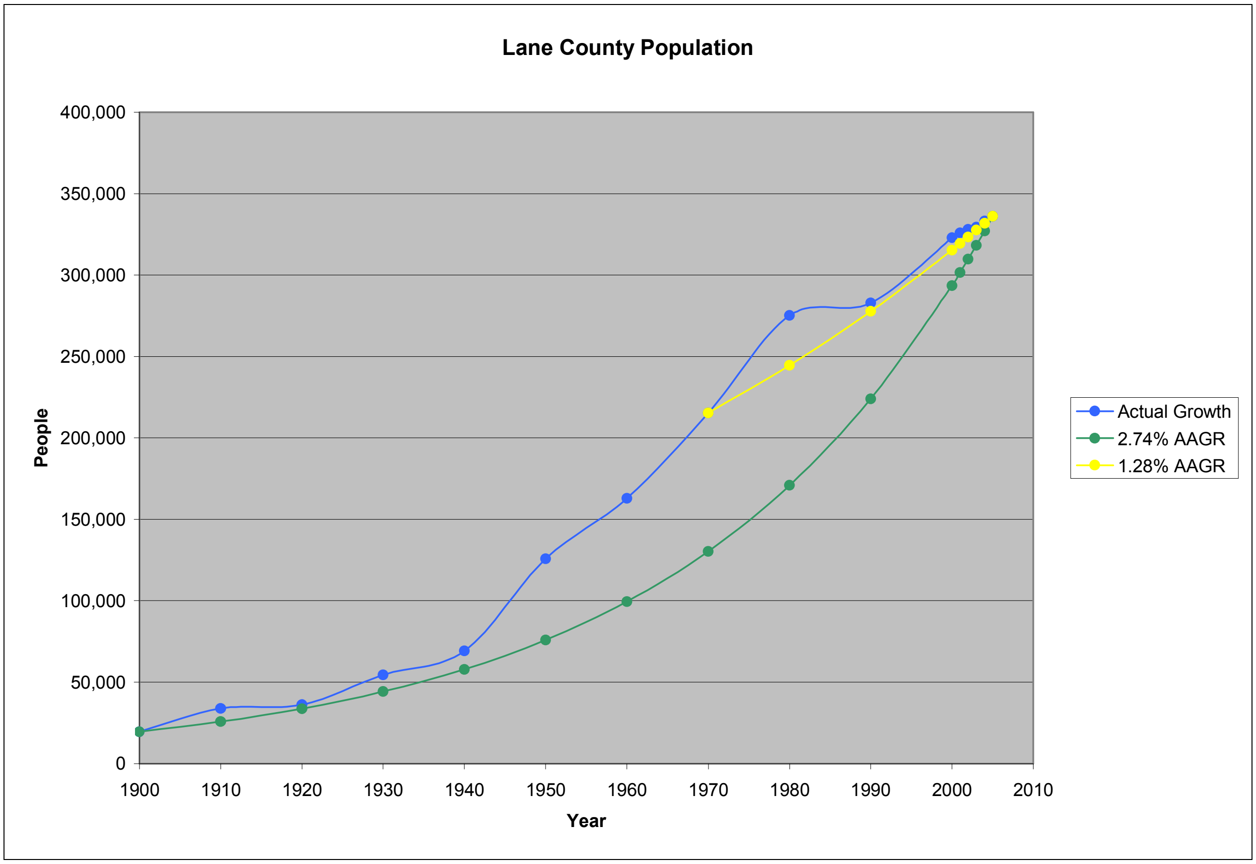


Figure 2.2-1 – Lane County Population Trends

Figure 2.1-1 – Regional Location Map

Figure 2.1-2 – Service Area Map

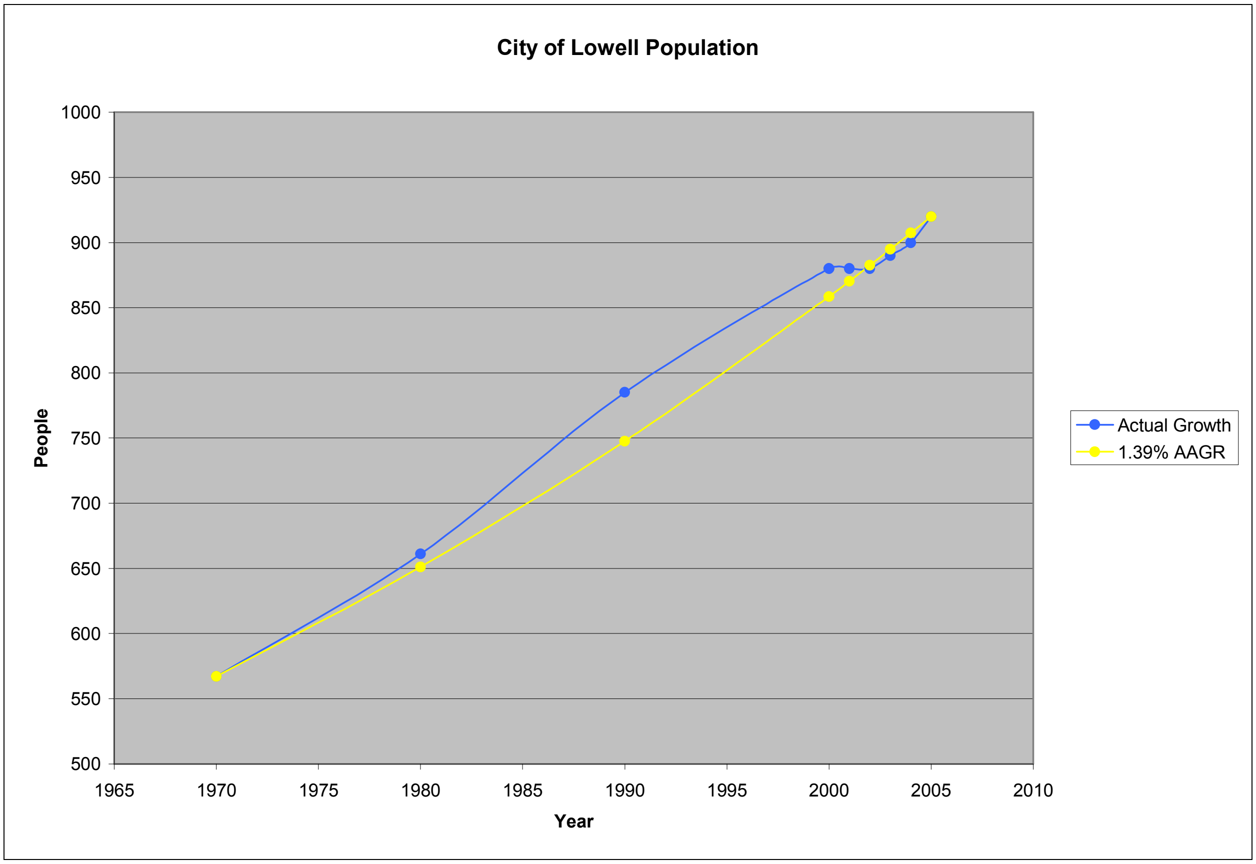


Figure 2.2-2 – City of Lowell Population Trends

An accurate estimate of the existing residential population for a community serviced by a single water supply system can be obtained by examination of the annual water usage records for residential dwellings connected to that system. For the time period from July 2005 through June 2006, the city had an average of 346 active water usage accounts. Of these active accounts, 329 were for residential dwellings which can be categorized as indicated in Table 2.2-3 below:

Table 2.2-3 – Existing Residential Service Population

Dwelling Type	Usage Accounts	Dwelling Units	EDUs	People	People per Unit (Average)
Single-Family Home	324	324	324.0	904	2.79
Apartment Duplex	3	6	3.3	9	1.53
Apartment Complex	1	16	22.0	61	3.84
Mobile-Home Park	1	7	4.8	14	1.91
Total	**329**	**349**	**354.1**	**988**	

Source: 07/2005–06/2006 Lowell Water Usage Records

The elements appearing in this table were determined by the following procedure:

- #(Usage Accounts) and #(Dwelling Units) are directly entered from water usage records.
- #(EDUs) is separately calculated by a methodology described below (refer to Table 2.4-3).
- #(People) = #(EDUs) × 2.79, rounded to the nearest integer.

- #(People per Unit) = #(EDUs) × 2.79 ÷ #(Dwelling Units).

As a result, the serviced residential population for Lowell is estimated to be 988. In order to estimate the total residential population, the unserviced residential population must be included with this number. An inspection of "sewer usage only" accounts reveals that about 27 households, or approximately 75 people ($75 \approx 27 \times 2.79$), rely upon private wells. Thus, the total residential population is estimated to be 1,063.

2.3 Demographics and Economic Conditions

From the U.S. Census report in 2000, the demographics for Lowell are summarized below:

- Age: 34 years or below, 51%; 35 years or above, 49%; median is 34.5 years
- Education: only a high-school diploma/GED; 35%; at least one college degree, 20%
- Household Income: $34,999 or below, 50%; $35,000 or above, 50%; median is $35,540
- Household Occupants: families, 75%; non-families, 25%
- Household Occupants: with age 18 or under, 40%; with age 65 or over, 17%
- Housing Units: owner-occupied, 74%; renter-occupied, 26%
- Housing Units: structures built 1979 or before, 66%; structures built 1980 or after, 34%
- Average number of persons per occupied household: 2.79

A slightly dated economic profile of Lowell is available from the Region 2050 report, published in 2000 by the Lane Council of Governments (LCOG).

In 1998, there were 148 jobs in Lowell. The primary sources of employment in the city include the U.S. Forest Service, two predominant local manufacturers, the local high school, and several small retailers. The aforementioned manufacturers are Eagle Rock Logging and Tumac Industries (a fabricator of custom metal products), each with about 20 employees.

As elsewhere in Oregon, employment has been traditionally oriented toward forestry operations and forest products, but such employment sources have declined in recent years. In contrast, independently-owned small-scale businesses have grown in the Lowell area and in neighboring communities, and there is room for expansion in the retail sector. Specialty agriculture has been identified as having growth potential for the local economy as well.

Although an imminent demand for industrial land was not identified in the Lowell Comprehensive Plan, the community decided to reserve an inventory of land for manufacturing/research activities in the event that such a need arises. Also, Lowell has established an industrial park to which city services have been extended, and space exists for as many as four more businesses to locate there. Finally, Lowell seeks to capitalize on its natural beauty, recreational assets, small-town character, and available land to draw in companies that desire to locate in an attractive environment.

2.4 Population and Projections

According to U.S. Census Bureau data for Lane County, the county population increased from 19,604 in 1900 to 322,959 in 2000. According to Population Resource Center data from PSU, this same population increased to 336,085 by 2005. This total increase corresponds to an average annual growth rate (AAGR) of 2.74% over the 105-year period. However, the AAGR for Lane County from 1970 to 2005 was only

1.28%.

The U.S. Census data for Lowell indicates that the total number of housing units grew at an AAGR of 2.00% from 1970 to 2000 while the population grew at an AAGR of 1.39% for the same 30-year period. As previously mentioned, the average number of persons per occupied household in 2000 was 2.79. The average population trends for the county and city are displayed in Figures 2.2-1 and 2.2-2 (refer to pp. 2-2 and 2-3).

Table 2.4-1 – Historical Population and Housing Units

Year	City of Lowell Population	Total Housing Units	Occupied Housing Units
1960	503	(unavailable)	(unavailable)
1970	567	189	(unavailable)
1980	661	280	(unavailable)
1990	785	288	(unavailable)
2000	880	342	315

Source: U.S. Census Bureau – Decennial Population and Housing Reports

In the 1998 water system master plan for the City of Lowell, water usage records were utilized to more accurately estimate the service population based upon 1997 data. The study arrived at an estimate of 958 people dependent upon the city water system (about 85 additional people relied upon private wells). This study assumed a *linear* growth rate of 5% from 1998 to 2000, and a *linear* growth rate of 3% thereafter until 2020 for purposes of planning. These growth trends led to population estimates of about 1,450 and 2,110 for the years 2006 and 2020, respectively. Clearly, the prediction for 2006 is erroneous, while the prediction for 2020 is optimistic but somewhat dubious.

A projected growth rate must be selected which ideally but accurately represents the next twenty years of growth in Lowell. If the projected growth is less than actually occurs, then the upgraded facilities may become undersized before anticipated, incurring further expenses beyond what prudent planning would have required. If the projected growth is more than actually occurs, then the upgraded facilities remain oversized and underutilized. The disadvantage of underutilization is that excessive capitalization costs become burdensome to both local taxpayers and water consumers. Furthermore, an underutilized facility will not be upgraded for longer periods of time, and so advances in water treatment technology — which generally enable more efficient and economical operations — will not be exploited.

However, long-term practice has revealed that it is preferable to be slightly conservative when adopting growth projections (i.e., avoid underestimation of population growth). Two recent population projection efforts for Lane County are cited below, along with the proposed growth rates for Lowell:

- Lane County Coordinated Population Projections – 2.22% AAGR
- Southern Willamette Valley Regional Growth Management Strategy – 3.30% AAGR

Due to a significant number of pending subdivisions for residential development in the Lowell area, the city administrator has suggested that a 3.30% AAGR be adopted for population projections. For purposes of comparison, AAGR-values of 2.22% and 1.12%, which are closer to historic and recent growth trends for the county and city, also will be considered (1.12% corresponds to the AAGR for Lowell from 2000 to 2005).

These population projections are displayed in Figure 2.4-1. The small jump occurring between 2005 and 2006 is due to inclusion of the more accurate estimate of existing residential service population, which is 988 in 2006.

A 5.00% annual linear growth rate (ALGR) has been included in this figure as well. It is observed that the curve for this growth model will be surpassed by the curve for the 3.30% AAGR model in twenty-five years. AAGR-based models are generally preferred over ALGR-based models for population projections.

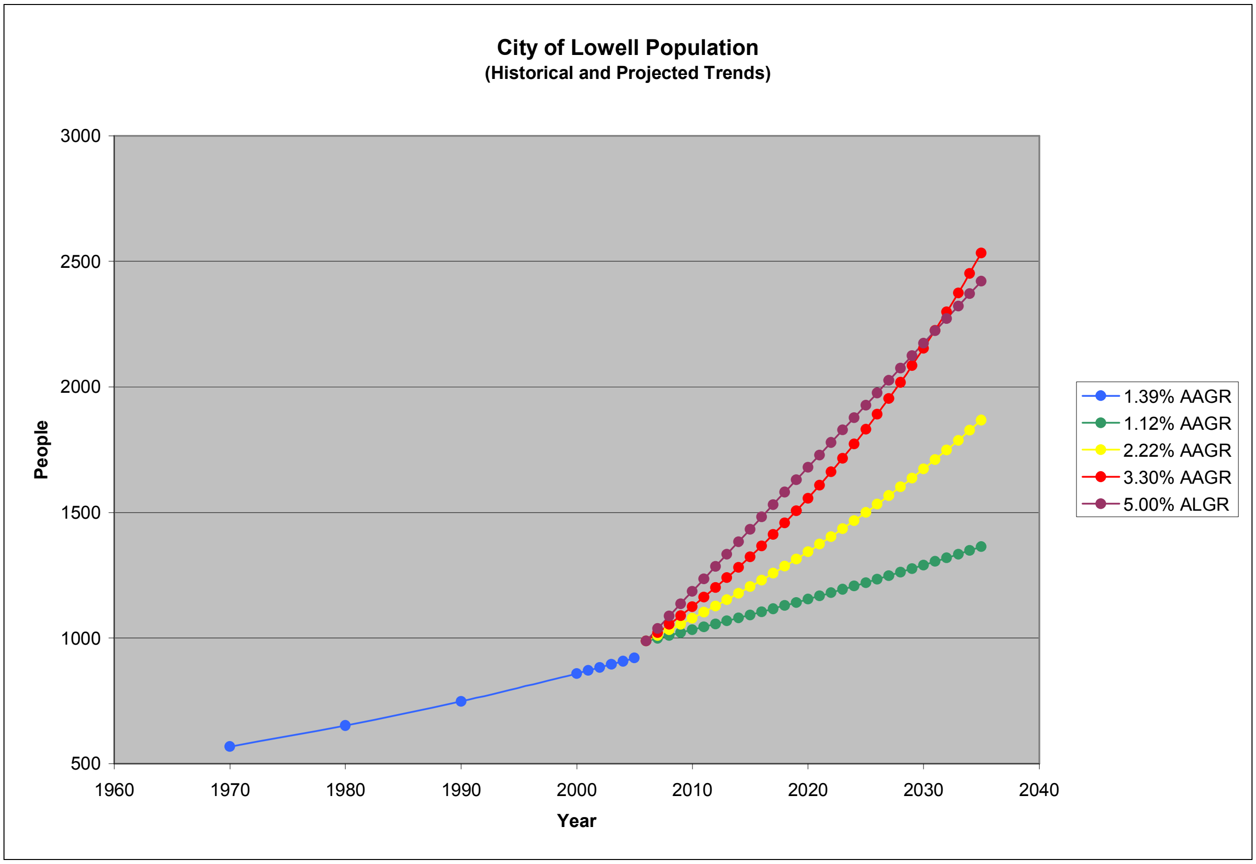


Figure 2.4-1 – Historical and Projected Population Trends

Using a 3.30% AAGR over a 25-year planning period results in a projected population of 2,225 at the end of this period, yielding an increase of 1,237 people. This increase can be understood as an average growth of 49.5 more people per year accompanied by 17.7 more dwelling units per year. Thus, it is estimated that 443 additional dwelling units will be required over the planning period to accommodate this level of growth. It should be confirmed that sufficient land is available within the UGB to support the necessary housing development associated with this level of growth.

Table 2.4-2 – Projected Population and Dwelling Units (3.30% AAGR)

Year	People	People per Unit (Average)	Dwelling Units
2006	988	2.79	354
2011	1,162	2.79	416
2016	1,367	2.79	490
2021	1,608	2.79	576
2026	1,891	2.79	678
2031	2,225	2.79	797

Note: Dwelling Units include Single-Family Homes, Apartment Units, and Mobile Homes.

Equivalent Dwelling Units and Service Population

An accurate estimate of the existing residential population for a community serviced by a single water supply system can be obtained by examination of the annual water usage records for residential dwellings connected to that system. The methodology employed to obtain this estimate is based upon the following observations and assumptions:

- The total amount of treated water consumed by a community is attributable to two basic groups:
  - Residential Consumers (customarily categorized by dwelling type)
  - Non-Residential Consumers (businesses, industries, or institutions)
- The future demands for treated water in a community are estimated by population projections, and these population projections are based upon a predicted growth rate for the residential population.
- For purposes of analysis, it is reasonable to assume that the proportions of treated water consumed by these two basic groups remain constant over time.
- The number of *people per household* is a characteristic parameter for the population analysis. It is calculated from the most recent decennial census data for residential population and is assumed to remain valid at present. Also, it is most representative of residents in single-family homes.
- Thus, it is convenient to express each subgroup of residential consumers (each being distinguished by dwelling type) in terms of *equivalent dwelling units*, or EDUs. As will be seen, each subgroup of non-residential consumers will be handled in this manner as well.
- One EDU is operationally defined as a “water consumer unit” to which can be ascribed an amount of water usage (over a time period) equal to the average water usage (over the same time period) of a single-family home connected to the water supply system. As a result, an apartment complex or a mobile-home park can be assigned an EDU-value that accounts for its water usage in comparison to a single-family home. Typically, the time period is a year.
- After all the residential and non-residential consumers are assigned appropriate EDU-values, it is possible to calculate a total *equivalent service population* (ESP) for the community which reflects current water demands (when significant, well-water users may also be included).
- Finally, in order to assess future water demands for the community, the total ESP is projected over the planning time period by applying the same predicted growth rate as taken for the residential population.

Based upon usage records from July 2005 through June 2006, Lowell water customers consumed a total of 376.8 EDUs, of which residential customers accounted for 354.1 EDUs. The average consumption for a single-family home was 62,253 gallons of water in one year. This amount of water corresponds to the usage associated with one EDU. The usages for non-residential consumers can be expressed in terms of EDU-values as well. For example, the local grocer (CGM) consumed about 249,990 gallons in one year. Dividing this amount of water by 62,253 gallons reveals that this consumer is equivalent to about 4.0 EDUs based upon water usage. EDU-values along with other characteristics for the various city water system customers are provided in Table 2.4-3.

Table 2.4-3 – Annual Usages for City Water System Customers

Consumer Group	Usage Accounts	Consumer Units	Annual Water Usage (gal)	Gallons per Account	EDUs
Single-Family Homes	324	324	20,169,930	62,253	324.0
Apartment Duplexes	3	6	204,840	68,280	3.3
Apartment Complexes	1	12	1,370,710	1,370,710	22.0
Mobile-Home Parks	1	7	298,200	298,200	4.8
Non-Res. Consumers	16	16	1,166,110	72,882	18.7
Local Grocer (CGM)	1	1	249,990	249,990	4.0
Total	**346**	**366**	**23,459,780**		**376.8**

Source: 07/2005–06/2006 Lowell Water Usage Records

The EDU-values provided in Table 2.4-3 directly support the results indicated in Table 2.2-3.

Ordinarily, the equivalent service population (ESP) is obtained by adding the numbers of residential and non-residential consumers as determined from water usage records. In this instance, it is legitimate to ask how the well-water users should be taken into account (in many studies, no well-water users exist within the community serviced by the water system under consideration). It could be argued that the well-water users should be combined with the existing residential service population for the purposes of population projection. However, records suggest that the well-water users are gradually converting over to city water service (due in part to the detection of arsenic in local ground water sources).

Since the new housing developments commensurate with population growth must (as mandated by city ordinance) exclusively rely upon the city water system, only the existing residential service population should form the basis for population projections (i.e., the number of well-water users will not increase in proportion over time). Also, by the end of the 25-year planning period, the 75 well-water users will be negligible in comparison to the projected residential service population of 2,225, and conservative design factors utilized later in this study will easily allow for the well-water users to be absorbed into the water system as they eventually convert to city water service.

Therefore, the residential population (estimated to be 988 without inclusion of the well-water users) will be added to the effective non-residential population, which amounts to approximately 63 people from the total non-residential 22.7 EDUs indicated in Table 2.4-3 ($63 \approx 22.7 \times 2.79$). Thus, the current ESP is estimated to be 1,051 ($1,051 = 988 + 63$).

Based upon a 3.30% AAGR, the projected population and EDU-values are indicated in Table 2.4-4 below for the City of Lowell.

Table 2.4-4 – Projected Population and EDU-Values (3.30% AAGR)

Year	Residential Population	Residential EDUs	Non-Res. EDUs	Local Grocer EDUs	Total System EDUs	Equiv. Serv. Population
2006	988	354.1	18.7	4.0	376.8	1,051
2007	1,021	365.8	19.3	4.1	389.2	1,086
2008	1,054	377.9	20.0	4.3	402.1	1,122
2009	1,089	390.3	20.6	4.4	415.3	1,159
2010	1,125	403.2	21.3	4.6	429.1	1,197
2011	1,162	416.5	22.0	4.7	443.2	1,237
2012	1,200	430.3	22.7	4.9	457.8	1,277
2013	1,240	444.5	23.5	5.0	472.9	1,320
2014	1,281	459.1	24.2	5.2	488.6	1,363
2015	1,323	474.3	25.0	5.4	504.7	1,408
2016	1,367	489.9	25.9	5.5	521.3	1,455
2017	1,412	506.1	26.7	5.7	538.5	1,503
2018	1,459	522.8	27.6	5.9	556.3	1,552
2019	1,507	540.0	28.5	6.1	574.7	1,603
2020	1,557	557.9	29.5	6.3	593.6	1,656
2021	1,608	576.3	30.4	6.5	613.2	1,711
2022	1,661	595.3	31.4	6.7	633.5	1,767
2023	1,716	614.9	32.5	6.9	654.4	1,826
2024	1,772	635.2	33.5	7.2	676.0	1,886
2025	1,831	656.2	34.7	7.4	698.3	1,948
2026	1,891	677.8	35.8	7.7	721.3	2,012
2027	1,954	700.2	37.0	7.9	745.1	2,079
2028	2,018	723.3	38.2	8.2	769.7	2,147
2029	2,085	747.2	39.5	8.4	795.1	2,218
2030	2,154	771.9	40.8	8.7	821.3	2,292
2031	2,225	797.3	42.1	9.0	848.4	2,367

An Explanation of AAGR versus ALGR for Population Growth

In the field of population analysis and research, AAGR customarily denotes *average annual growth rate*. It might be better to understand and describe AAGR as *annual accumulated growth rate*.

A 5% AAGR means that a population increases by 5% each year based upon its size at the beginning of that year. For example, an original population of 1,000 would increase to 1,050 ($1{,}050 = 1.05 \times 1{,}000$) at the end of the first year, 1,102 ($1{,}102 \approx 1.05 \times 1{,}050$) at the end of the second year, and so on.

For the purposes of this study, ALGR will denote *annual linear growth rate*. A 5% ALGR means that a population *perpetually* increases each year by 5% of its *original* size. For example, an original population of 1,000 would increase to 1,050 ($1{,}050 = 1{,}000 + 50$, where $50 = 0.05 \times 1{,}000$) at the end of the first year, 1,100 ($1{,}100 = 1{,}050 + 50$) at the end of the second year, and so on.

When AAGR and ALGR values are equal, an AAGR-based model always results in faster growth than an ALGR-based model.

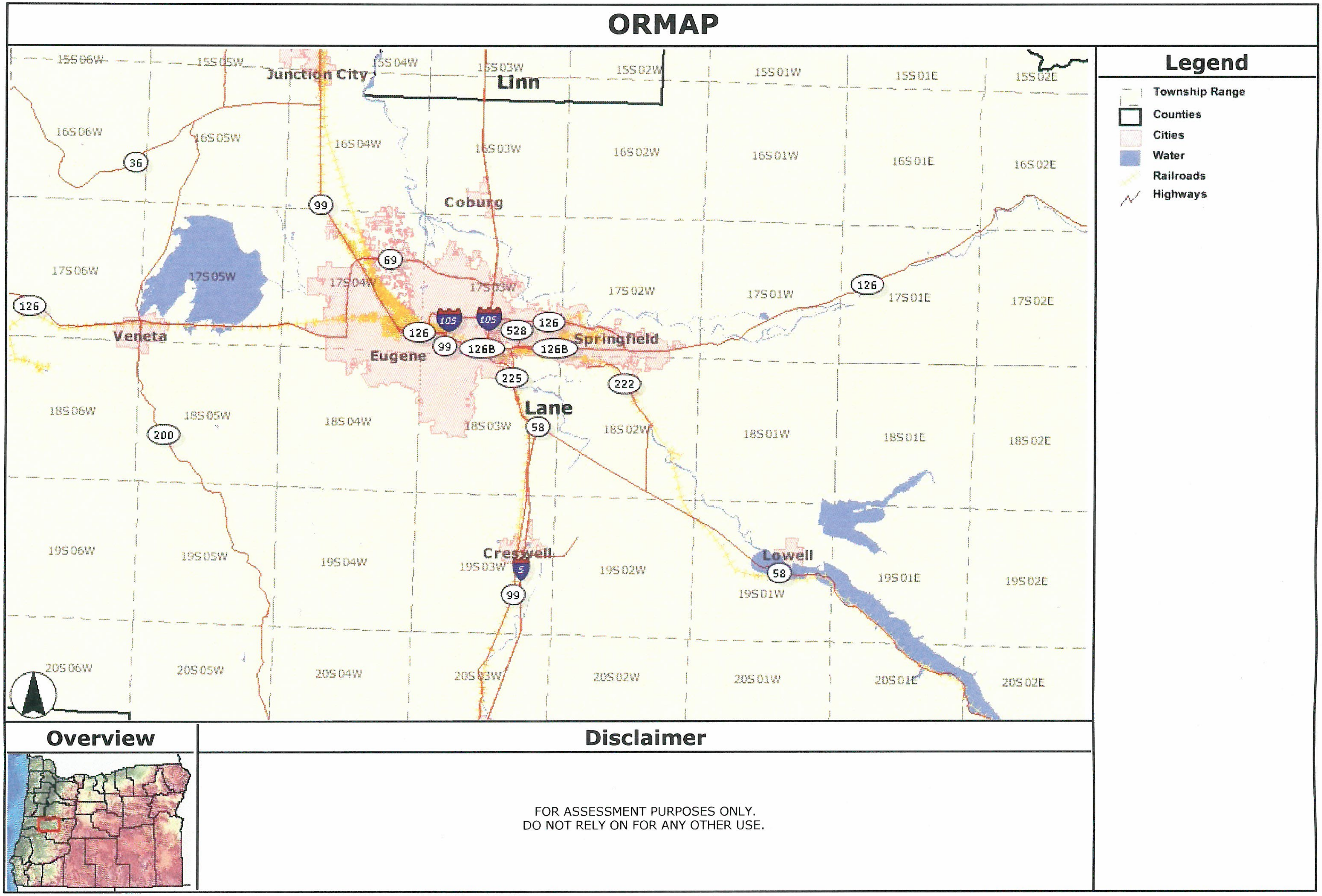


Figure 2.1-1 – Regional Location Map

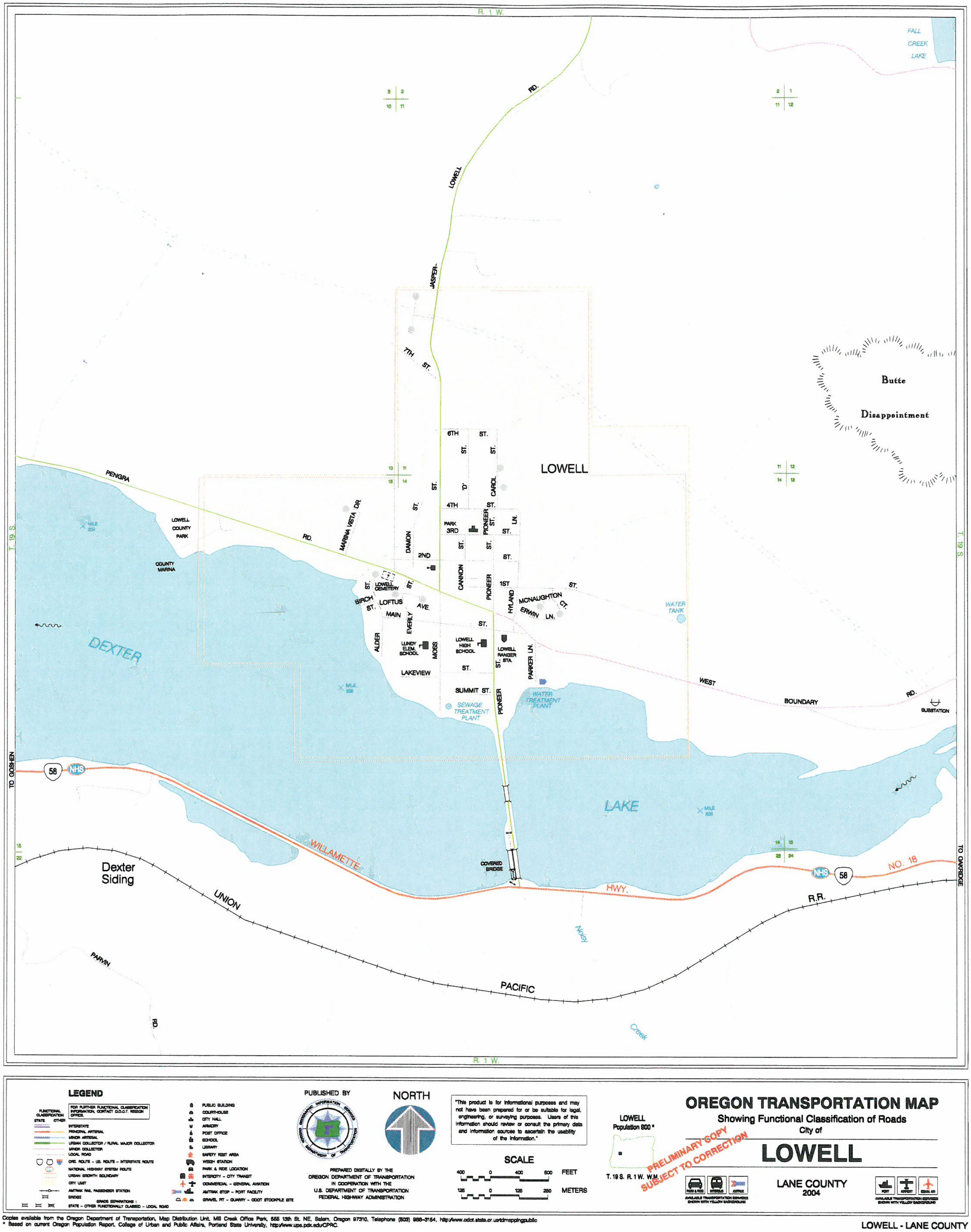


Figure 2.1-2 – Service Area Map

Section 3

Regulatory Framework

3.1 Responsibilities of Water Suppliers

Per Oregon Administrative Rules (OAR) 333–061–0025, water suppliers are responsible for taking all reasonable precautions to ensure that the water delivered to water consumers does not exceed maximum contaminant levels, to ensure that water system facilities are devoid of public health hazards, and to ensure that water system operation and maintenance are performed as required by these rules. This includes, but is not limited to, the following responsibilities:

- Routinely collect and submit water samples for laboratory analyses at the frequencies and sampling points prescribed by OAR 333-061-0036 "Sampling and Analytical Requirements";
- Take immediate and appropriate corrective action when the results of analyses or measurements indicate that maximum contaminant levels have been exceeded, and report the results of these analyses as prescribed by OAR 333-061-0040 "Reporting and Record Keeping";
- Continue to report, as prescribed by OAR 333-061-0040, the results of analyses or measurements which indicate that maximum contaminant levels (MCLs) have not been exceeded;
- Notify all customers of the system, as well as the general public in the service area, when the MCLs have been exceeded;
- Notify all customers serviced by the system when the reporting requirements are not being met, or when public health hazards are found to exist in the system, or when the operation of the system is subject to a permit or a variance;
- Maintain monitoring and operating records, and make these records available for review when the system is inspected;
- Maintain a water pressure of at least 20 pounds per square inch (psi) at all service connections at all times (at the property line);
- Respond to complaints relating to water quality from consumers, and maintain records and reports on actions undertaken;
- Conduct an active program for systematically identifying and controlling cross connections;
- Submit, to the DWP, plans prepared by a professional engineer registered in Oregon for review and approval before undertaking the construction of new water systems or major modifications to existing water systems, unless exempted from this requirement;
- Ensure that the water system is in compliance with OAR 333–061–0205 "Water Personnel Certification Rules – Purpose", relating to certification of water system operators;
- Ensure that Transient Non-Community water systems utilizing surface water sources or sources under the influence of surface water are in compliance with OAR 333–061–0065 "Operation and Maintenance" (2c), relating to required special training.

3.2 Public Water System Regulations

Water suppliers should always be informed of current standards, which can change over time, and should also be aware of pending future regulations. As of this writing, OAR Chapter 333, Division 61 (covering Public Water Systems) is over 280 pages in length. This section is not meant to be a comprehensive list of all requirements but a summary of the general requirements.

Specific information on the regulations concerning public water systems may be found in OAR Chapter 333, Division 61. The rules can be found on the Internet at http://oregon.gov/DHS/ph/dwp/rules.shtml, where copies of all rules and regulations can be downloaded or printed out for reference. A summary of Oregon drinking water quality standards published in the *Pipeline Newsletter* (Volume 19, Issue 4, Fall 2004) by DWP is included in Appendix A.

Drinking water regulations were established in 1974 with the signing of the Safe Drinking Water Act (SDWA). This act and subsequent regulations were the first to apply to all public water systems in the United States. The Environmental Protection Agency (EPA) was authorized to set standards and implement this act. With the enactment of the Oregon Drinking Water Quality Act in 1981, the state accepted primary enforcement responsibility for all drinking water regulations within the state. Requirements are detailed in OAR Chapter 333, Division 61. The SDWA and associated regulations have been amended several times since inception with the goal of further protection of public health.

The SDWA requires the EPA to regulate contaminants that present health risks and which are known (or are likely) to occur in public drinking water supplies. For each contaminant requiring federal regulation, the EPA sets a non-enforceable health goal, or maximum contaminant level goal (MCLG). This is the level of a contaminant in drinking water below which there is no known or expected risk to health. The EPA is then required to establish an enforceable limit, or maximum contaminant level (MCL), which is as close to the MCLG as is technologically feasible, taking cost into consideration. Where analytical methods are not sufficiently developed to measure the concentrations of certain contaminants in drinking water, the EPA specifies a treatment technique instead of an MCL to protect against these contaminants.

Water systems are required to collect water samples at designated intervals and locations. The samples must be tested in state approved laboratories. The test results are then reported to the state, which determines whether the water system is in compliance or violation with the regulations. There are three main types of violations:

1) MCL violation – occurs when tests indicate that the level of a contaminant in treated water is above the EPA's or the state's legal limit (states may set standards equal to or more protective than those of the EPA). These violations indicate a potential health risk, which may be immediate or long-term.
2) Treatment technique violation – occurs when a water system fails to treat its water in the way prescribed by the EPA (for example, by not disinfecting). Similar to MCL violations, treatment technique violations indicate a potential health risk to consumers.
3) Monitoring and reporting violation – occurs when a water system fails to test its water for certain contaminants, or fails to report test results in a timely fashion. If a water system does not properly monitor its water, then potential health risks to consumers cannot be adequately detected.

When a system violates EPA/state rules, that system is required to notify the state and the public. States are primarily responsible for taking appropriate enforcement actions if systems with violations do not return to compliance. States are also responsible for reporting violation and enforcement information to the EPA quarterly.

There are now EPA-established drinking water quality standards for 91 contaminants, including seven microbials and turbidity, seven disinfection byproducts and residuals, 16 inorganics (including lead and copper), 56 organics, and five radiologic contaminants. These standards have either established MCLs or specified treatment techniques.

New rules in effect since the year 2000 include the Interim Enhanced Surface Water Treatment Rule, the Filter Backwash Recycling Rule, the Long-Term Stage 1 Enhanced Surface Water Treatment Rule, the Stage 1 Disinfectant/Disinfection Byproducts Rule, and revisions to the Lead and Copper Rule.

A general summary of current rules is provided below for a surface water system using conventional filtration treatment and servicing less than 10,000 people.

Total Coliform Rule

Routine samples collected by Oregon public water suppliers are analyzed for total coliform bacteria. All community systems, as well as non-community systems utilizing surface water sources or servicing over 1,000 people, must sample monthly. All other systems must test for coliform bacteria once per quarter. For systems servicing between 1,001 and 2,500 people, 2 samples per month are required. Systems servicing between 2,501 and 3,300 people are required to take 3 samples per month. Systems servicing between 3,301 and 4,100 people are required to take 4 samples per month. Systems servicing between 4,101 and 4,900 people are required to take 5 samples per month. Systems that service more than 4,900 people are subject to additional sampling requirements.

Compliance is based upon the presence or absence of total coliforms in any calendar month (or quarter). Sample results are reported as either “coliform-absent” or “coliform-present”. When any sample yields coliform-present results, a set of at least three repeat samples must be collected within 24 hours. Small water systems that collect one routine sample per month or fewer must collect a fourth repeat sample. Repeat sampling continues until a set of repeat samples with coliform-absent results is obtained or the MCL for total coliforms in OAR 333–061–0030 (4) has been exceeded (in which case the system is out of compliance).

Small systems (fewer than 40 samples/month) are allowed no more than one coliform-present sample per month, including any repeat sample results. Larger systems (40 or greater samples/month) are allowed no more than five percent coliform-present samples during any month, including any repeat sample results. Confirmed presence of fecal coliform, or *E. coli*, presents an acute health risk and requires immediate notification of the public to take protective actions such as boiling of water or using bottled water.

Surface Water Treatment Rules

Water systems must provide a total level of filtration and disinfection treatment to remove/inactivate 99.9% (“3-log”) of *Giardia lamblia*, and to remove/inactivate 99.99% (“4-log”) of viruses. In addition, filtered water systems must physically remove 99% (“2-log”) of *Cryptosporidium*. Furthermore, these systems must meet specified performance standards for combined filter effluent turbidity levels, and water systems using conventional and direct filtration must also record individual filter effluent turbidity and take appropriate action if specified action levels are exceeded. When more than one filter exists, the effluent turbidity of each filter must be continuously monitored and recorded at least once every 15 minutes. The combined flow from all filters must have a turbidity measurement taken at least once every four hours by grab sampling or continuous monitoring. Compliance is based upon the combined filter effluent, and 100% of measurements must be less than or equal to 1.0 NTU while 95% of the readings taken during any month must be less than or equal to 0.3 NTU.

- Individual filter turbidity monitored continuously and recorded every 15 minutes or less
- Combined filter turbidity monitored continuously or grab sample taken every four hours or less

- Combined filter turbidity less than 1.0 NTU in 100% of measurements
- Combined filter turbidity less than or equal to 0.3 NTU in 95% of monthly measurements
- Specific follow-up actions required if individual filter turbidity exceeds 1.0 NTU twice

All water systems must meet specified C × T [concentration × time] requirements for disinfection, as well as meet required removal/inactivation levels. In addition, a disinfectant residual must be maintained in the distribution system.

- Conduct continuous recording of disinfectant residual at entry point to the distribution system. Small systems may be allowed to substitute one to four daily grab samples.
- Perform daily calculation of C × T at highest flow (peak hourly flow).
- Provide adequate C × T to meet needed removal/inactivation levels.
- Maintain a continuous minimum 0.2 mg/L disinfectant residual at entry point to the distribution system.
- Maintain a minimum detectable disinfectant residual in 95% of the distribution system samples (collected at coliform bacteria monitoring points).
- Conduct disinfection profiling and benchmarking.

Filtered water systems that recycle spent filter backwash water or other waste flows must return those flows through all treatment processes in the filtration plant. Systems wishing to recycle filter backwash water must provide notice to the State including a plant schematic showing the origin, conveyance, and return location of the recycled flows. Design flows, observed flows, and typical recycle flows are also required along with a state-approved plant operating capacity.

Disinfectants and Disinfection Byproducts

Disinfection treatment chemicals used to kill microorganisms in drinking water can react with naturally occurring organic and inorganic matter in the source water, called DBP precursors, to form disinfection byproducts (DBPs). Some DBPs have been shown to cause cancer and adverse reproductive effects in laboratory animals and suggested bladder cancer and adverse reproductive effects in humans. The challenge is to apply levels of disinfection treatment needed to kill disease-causing microorganisms while limiting the levels of disinfection byproducts produced. The primary disinfection byproducts of concern in Oregon are the trihalomethanes (TTHM) and the haloacetic acids (HAA5).

Disinfection byproducts must be monitored throughout the distribution system at frequencies daily, monthly, quarterly or annually, depending on the population serviced, type of water source, and the specific disinfectant applied, and in accordance with an approved monitoring plan. Disinfectant residuals must be monitored at the same locations and frequency as coliform bacteria.

Total organic carbon (TOC) is an indicator of the levels of DBP precursor compounds in the source water. Systems utilizing surface water sources and conventional filtration treatment must monitor source water for TOC and alkalinity monthly, as well as practice enhanced coagulation to remove TOC if it exceeds 2.0 mg/L as a running annual average.

Compliance is determined based upon meeting maximum contaminant levels (MCLs) for disinfection byproducts and maximum levels for disinfectant residual (MRDLs) over a running annual average of the sample results, to be computed quarterly.

- TTHM/HAA5 monitoring throughout distribution system. One sample per quarter is required for systems servicing between 500 and 9,999 people. One sample per year during warmest month is required for systems servicing less than 500 people.
- MCL for TTHM is 0.080 mg/L; MCL for HAA5 is 0.060 mg/L.
- TOC and alkalinity monitoring in source water monthly. Enhanced coagulation if TOC is greater than 2.0 mg/L.
- Compliance with MRDLs:
  - Limit for chlorine (free Cl_2 residual) is 4.0 mg/L.
  - Limit for chloramines is 4.0 mg/L (as total Cl_2 residual).
  - Limit for chlorine dioxide is 0.8 mg/L (as ClO_2).
- MCL for Bromate is 0.010 mg/L.
- MCL for Chlorite is 1.0 mg/L.

Lead and Copper

Excessive levels of lead and copper are harmful, and rules exist to reduce exposure to these elements via drinking water. Lead and copper enter drinking water primarily from corrosion of plumbing materials containing lead and copper. Lead comes from solder and brass fixtures. Copper comes from copper tubing and brass fixtures. Protection is provided by limiting the corrosiveness of water transmitted into the distribution system. Treatment alternatives include pH adjustment, alkalinity adjustment, or both, or adding passivating agents such as orthophosphates.

Samples from community systems are collected from homes built prior to the prohibition of lead solder in Oregon (1985). One-liter samples of standing water (first drawn after six hours of non-use) are collected at homes identified in the water system sampling plan. Two rounds of initial sampling are required, collected at six-month intervals. Subsequent annual sampling from a reduced number of sites is required after demonstration that lead and copper action levels are met. After three rounds of annual sampling, samples are required every three years. The number of initial and reduced samples required is dependent on the population serviced by the water system.

In each sampling round, 90% of samples from homes must have lead levels less than or equal to the Action Level of 0.015 mg/L and copper levels less than or equal to 1.3 mg/L. Water systems with lead above the Action Level must conduct periodic public education, and either install corrosion control treatment, change water sources, or replace plumbing apparatuses.

- Document and maintain a sampling plan for applicable homes.
- Collect required samples.
- Meet Action Levels for Lead and Copper:
  - 0.015 mg/L for Lead
  - 1.3 mg/L for Copper
- Provide corrosion control treatment and take other remediation steps when Action Levels not met.

Inorganic Contaminants

The permissible levels of many inorganic contaminants are regulated for public health protection. These contaminants occur both naturally and as the result of agricultural or industrial operations. Inorganic contaminants most often come from the source of water supply, but can also enter water systems from contact with materials used for pipes and storage tanks. Regulated inorganic contaminants include: arsenic, asbestos, fluoride, mercury, nitrate, nitrite, and others. Compliance is achieved by meeting the established MCLs for each contaminant. Systems that cannot meet one or more MCLs must either install treatment systems (such as ion exchange or reverse osmosis) or develop alternate sources of water.

- Sample quarterly for Nitrate (reduction to annual sampling may be permitted).
- Communities with Asbestos Cement (AC) pipe must sample every 9 years for Asbestos.
- Sample annually for Arsenic. **New MCL of 0.010 mg/L effective January 2006**.
- Sample annually for all other inorganic contaminants. Waivers available when monitoring records show three samples with results below MCLs. MCLs will vary based upon contaminant.

Organic Contaminants

Organic contaminants are regulated to reduce exposure to harmful chemicals via drinking water. Examples include acrylamide, benzene, 2,4-dichlorophenoxyacetic acid (2,4-D), styrene, toluene, and vinyl chloride. Major types of organic contaminants are Volatile Organic Chemicals (VOCs) and Synthetic Organic Chemicals (SOCs). Organic contaminants are usually associated with industrial or agricultural activities that affect sources of drinking water supply, including industrial and commercial solvents and chemicals, and pesticides. These contaminants can also enter from materials in contact with the water such as pipes, valves, and paints and coatings used inside water storage tanks.

At least one test for each contaminant from each water source is required during every 3-year compliance period. Public water systems servicing more than 3,300 people must test twice during each 3-year compliance period for SOCs. Public water systems using surface water sources must test for VOCs annually. Compliance is achieved by meeting the established MCL for each contaminant. Quarterly follow-up testing is required for any contaminants that are detected above the specified MCL. Only those systems determined by the State to be at risk must monitor for dioxin. Water systems using polymers containing acrylamide or epichlorohydrin in their water treatment process must keep their dosages below specified levels. Systems that cannot meet one or more MCL must either install or modify water treatment systems (such as activated carbon and aeration) or develop alternate sources of water.

- At least one test for each contaminant (from each water source) every 3-year compliance period.
- Sample twice during compliance period for each SOC when system services over 3,300 people.
- Test for VOCs annually.
- Quarterly follow-up testing required for any contaminant detected above its MCL.
- Maintain polymer dosages in treatment process below specified levels.
- MCLs will vary based upon contaminant.

Radiologic Contaminants

Radioactive contaminants, both natural and man-made, can result in an increased risk of cancer from long-term exposure and are regulated to reduce exposure via drinking water. Rules were recently revised to include a new MCL for uranium, and to clarify and modify monitoring requirements. Initial monitoring tests, quarterly for one year at the entry point from each source, must be completed by December 31, 2007 for gross alpha, radium-226, radium-228, and uranium. A single analysis for all four contaminants collected between June 2000 and December 2003 will substitute for the four initial samples. Gross alpha may substitute for radium-226 if the gross alpha result does not exceed 5 pCi/L and may substitute for uranium monitoring if the gross alpha result does not exceed 15 pCi/L. Subsequent monitoring is required every three, six, or nine years depending on the initial results, with a return to quarterly monitoring if the MCL is exceeded. Compliance with MCLs is based upon the average of the four initial test results, or subsequent quarterly tests. Community water systems that cannot meet MCLs must install treatment (such as ion exchange or reverse osmosis) or develop alternate water sources.

- Conduct initial quarterly tests for one year by 12-31-2007 (prior tests may be accepted).
- Subsequent monitoring every 3, 6, or 9 years, depending on initial results.
- Comply with MCLs based upon average of test results.
- **New MCL of 30 µg/L for Uranium**. Other MCLs will vary based upon contaminant.

3.3 Future Water System Regulations

The 1996 Safe Drinking Water Act (SDWA) requires the EPA to review and revise as appropriate each current standard at least every six years. Data continues to be collected on contaminants currently unregulated in order to support development of future drinking water standards. Drinking water contaminant candidate lists (DWCCL) are prepared and revised every five years. The first DWCCL was published on March 2, 1998 and included 51 chemicals and 9 microbials. In 2003, the EPA decided not to regulate any of the 9 microbials from the initial list. On April 2, 2004, the EPA published a draft second DWCCL consisting of the remaining 51 contaminants from the first list. The EPA must publish a decision on whether to regulate at least five contaminants from the DWCCL every 5 years. As a result, additional contaminants can become regulated in the future.

In addition, rule revisions and new rules will occur to further address health risks from disinfection byproducts and pathogenic organisms. Rules such as the Long-Term Stage 1 Enhanced Surface Water Treatment Rule (LT1ESWTR) and the Stage 1 Disinfectants/Disinfection Byproducts (Stage 1 D/DBP) Rule have recently gone into effect. These rules added *Cryptosporidium* as a pathogen of concern, decreased the acceptable turbidity levels, addressed disinfectants and disinfection byproducts, and lowered MCLs for certain contaminants. New and revised drinking water quality standards are mandated under the 1996 federal SDWA. Known future standards (along with their likely EPA promulgation date) include:

- Groundwater Rule (2005)
- Enhanced Surface Water Treatment Rule, Stage 2 (2005) **– EPA published Jan. 5, 2006**
- Disinfectants and Disinfection Byproducts Rule, Stage 2 (2005) **– EPA published Jan. 4, 2006**
- Radon Rule (2005-06)
- Distribution Rule, including revised coliform bacteria requirements (2008)

Water suppliers should be aware of and familiar with these mandates and deadlines, and plan strategically to meet them. The Oregon DHS, under the Primacy Agreement with the EPA, has up to two years to adopt each federal rule after it is finalized. Water suppliers generally have at least three years to comply with each federal rule after it is finalized; however, some of these rules will likely establish a significant number of compliance dates for water suppliers that will occur prior to state adoption of the rules. These "early implementation" dates will likely have to be implemented in Oregon directly by the EPA, because the state program will not yet have the rules in place or the resources to carry them out.

These anticipated rules are described generally below. Additional details will be found in the final EPA rules once they are promulgated.

Groundwater Rule

Monitoring will be required for specific pathogenic organisms and/or indicator organisms, such as enteric viruses or surrogate organisms. In addition, all public water supply wells must be evaluated for their hydrogeologic sensitivity to viruses, including well construction, site geology, and source water area. Compliance will be achieved by demonstrating a low hydrogeologic sensitivity to viruses, modifying well construction if needed, or by installing disinfection treatment to inactivate viruses.

Long-Term Stage 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)

The rule will apply to all public water systems using surface water sources of supply. The rule will identify those surface water supplies that are at high risk of *Cryptosporidium*, and prescribe additional levels of treatment selected from a matrix of options. Future standards are likely to require water systems with high levels of pathogens in the source water to add treatment beyond standard filtration and disinfection. Monitoring of source water will be required for specific pathogenic organisms including *Cryptosporidium*, *E. coli*, and turbidity. Compliance will be demonstrated by meeting a maximum running annual average in source water for pathogens, or by meeting additional treatment technique requirements associated with the levels of pathogens found if those levels exceed the maximum.

- Filtered water systems will be classified in one of four treatment categories (bins) based upon their monitoring results. Most systems are expected to be classified in the lowest bin and will face no additional requirements. Systems classified in higher bins must provide additional water treatment to further reduce *Cryptosporidium* levels by 90 to 99.7 percent (1.0 to 2.5-log), depending on the bin. Systems will select from different treatment and management options in a "microbial toolbox" to meet their additional treatment requirements.
- All unfiltered water systems must provide at least 99 or 99.9 percent (2 or 3-log) inactivation of *Cryptosporidium*, depending on the results of their monitoring.
- Systems that store treated water in open reservoirs must either cover the reservoir or treat the reservoir discharge to inactivate 4-log virus, 3-log *Giardia lamblia*, and 2-log *Cryptosporidium*. These requirements are necessary to protect against the contamination of water that occurs in open reservoirs.
- Disinfection Benchmarking: Systems must review their current level of microbial treatment before making a significant change in their disinfection practice. This review will assist systems in maintaining protection against microbial pathogens as they take steps to reduce the formation of disinfection byproducts under the Stage 2 Disinfection Byproducts Rule, which the EPA is finalizing along with the LT2ESWTR.

Disinfectants and Disinfection Byproducts Rule, Stage 2 (Stage 2 D/DBP)

The rule will apply to all water systems that apply disinfectants or distribute water that has been disinfected. The main goal of the Stage 2 rule is to control peak DBP levels within the water distribution system. Systems will monitor for DBPs at sample locations where peak levels are expected, as identified in an Initial Distribution System Evaluation (IDSE). Large systems must complete the IDSE within two years of the final rule date and small systems within 4 years of the final rule date. Compliance is based upon meeting the Locational Running Annual Average (LRAA) for DBPs at each sampling location in the distribution system in two phases. Phase 1: meet LRAA at each stage 1 sampling point for TTHM (120 µg/L) and HAA5 (100 µg/L). Phase 2: meet LRAA at each IDSE-identified sampling point for TTHM (80 µg/L) and HAA5 (60 µg/L) within 6–8½ years of the final rule, depending on system size.

- Systems will conduct an evaluation of their distribution systems, known as an Initial Distribution System Evaluation (IDSE), to identify the locations with high disinfection byproduct concentrations. These locations will then be used by the systems as the sampling sites for Stage 2 DBP rule compliance monitoring.
- Compliance with the MCLs for two groups of disinfection byproducts (TTHM and HAA5) will be calculated for each monitoring location in the distribution system. This approach, referred to as the locational running annual average (LRAA), differs from current requirements, which determine compliance by calculating the running annual average of samples from all monitoring locations across the system.
- The Stage 2 DBP rule also requires each system to determine if they have exceeded an operational evaluation level, which is identified using their compliance monitoring results. The operational evaluation level provides an early warning of possible future MCL violations, which allows the system to take proactive steps to remain in compliance. A system that exceeds an operational evaluation level is required to review their operational practices and submit a report to their state that identifies actions that may be taken to mitigate future high DBP levels, particularly those that may jeopardize their compliance with the DBP MCLs.

Radon Rule

All community water systems using groundwater sources will conduct quarterly initial sampling at distribution system entry points for one year. Subsequent sampling will occur once every 3 years. The Radon MCL is expected to be 300 pCi/L. An alternative MCL (AMCL) of 4,000 pCi/L is proposed if the State develops and adopts an EPA-approved statewide Multi-Media Mitigation (MMM) program. Local communities may have the option of developing an EPA-approved local MMM program in the absence of a statewide MMM program, and meeting the AMCL.

Distribution Rule

Under this rule, current requirements for coliform bacteria will be revised, emphasizing fecal coliforms and *E. coli*, and focusing on protection of water within the distribution system. The rule will apply to all public water systems and will involve: identifying and correcting sanitary defects and hazards in a water system, and using best management practices for disinfection to control coliform bacteria in the system.

3.4 Water Management and Conservation Plans

The Municipal Water Management and Conservation Planning program provides a process for municipal water suppliers to develop Water Management and Conservation Plans (WMCPs) to meet future water needs. Many municipal water suppliers are required to prepare plans under water right permit conditions. In addition, with the revision of the permit extension rules in Fall 2002, communities seeking long-term permit extensions will be required to prepare such plans. These plans will be used to demonstrate a community's needs for increased diversions of water under the permits as their demands grow. The rules for WMCPs are detailed in OAR 690, Division 86.

A WMCP provides a description of the water supply system, identifies the sources of water used by the community, and explains how the water supplier will manage and conserve supplies to meet future needs. Preparation of a plan is intended to represent a pro-active evaluation of the management and conservation measures that suppliers can undertake. The planning program requires municipal water suppliers to consider water that can be saved through conservation practices as a source of supply to meet growing demands if the saved water is less expensive than the option of developing new supplies. As such, a WMCP represents an integrated resource management approach to securing a community's long-term water supply.

Many elements required in a WMCP are also required in similar plans by the Oregon Department of Human Services (water system master plans) and the Oregon Department of Land Conservation and Development (public facilities plans). Water suppliers may consolidate overlapping plan elements and create a single master plan that meets the requirements of all three plans.

Every municipal water supplier required to submit a WMCP shall exercise diligence in implementing the approved plan and shall update and resubmit a plan consistent with the requirements of the rules as prescribed during plan approval. Progress reports are required showing 5-year benchmarks, water use details, and a description of the progress made in implementing the associated conservation or other measures.

A WMCP shall include the following elements:

1) Water System Description, including infrastructure details, supply sources, service area and population, details of water use permits and certificates, water use details, customer details, system schematic, and leakage information.
2) Water Conservation Element, including description of conservation measures implemented and planned, water use and reporting program details, progress on conservation measures, and conservation benchmarks.
3) Water Curtailment Element, including current capacity limitations and supply deficiencies, three or more stages of alert for potential water shortages or service difficulties, levels of water shortage severity and curtailment action triggers, and specific curtailment actions to be taken for each stage of alert.
4) Water Supply Element, detailing current and future service areas, estimates of when water rights and permits will be fully exercised, demand projections for 10 and 20 years, evaluation of supply versus demand, and additional details should an expansion of water rights be anticipated.

Failure to comply with these rules for WMCPs can result in enforcement actions by the Water Resources Department Director. Enforcement actions may include requirements for additional information and planning, water use regulation, cancellation of water use permits, or civil penalties under OAR 690–260–0005 to 690–260–0110.

Section

4

Existing Water System

4.1 Introduction/Background

The City of Lowell water supply system is classified by the Oregon Department of Human Services (DHS) Drinking Water Program (DWP) as a "community" water system, identified within the public water system inventory by Public Water System (PWS) identification number OR4100492. At present, the DWP database lists the service population for this water system as 1,075 persons with 350 service connections.

Past planning efforts for the City of Lowell water system include:

- "Water System Analysis, City of Lowell, Oregon", prepared by Westech Engineering, Inc., August 1989
- "City of Lowell Water System Master Plan", prepared by Systems West Engineers, Inc., November 1998

A significant rehabilitation/upgrade to the Lowell water treatment plant (WTP), designed in 2000 and initiated in 2001, is described in:

- "City of Lowell Water Treatment Plant Rehabilitation", 50 engineering-size sheets of design plans prepared by Lee Engineering, Inc., February 2000

The WTP improvements were implemented by 2002 and included raw water inflow from Dexter Lake via an intake near the covered bridge on the county causeway. Furthermore, a study was conducted in 2001 to determine a suitable location and other requirements for an additional "high-level" storage reservoir that would service properties at 870 to 980 ft in elevation within the UGB of Lowell:

- "Water Management and Conservation Plan [for the City of Lowell]", prepared by Systems West Engineers, Inc., September 2001

However, an additional reservoir has not yet been constructed.

Since the completion of the last water system master plan in 1998, significant changes in drinking water standards required by the Safe Drinking Water Act (SDWA) have occurred, including the:

- Long-Term Enhanced Surface Water Treatment Rule (Stages 1 and 2)
- Disinfectants and Disinfection Byproducts Rule (Stages 1 and 2)

In addition, the Stage 2 implementation of both rules will occur during the planning period of this study. These requirements were not addressed in previous planning efforts. As a result, a master plan update is needed to ensure that the City of Lowell is properly prepared for eventual compliance with these rules.

Furthermore, given the recent housing development activity in the study area, it seems apparent that the capability of the existing WTP (operating with only two filter beds) and single storage reservoir will be exceeded in the near future. It should be mentioned that the 1998 study was greatly influenced by an almost exclusive reliance upon ground water sources, as well as a principal recommendation that did not materialize (installation of a new modular, packaged WTP). Consequently, the projections for the design life of the system proposed in the 1998 study are invalid.

Population growth in Lowell has been slower than was predicted, but recent growth has been substantial. It is estimated that the current population is approximately 387 persons less than was projected in the last master plan; nevertheless, ongoing and emerging usages have stretched the existing WTP capabilities to the limit during periods of peak usage. An update to the water system master plan is needed on growth projections in order to ensure prudent and economical planning. Also, updated guidance is desired so that city infrastructure improvements targeted at development are congruent with long-term community needs and goals.

Finally, available water system mapping is incomplete and does not document recent changes that have occurred. Updated system mapping is desirable along with new analysis to reflect recent growth patterns and updated recommendations for system improvements.

4.2 City Water System History

Over the years, the city has endeavored to secure and maintain a reliable supply of clean and safe water. This situation is primarily due to the natural geology and ecosystems of the area, as well as the impact of previous efforts to utilize the water sources available to the city.

When the city was formed, it assumed operation of two wells from the Army Corps of Engineers. Later, about 1958, a short intake line was laid in Dexter Reservoir, and water drawn from the reservoir via this intake received minimal treatment before its delivery to consumers. In 1969, construction of a water treatment plant was initiated. However, the original design criteria and plans for that plant apparently cannot be found. Estimates of its production capacity range from 100 to 300 gpm.

When the plant began operation, records seem to indicate that the treated water produced either met or exceeded standards existing at the time. However, the plant evidently failed to consistently produce palatable water, which was reflected in recurring customer complaints about water quality. As a result, the city arranged for a well to be drilled (Well #1), which was completed in 1971 and reached a depth of 190 ft. Another well (Well #2) was added in 1978 and reached a depth of 205 ft.

Then, in 1979, the city proposed and scheduled various modifications to the plant and its intake, but many of these improvements were not implemented. A modified intake was installed in 1980. However, the city ceased plant operation shortly thereafter, apparently the result of continuing difficulties with efforts to achieve acceptable water quality. Fortunately, the well-based system provided an adequate supply of water, in terms of both capacity and quality, during the 1980s.

During the 1990s, an increased demand for water rendered the city wells unable to maintain a full water storage tank, whose presence is crucial to the proper operation of a well-based system as well as to ensure adequate fire flows. As a result, usage restrictions were imposed on customers, and a moratorium on new service connections was initiated.

It was originally believed that supply line leaks were the primary cause of the increased demand, and that these leaks prevented the wells from recovering after high-demand periods, thereby limiting their ability to nominally maintain full service levels in the storage tank. Subsequent ground water investigations and a grant from the Rural Investment Fund eventually yielded a third well (Well #3), but operation of this well interfered with the performance of the other two wells, so that only a minimal improvement to the overall production capacity of the system was achieved. Furthermore, a new concern emerged after the detection of arsenic in the ground water of this system.

The combined effect of these problems eventually led the city to reconsider upgrading the dormant WTP and bringing it back online. As mentioned above, significant improvements to this plant were initiated in 2001. Much of the city water system history is documented in reports which are referenced in Section 2 of the 1998 water system master plan.

4.3 City Water Rights

Currently, the City of Lowell utilizes Dexter Lake (or Reservoir) as its primary source of raw water. Two wells, which served to supply all water until 2001, remain as backup sources in the event that the Dexter Lake supply becomes compromised in any way. The water right to another well (Well #2), which did not function adequately, has been cancelled by the Oregon Water Resources Department (WRD).

Data on the water rights (both past and present) for the City of Lowell are available from the database of the Oregon WRD, and are summarized below:

Table 4.3-1 – City of Lowell Water Rights

Source Type	Applic. No.	Permit No.	Certif. No.	Max. Flow Rate (cfs)	Priority Date
Ground (Well #1)	G05520	G05408	46884	0.45	05/19/1971
Ground (Well #2)	G08999	G08386	Not Issued	0.45	11/06/1978
Ground (Well #3)	G14204	G13499	Not Issued	0.45	11/20/1995
Surface (Dexter Lake)	S30077	S23705	23721	1.00	06/20/1955

Source: Oregon Water Resources Department – Ground and Surface Water Rights Records

Note: The water right to Well #2 was cancelled, effective 03/01/1983, as recorded by Special Order of the Oregon WRD Director (volume 37, pages 85–87).

4.4 Treatment Plant Facilities

The Lowell WTP is a conventional-filter plant. The basic plant processes include chemical coagulation, mechanical flocculation, tube-settler sedimentation, dual-media filtration, and chemical disinfection and conditioning. The major system components that accomplish these processes are described in more detail below.

Instrumentation at the plant includes an array of turbidity meters, pH sensors, flow meters, a chlorine analyzer, and other equipment. All data from these instruments is collected and displayed on a central computer equipped with a supervisory control and data acquisition (SCADA) system, which receives and processes data from the entire water system. When operating normally, the plant processes 163 gpm of raw water with turbidities typically ranging from 2–5 NTU throughout the year.

Raw Water Supply, Intake, and Pumping

The source of water supply for the City of Lowell is Dexter Lake (or Reservoir). This lake is actually a broadened portion of the middle fork of the Willamette River, which is a major tributary for the Columbia River. The existing intake from Dexter Lake is located near the covered bridge on the county causeway.

In 1999, a letter was sent from the Army Corps of Engineers to Lee Engineering, Inc. (the designers of the Lowell WTP rehabilitation/upgrade in 2001), specifically indicating that the intake structure was 2 ft wide and deep, and 3 ft high, and that it was securely fastened at a depth of 15 ft to the center pier on the east (upstream) side of the bridge from the city to State Highway 58. The intake is screened on three sides, each side being 36 in by 23 in, with a total screen area of approximately 17 sq ft.

Other documents and the WTP operator state that the intake line is about 2500 ft in length and comprised of 10-in diameter Schedule 80 PVC pipe. The intake screen, which was replaced in 2005, is fabricated from stainless steel (of 3-mesh 14-gauge construction) and is annually inspected. The screen openings size is currently unknown, and there are no systems in place to automatically or remotely clear the screen of accumulated debris.

There are two intake pumps located in the raw water supply room of the WTP. Only one of these pumps (the newer pump) is currently utilized to draw water from Dexter Lake. The older pump, existing prior to 2001, has a 400 gallon per minute (gpm) capacity at 20-ft total dynamic head (TDH) and is driven by a 7.5 horsepower (HP) motor. This pump sits idle because of problems with maintaining a proper seal during operation. The newer pump, added after the plant upgrade, has a 200 gpm capacity at 37-ft TDH and is driven by a 2.5 HP motor. Both of these pumps are of centrifugal type.

Note: Both the intake and distribution pump performance characteristics reported herein are based upon stated specifications taken from the "City of Lowell Water Treatment Plant Rehabilitation" design plans, since documentation of engineering analyses for the rehabilitation/upgrade were not available.

Chemical Pre-Treatment and Rapid Mix

After being discharged from the supply pump, the raw water enters a 6-in chemical injection header. Originally, the upgraded WTP was designed and built with the capability to introduce several chemical conditioners to pre-treat the raw water for coagulation, flocculation, disinfection, and eventual filtration. These potential conditioners included:

- Various Polymer Coagulants
- Liquid Alum – $Al_2(SO4)_3$
- Soda Ash – Na_2CO_3
- Potassium Permanganate – $KMnO_4$
- Gaseous Chlorine

However, it was later learned that a different set of chemical conditioners could accomplish the required pre-treatment, and that potassium permanganate (a strong oxidant utilized to alleviate the presence of algae from the lake source) potentially results in operational difficulties, including the creation of "pink" water.

Pre-chlorination was also discontinued because of the relatively-high total organic carbon (TOC) levels in the raw water and concerns about the formation of potentially-harmful disinfection byproducts (DPBs).

Currently, the only chemical conditioners utilized for pre-treatment are:

- Polyaluminum Chloride Sulfate (SternPAC™)
- Powdered Activated Carbon

The SternPAC polymer serves as a coagulant to aid in the flocculation process. It has little impact upon pH levels and offers effective performance in cold water. The powdered activated carbon is utilized only during the summer months to improve taste and odor related to the presence of algae.

Following the chemical conditioning processes, the pre-treated water then enters a 6-in static mixer tube for the rapid mix process.

Slow Mix and Solids Contact Clarification

After exiting the static mixer tube, the pre-treated water leaves the supply pump room and is delivered into a slow mix chamber for mechanical stirring at approximately 50 rpm in order to further enhance flocculation. This chamber has a hopper-like shape that, due to gravity and the stirring action, funnels the accumulated floc downward to pass beneath a redwood baffle that essentially separates the chamber from the clarifier tank. The passage below the baffle occurs through a restricted opening that serves to partially entrap the accumulated floc and thereby forms a “sludge blanket”, which extends into the clarifier tank and achieves a certain “straining effect” as the pre-treated water flows through it. Except for removal due to annual inspections or blanket thickness maintenance, this blanket should remain intact for optimal clarification (hence, the designation *solids contact* clarifier).

As the pre-treated water travels on into the clarifier tank, natural uplift (in a finite tank) causes the water to move upward through tube settlers, promoting sedimentation of heavier impurities onto the tubes. At this stage, the topmost layer of water in the tank is the purest. This water is then “skimmed off” by means of fiberglass-reinforced plastic (FRP) weir plates and is conveyed to the filter cells by means of collection troughs. The weir plates are adjustable and have a typical 90° “V-notch” design.

There is about 217 sq ft of tube settlers and about 45 ft of FRP weir plates (three collection troughs with two weir plates mounted lengthwise along the sides of each trough, each weir plate being 7.5 ft long). Like the slow mix chamber, the clarifier tank has a hopper-like shape. The length of this tank is about 25.5 ft and the cross section of this shape is about 104.1 sq ft, yielding a volume of approximately 2655 cu ft for the clarifier tank. As a result, for a nominal treated water flow rate of 163 gpm, the theoretical clarifier detention time is approximately 2 hours.

Filtration and Backwashing

As water enters the top of the filter bed, gravity pulls the fluid down through the filter media. Larger floc particles are caught higher up by the coarser media grains. Smaller floc particles are caught lower down by the finer media grains. Over time, the effectiveness of the filter will diminish due to trapped particles within the media. As a result, the filter media must be periodically cleansed of these impurities, and this cleansing is accomplished through backwashing, during which the water flow direction through the filter is reversed. This flow reversal causes the layers of media to expand, enabling the grains to rub against each other and thereby scour the filter. The particles released from the filter media during backwashing are typically flushed into an outflow system and transported to a sludge containment area or backwash settling tank. Surface washing of the filter beds is also accomplished during backwashing by means of rotating sprinklers that spray water downward from above the filter beds.

Although three filter cells are available, only two of these cells contain filter media and underdrains. Each bed has a filtration area of 57.5 sq ft and is rated for 3.5 gpm per sq ft, yielding a filtration capacity of about 200 gpm per filter. Both WTP records and communications with the operator have revealed that the plant is ordinarily operated utilizing only a single filter bed at a time and at a nominal flow rate of 163 gpm. The alternating duty cycles for the filters are dictated by the backwash intervals, which occur every two to three days. The WTP records also indicate that the filter performance target of 0.3 NTU or less (a measure of turbidity of the finish water) is consistently and easily met.

Each filter bed is of the dual-media-type and sits upon a Leopold Universal® Type SL™ underdrain with an Integral Media Support® (IMS) Cap, an effective filter system configuration which has been widely adopted in modern conventional WTP designs. The upper and lower media layers consist of anthracite coal and a sand mixture, respectively. These layers directly rest upon the IMS Cap, which is simply a porous plate fabricated of high density polyethylene (HDPE) beads sintered together. This arrangement eliminates the need for supporting gravel, thereby allowing for deeper media depth within the filter cell.

When the filter beds have been backwashed, it is prudent to temporarily divert water to waste disposal immediately after forward filter operation is resumed (a practice known as filter-to-waste). The reason for this practice is that a filter bed may retain loose impurities immediately following a backwash, and so contaminants then could be carried into the clearwell. A brief period of filter-to-waste operation ensures that the water exiting the filter is sufficiently clarified before entering the clearwell and ultimately the distribution system.

The backwashing and filter-to-waste processes at the Lowell WTP are manually accomplished utilizing existing system pressure along with pipes and valves in the piping gallery. A single filter is backwashed at a flow rate of 1,000 gpm for about 8 minutes. This much larger flow rate is achieved with finish water externally supplied by the storage reservoir. During the filter-to-waste operation, the 163 gpm flow rate is resumed, but the duration of this operation can vary considerably, from 6 to 45 minutes depending on turbidity levels.

Disinfection and Chemical Post-Treatment

After being filtered, the water is disinfected with chlorine by injection in gaseous form (although capable, the Lowell WTP does not utilize pre-treatment chlorine injection). The chlorine is introduced by means of a Wallace & Tiernan V-100 chlorinator coupled to a single 150-lb gas cylinder and a rotameter rated to feed up to 4 ppd. A Hach CL17 free-residual chlorine analyzer monitors chlorine residuals, with samples continuously taken from the outlet pipe of the clearwell. Typically, a chlorine residual of 0.80 mg/L is maintained in the clearwell with a gaseous chlorine feed of 2 ppd.

The chlorine contact chamber (clearwell) is a concrete tank structure located under the plant building. The original clearwell was modified in 2001 to include baffling at the inlet as well as minor intermediate baffling to promote a serpentine flow path. The water depth in the clearwell varies between the high and low-float switches, currently with a high depth of 6-ft 2-in and a low depth of 3-ft 6-in. Because of the presence of an angled wall separating the clarifier tank from the clearwell, the geometry of the clearwell is irregular, which results in a more complicated relation between the water depth and the volume of the clearwell occupied by water. This relation is given by

$$V = (71 \times d) - (0.1042 \times d^2) - 84.15$$

where V is the volume measured in cubic feet and d is the water depth measured in inches. At a water depth of 74 inches, the volume of water contained in the clearwell is 4,599 cu ft (34,402 gal). Also, this relation is only valid for water depths corresponding to $48 \leq d \leq 120$ in.

A chlorine contact time (CT) tracer study was conducted in 2003, and the available chlorine CT was determined to be approximately 57 minutes (prior to reaching the first customer).

The Lowell WTP is configured for post-treatment injection of the filtered water with soda ash (Na_2CO_3) for pH control to prevent corrosion within the plumbing of Lowell water system customers. However, the finish water produced under ordinary plant operating conditions essentially possesses a neutral pH, and so pH adjustment is typically not required.

4.5 Distribution and Storage System

Finish Water Pumping

As water leaves the clearwell, it is pumped by means of either one of two available centrifugal pumps. Only one of these pumps (the primary pump) is operated at a time while the other pump (the secondary pump) is on standby should primary pump replacement become necessary. Each of these pumps has a 130 gpm capacity at 246-ft TDH and is driven by 25 HP motor. (**Note:** These values reflect the design parameters only. Results obtained from a plant flow test, which is described at the end of this section, revealed that the actual flow rate for the primary pump was about 177 gpm).

During routine operation, the primary pump discharges to a nominal system pressure of 107 psi and operates at a flat portion of the pump performance curve with about a 55% efficiency. The pump duty roles are alternated daily, and the total flow readings for the primary pump are recorded daily, but there appears to be a calibration error since a significant discrepancy exists between the total flows indicated for this pump and the active filter.

Distribution System

Table 4.5-1 provides approximate footages for various sizes of pipe employed in the distribution system. About one-half of the system consists of 6-in asbestos cement (AC) pipe installed when the system was originally constructed. Other pipe materials in the system include plastic (PVC), steel, and ductile iron. Significant sections of 12-in PVC pipe, conforming to the ANSI/AWWA C900-97 standard, have been installed with the storage reservoir and the industrial park.

Table 4.5-1 – Lowell Transmission/Distribution Line Inventory

Pipe Diameter (in)	Pipe Length (ft)	% of System
12	6,030	20.8
10	2,150	7.4
8	1,000	3.4
6	14,760	50.9
4	2,740	9.4
2	2,340	8.1
Total	**29,020**	**100.0**

Source: 1998 Water System Master Plan (City of Lowell) and Updated (2006) Distribution System Map

At present, traditional transmission or booster pump stations are not necessary for the distribution system. However, there are several small pumps which operate in a lead/lag manner to lift water from the single storage reservoir to a temporary 2,500 gallon tank which was installed for a recent housing development.

A map of the distribution system is provided in Figure 4.5-1. This map was created from recent digitized aerial photographs, and the distribution lines indicated are based upon information provided in the 1998 water system master plan for Lowell.

Figure 4.5-1 – Distribution System Map

Finish Water Storage System

The Lowell water system employs a single storage tank that replaced a previous redwood tank in 1992. The existing storage tank is a 500,000-gallon concrete reservoir of steel-reinforced construction, which is situated to provide a high-water-surface elevation of 953 ft (the top of the reservoir is at 954 ft). The tank has a cylindrical shape, a height of 32 ft, an outside diameter of 54 ft, a wall thickness of 16 in, and can accommodate a maximum water depth of 30 ft. From this information, it is deduced that the base of the tank is situated at an elevation of 922 ft.

With appropriately-sized distribution pipes, a reservoir at this elevation should provide adequate service from an elevation of 870 ft (approximately 35 psi) down to the level of Dexter Lake, which is nominally taken to be 690 ft (approximately 114 psi). It is apparent that the homes along the lake experience high water pressures, necessitating pressure-control equipment at these service connections.

The UGB for the city allows for the development of areas at elevations up to 1200 ft, though it is likely that an elevation of 1000 ft will not be exceeded. Consequently, a new reservoir should be contemplated to service these higher elevations. A reservoir situated at 1060 ft could service homes between 870 and 980 ft (approximately 82 and 35 psi, respectively). A reservoir situated at 1200 ft could service homes within a larger range of elevations but may necessitate utilization of pressure control valves for homes at elevations close to 870 ft.

Besides ensuring adequate supply during peak consumer demand periods, another important function of storage reservoirs is to provide adequate supply for fire protection of structures and properties within the community serviced by the water system. The Uniform Fire Code requires that residential developments have the capability of providing a minimum of 1,000 gpm for two hours (120,000 gallons). In the case of larger facilities, such as public schools or business establishments, higher "fire flows" are required. A fire-contingency storage requirement of 360,000 gallons was determined in previous studies and seems to be a reasonable amount for the City of Lowell. As a result, the existing reservoir should be maintained at 72% of full in order to ensure sufficient reserves for fire protection. This condition may not be realistic, depending upon usage patterns during peak consumer demand periods.

Finally, it should be mentioned that a relatively-small 2,500-gallon tank was installed uphill of the larger storage reservoir to service homes in a new development (known as the Sunridge subdivision). Although it assists in maintaining adequate water pressures, this tank is completely inadequate for ensuring proper fire protection for this development.

Water Treatment Plant Flow Test

On September 11, 2006, HBH Consulting Engineers personnel conducted two tests at the Lowell WTP in order to assess the accuracy of several plant instruments and to calculate the flow rates of the raw water intake pump and the finish water distribution pump.

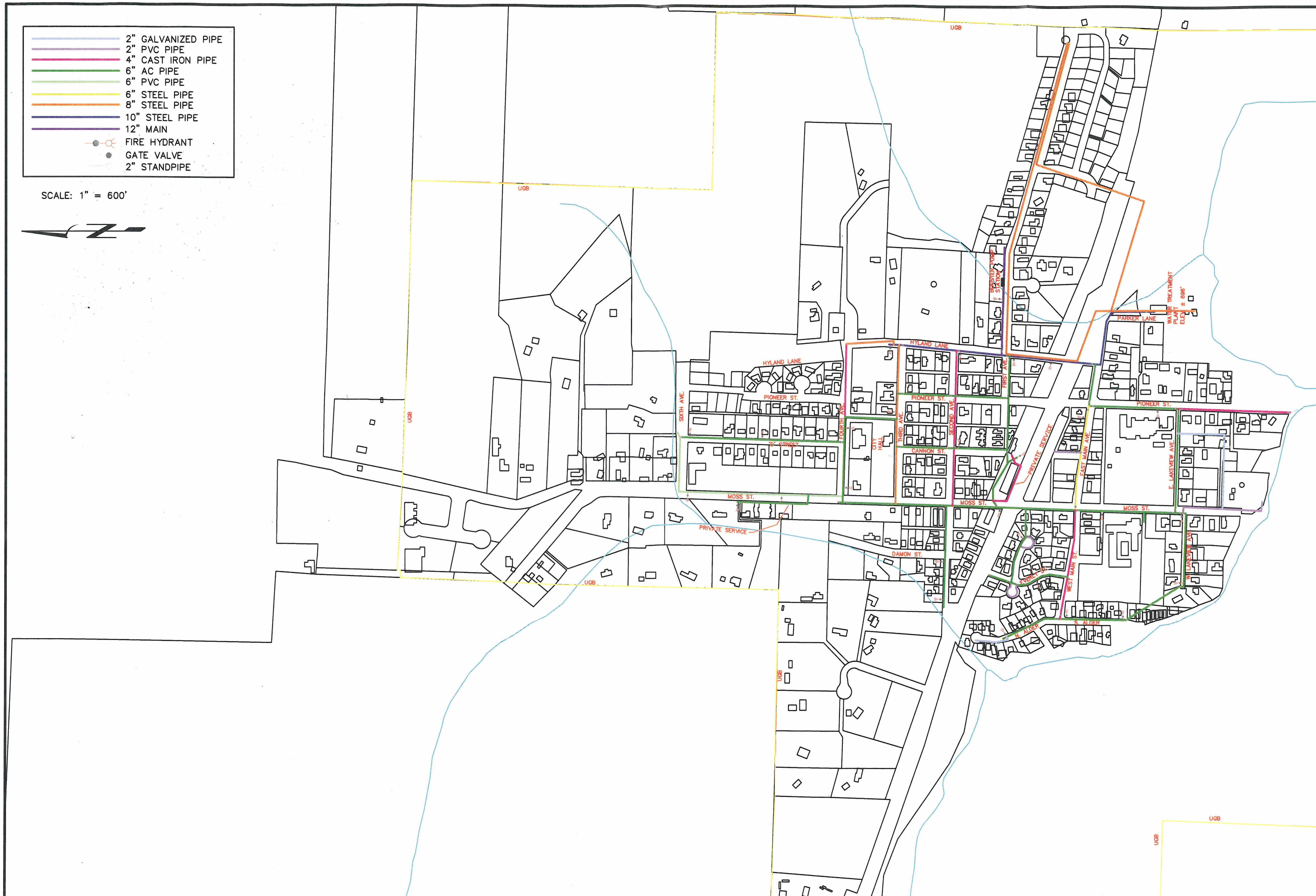
The first test involved filling the clearwell by running only the intake pump (with distribution pump off). This test had a duration of 47.25 minutes, during which time the clearwell depth increased from an initial value of 57.48 in to a final value of 75.48 in. These depths were measured by direct observation in the clearwell. It was then determined (by means of the clearwell volume-depth relation given above) that a total of 7,694 gallons of water was introduced into the clearwell during this test, yielding an average flow rate of 162.8 gpm. The filter effluent flow meter indicated that a total of 7,800 gallons was processed, which is in good agreement with the actual value (7,694 gallons), and represents an error of only 1.38%. For this reason, the plant output will be based upon the filter meters (Filters #1 and #2 are equipped with identical flow meters).

The raw water flow meter indicated a total of 6,957 gallons was introduced, and the liquid level sensor in the clearwell indicated that the depth rose from 53.6 in to 75.3 in. It is apparent that these indications are erroneous.

The next test involved draining the clearwell by running only the distribution pump (with intake pump off). This test had a duration of 58.03 minutes, during which time the clearwell depth decreased from an initial value of 75.48 in to a final value of 51.72 in. It was then determined that a total of 10,263 gallons of water was removed from the clearwell during this test, yielding an average flow rate of 176.8 gpm. It should be explicitly mentioned that the disparity between the intake and distribution pump flow rates (i.e., 163 versus 177 gpm) was confirmed by observation of a gradual but steady drop in the mean depth of the clearwell over several hours of normal plant operation.

The finish water flow meter indicated a total of 7336.8 gallons was removed, and the liquid level sensor in the clearwell indicated that the depth fell from 75.3 in to 47.2 in. It is apparent that these indications are considerably erroneous.

The finish water flow meter is a Model “6-in A HVT-CI” venturi-type flow meter, manufactured by Primary Flow Signal, Inc., which has 6-in inlet and outlet diameters and a 3-in throat diameter. In the operating instructions for this instrument, it is specified that a calibration factor must be properly set in order for the SCADA readout value to be accurate. Otherwise, this type of flow meter is generally accurate and very reliable.



H B H
Consulting Engineers

City of Lowell
WATER SYSTEM MASTER PLAN UPDATE

DISTRIBUTION SYSTEM MAP

DRAWN BY: JMM
DATE: 9/8/2006

FIG.
4.5-1

Section

5

Water Demands

5.1 Current Water Demands

Definitions

Water demand is an amount of water usage (over a time period) which is required to meet the needs of consumers and to supply the needs for fire fighting and system flushing efforts. Additionally, nearly all water systems exhibit some degree of leakage which, for economic as well as practical reasons, cannot be completely eliminated. Consequently, the difference between the recorded amount of water received by customers (i.e., "metered usage" or "water sold") and the amount of water delivered to the distribution system is attributable to leakages, system flushing, filter backwashing, fire fighting, and possibly other non-metered usages. Furthermore, meter indicator inaccuracies or misreadings also may occur.

In accordance with consumer usage patterns, water demand varies with time, and the time periods most commonly considered are annual and diurnal. Demand varies relative to time-of-year, with lower usages during winter months and higher usages during summer months. Demand varies relative to time-of-day, with higher usages during early morning/evening hours — when meals are prepared, showers are taken, and clothes are washed — and lower usages during nighttime hours.

The objectives of this section are to: (1) determine current demands; and (2) project future demands that will either confirm or deny the adequacy of existing system components, and possibly establish "sizes" for system components associated with a potential system upgrade. The following terms are employed to describe various water demand measures:

- Total Annual Production (TAP) – Total volume of treated water produced for a one-year period, expressed in gallons.
- Average Annual Demand (AAD) – Average value of TAP over several years, expressed in gallons. Same as TAP if only one year of data is available.
- Average Daily Demand (ADD) – Value of TAP divided by the #(days) for the year, expressed in gallons per day (gpd). Represents the average water usage per day during that year.
- Max. Monthly Production (MMP) – Largest total volume of treated water produced for a one-month period, expressed in gallons.
- Max. Monthly Demand (MMD) – Value of MMP divided by the #(days) for the month, expressed in gallons per day (gpd).
- Max. Daily Demand (MDD) – Largest total volume of treated water produced for a one-day period, expressed in gallons per day (gpd).

Certain water demand measures can be normalized by dividing by (a) the #(persons) in the residential population or (b) the total system EDU-value in order to express demands on the basis of (a) per-person or (b) per-EDU. These normalized demands can be multiplied by projected numbers for these quantities in order to assess future water demands.

Raw Water Acquisition and Finish Water Production

The volume of raw water acquired from the source (Dexter Lake) and pumped into the WTP will differ from the volume of finish water pumped into the distribution system, the difference resulting from such effects as filter-to-waste usages, occasional partial draining of the clarifier tank (for maintenance of the sludge blanket thickness), usages for various instrumentation, and possibly flow meter inaccuracies (recall that backwashing is accomplished with finish water). It is worthwhile to evaluate and compare these two volumes so that a sense of plant operation usages can be gained.

Records of water production were obtained from the WTP operator for January 2003 through June 2006. During normal operation, the WTP produces approximately 163 gpm of water. Characteristics of water production from the data contained in the records for complete years (2003–2005) are provided below:

- AAD: 39,880,090 gallons (per year)
- Average Monthly Production: 3,323,340 gallons
- Minimum Monthly Production: 2,074,240 gallons
- Maximum Monthly Production: 5,676,380 gallons

The WTP must operate about 14 hours per day on average during the year to meet production demands. For the summer season, when peak production demands occur, more than 5,500,000 gallons are needed each month and the WTP may operate close to 24 hours per day.

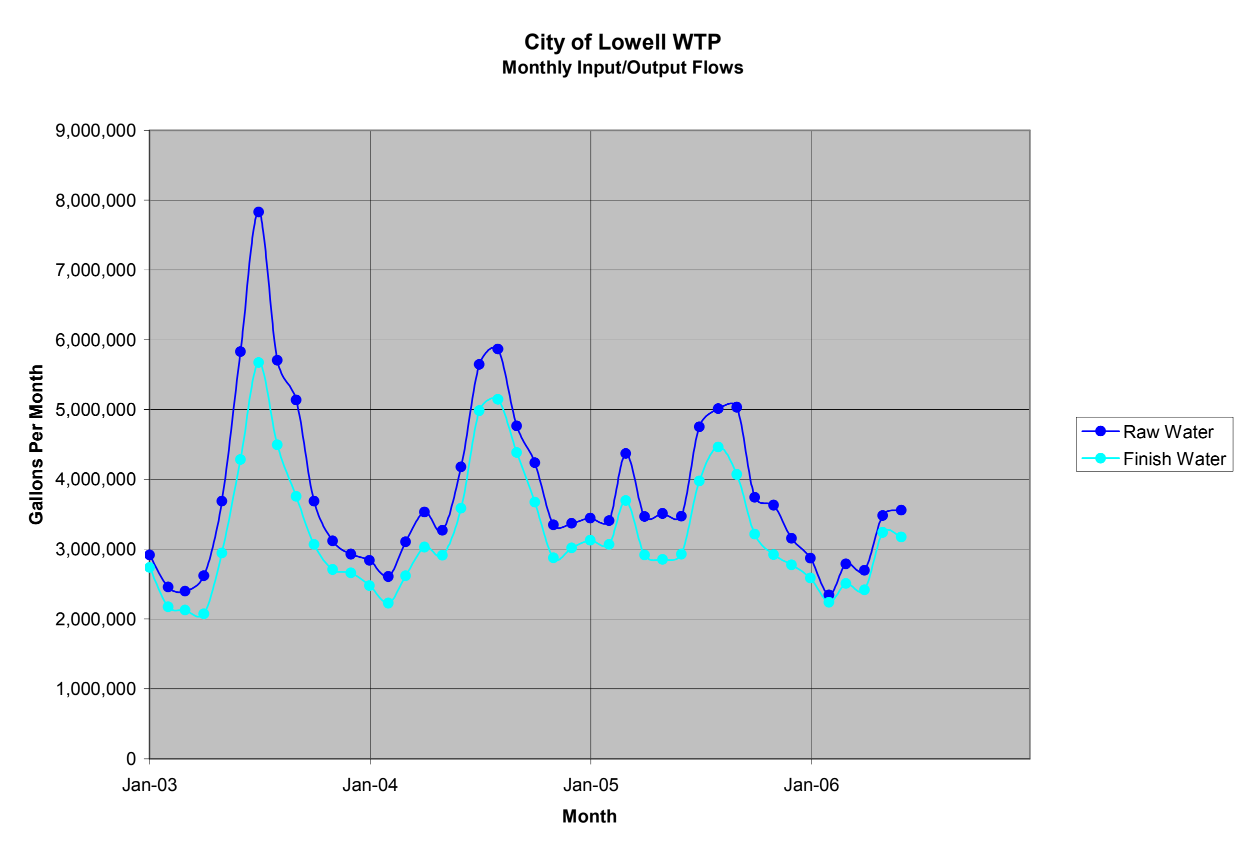


Figure 5.1-1 – Lowell WTP Monthly Production

The largest plant operation usages correspond to: (1) the filter backwash process, which occurs every 2 to 3 days from May through October, or every 3 to 4 days from November through April; and (2) the filter-to-waste process, which occurs when the plant is started after a period of non-operation, and immediately after each backwash cycle. Nominally, 8,000 gallons is used during the backwash process (supplied by finish water fed from the storage reservoir). The amount of usage for the filter-to-waste process (supplied by pre-treated water taken from within the plant) is difficult to estimate since usage varies considerably, depending on measured turbidity levels.

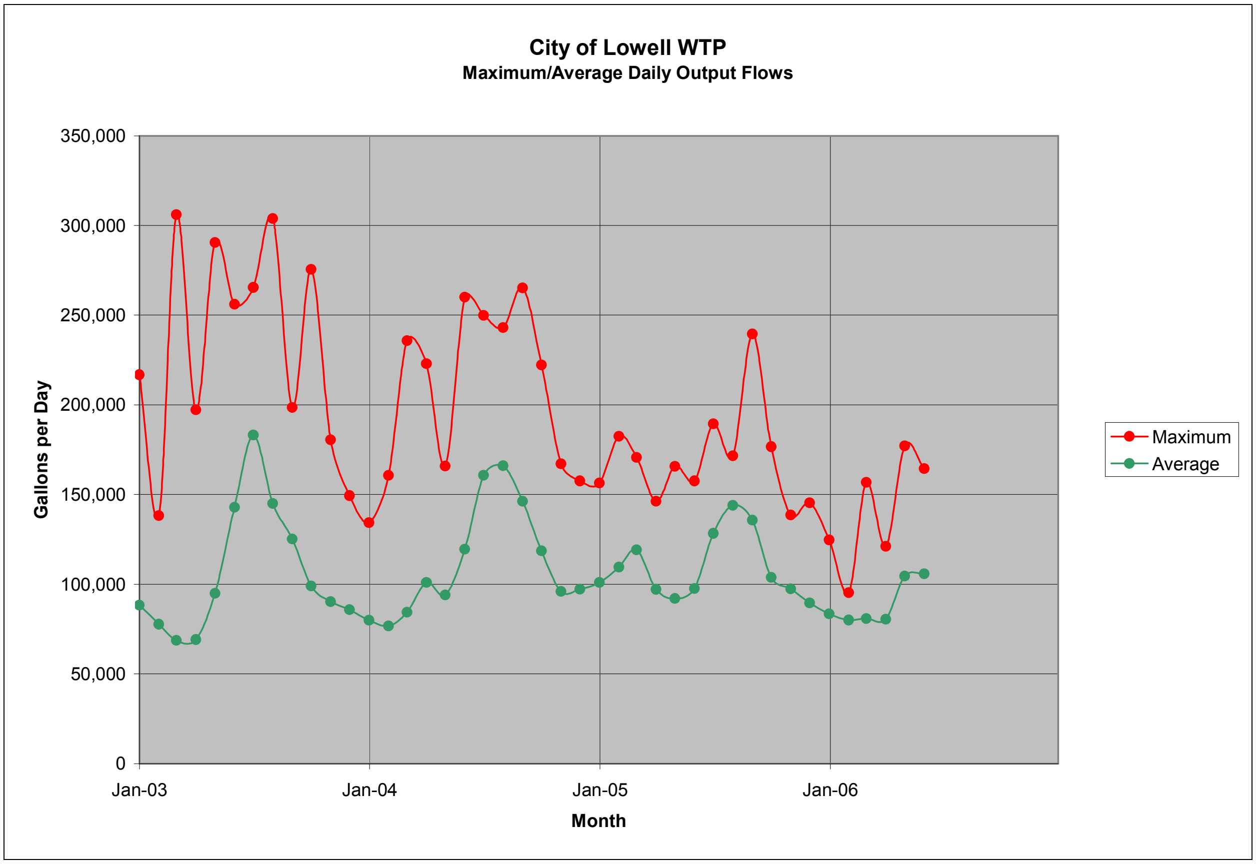


Figure 5.1-2 – Lowell WTP Max. and Ave. Daily Production

Table 5.1-1 – Lowell WTP Production Summary

Year	TAP (gal)	MMP (gal)	MMD (gpd)	ADD (gpd)	MDD (gpd)	MDD/ADD PF	MMD/ADD PF
2003	38,703,060	5,676,380	183,109	106,036	306,040	2.89	1.73
2004	40,931,230	5,146,340	166,011	111,834	265,260	2.37	1.48
2005	40,005,980	4,462,900	143,965	109,605	239,540	2.19	1.31
2006	16,160,130	3,239,700	104,506	44,274	177,130	4.00	2.36
Average	**39,880,090**	**5,095,207**	**164,362**	**109,158**	**270,280**	**2.48**	**1.51**

Source: 01/2003–06/2006 Lowell Water Production Records

Note: The average values indicated above are based upon 2003–2005 data since the only 2006 records available when this study was conducted were for January 2006 through June 2006.

Based upon the data contained in the records for complete years (2003–2005) and engineering judgment, the following design values are selected for estimating future water demands:

Table 5.1-2 – Design Values for Water Demand Measures (2006)

	AAD	MMP	MMD	ADD	MDD
gal	40,880,000	5,381,600	—	—	—
gpd	—	—	173,600	112,000	308,000
gpm	—	—	121	78	214
gpcd	—	—	176	113	312
gpd/EDU	—	—	461	297	817

The MMD and MDD *peaking factors*, defined as MMD ÷ ADD and MDD ÷ ADD, are then equal to 1.55 and 2.75, respectively.

Water Consumption (Water Sold)

The recorded amount of water received by customers (referred to as the "metered usage" or "water sold") over a representative year — July 2005 through June 2006 — was presented in Table 2.4-3 of this study. It has been repeated below in a slightly different form.

Table 5.1-3 – City Water System Annual Usages and EDU-Values

Consumer Group	Annual Water Usage (gal)	EDUs	% of Usage
Residential Consumers	22,043,680	354.1	94.0
All Non-Res. Consumers	1,416,100	22.7	6.0
Total	**23,459,780**	**376.8**	**100.0**

Source: 07/2005 – 06/2006 Lowell Water Usage Records

Over the 12-month period from July 2005 through June 2006, water usages in Lowell totaled 23,459,780 gallons. For this period, the average monthly usage was 1,954,982 gallons per month, or 64,273 gallons per day. Of the total usage, 94% was for residential consumers. With a residential service population of 988, water usage per capita was 65.0 gallons per person per day (gpcd). The average water usage per equivalent service account was 5,188 gallons per month (or 5,188 gallons per month per EDU).

Unaccounted Water

The difference between the amount of water produced at the WTP (and subsequently pumped into the distribution system) and the amount of water recorded by usage meters (i.e., water sold) is unaccounted water (UW). This difference is the overall result of leakages, filter backwashing, system flushing, fire fighting, or other non-metered usages.

The usage for filter backwashing already has been estimated. Fire fighting usage can be anticipated but is difficult to quantify. This issue was briefly discussed in Section 4 of this study. System flushing usage is necessary to purge sediment and rust that collect in water mains. The presence of such deposits can cause undesirable tastes, odors, and coloration, as well as reduce the effective diameter of a main, inhibiting the flow rate through that main. The mains must be flushed by opening a nearby hydrant, group of hydrants, or a blow-off valve and then operating main line valves to achieve a flow velocity of about 5 fps when possible. Since system flushing is aperiodically conducted, usage quantities are unknown.

It is also possible that many of the older water meters are indicating inaccurately. Such meters tend to indicate usages lower than actual usages, resulting in apparent loss of water. It is also possible that the distribution system has substantial leakage due to the combined effect of numerous small leaks which are not readily detectable.

Table 5.1-4 – Lowell WTP Monthly Water Ledger

Month	Raw Water (gal)	Plant Usage (gal)	Finish Water (gal)	Consumer Usage (gal)	Backwash Usage (gal)	Unaccoun. Water (gal)	Water Loss
Jul-05	4,751,940	775,020	3,976,920	2,346,898	96,000	1,534,022	38.6%
Aug-05	5,012,710	549,810	4,462,900	3,237,896	96,000	1,129,004	25.3%
Sep-05	5,033,430	963,850	4,069,580	2,427,894	96,000	1,545,686	38.0%
Oct-05	3,743,620	528,280	3,215,340	1,885,556	96,000	1,233,784	38.4%
Nov-05	3,629,360	707,510	2,921,850	1,724,348	68,600	1,128,902	38.6%
Dec-05	3,155,460	380,760	2,774,700	1,531,487	68,600	1,174,613	42.3%
Jan-06	2,873,480	288,780	2,584,700	1,641,207	68,600	874,893	33.8%
Feb-06	2,342,430	102,270	2,240,160	1,522,584	68,600	648,976	29.0%
Mar-06	2,791,810	284,530	2,507,280	1,470,640	68,600	968,040	38.6%
Apr-06	2,695,470	280,080	2,415,390	1,600,026	68,600	746,764	30.9%
May-06	3,481,010	241,310	3,239,700	2,058,883	96,000	1,084,817	33.5%
Jun-06	3,558,550	385,650	3,172,900	2,012,360	96,000	1,064,540	33.6%
Total	**43,069,270**	**5,487,850**	**37,581,420**	**23,459,780**	**987,600**	**13,134,040**	

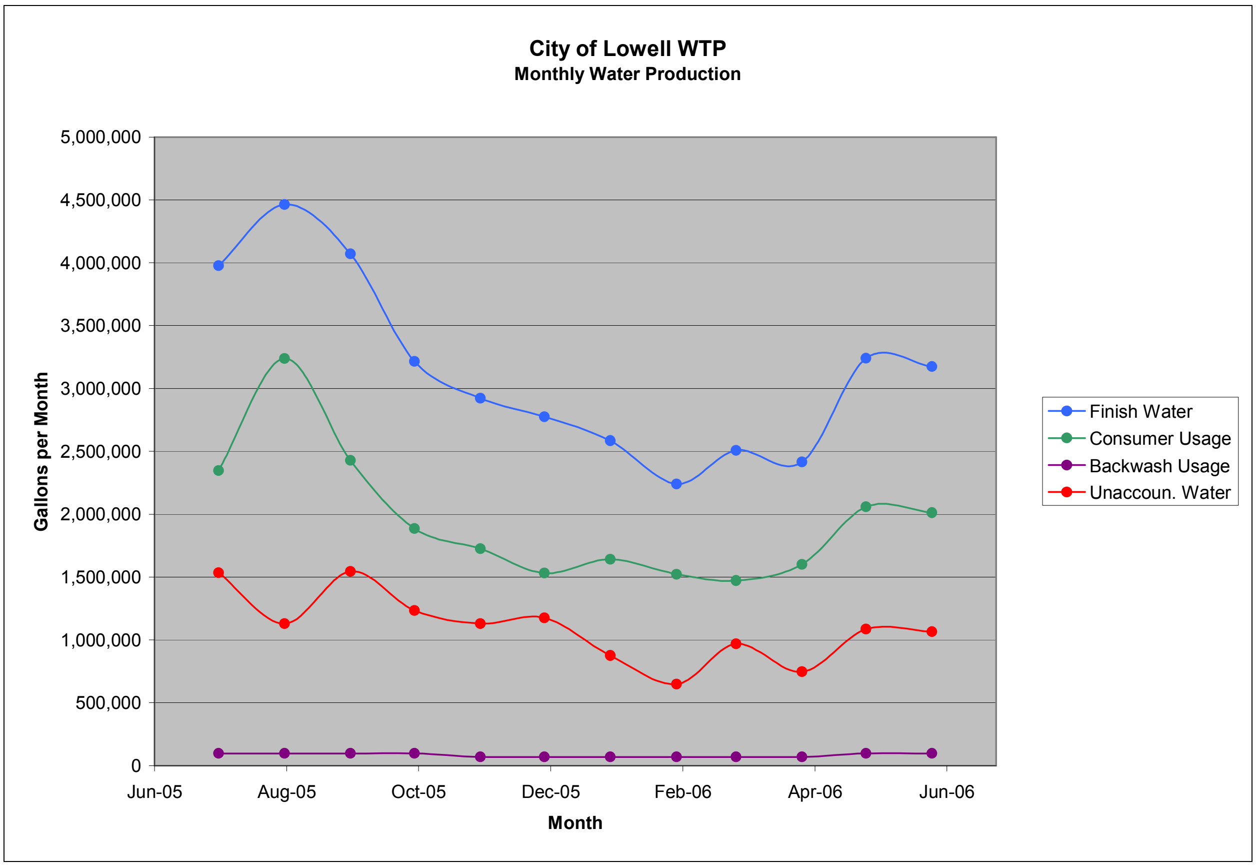


Figure 5.1-3 – Lowell WTP Monthly Water Ledger

Over the 12-month period from July 2005 through June 2006, the UW in Lowell ranged from 25% to 42%. During this period, a total of 37,581,420 gallons was produced by the WTP and pumped into the distribution system. Of this amount, 24,447,380 gallons was utilized, leaving 13,134,040 gallons of UW, or 35% of the finish water.

These figures indicate that the Lowell water distribution system exhibits a significant amount of leakage. Systems often manifest UW exceeding 20% of the finish water. In general, systems with excessive UW should strive for a reduction to less than 15%. In turn, systems with UW already less than 15% — or systems for which achieving 15% is readily attainable — should pursue a reduction to less than 10%, per OAR 690-086-0150 (6a).

Furthermore, the plant usages indicated above (which are simply the differences between the raw water and finish water amounts) seem inordinately high. As previously mentioned, the amount of pre-treated water utilized for the filter-to-waste process — the only substantial cause of internal plant usage — is difficult to estimate. However, based the results of the WTP flow test conducted on September 11, 2006, it is almost certain that the metered amount of raw water intake is very inaccurate.

5.2 Projected Water Demands

Normal Water Demand in Oregon

Typical values for metered usage, or water sold, can be found in the "Guidelines for the Preparation of Planning Documents for Developing Community Water System Projects", prepared by State of Oregon, Federal Government, and various non-profit organizations. This document affirms that normal water usage should be based upon 100 gpcd, or 250 gpd/EDU, or 7,500 gallons per month per EDU. These numbers are for metered usage rather than actual demand, which always exceeds metered usage. As was previously indicated, the metered usage for Lowell during the one-year period from July 2005 through June 2006 was 65.0 gpcd, while 5,188 gallons per month was consumed per equivalent service account.

Per capita water usage for Oregon is assessed by the U.S. Department of the Interior and documented in the 2000 U.S. Geological Survey Circular 1268, entitled "Estimated Usage of Water in the United States in 2000". According to this report, the average per capita water demand in Oregon is 165 gpcd for *public supply*, which refers to water produced by both public and private suppliers providing water to at least 25 people or at least 15 service connections. Public-supply water may be utilized for domestic, commercial, industrial, or thermoelectric-power purposes. Public-supply water may be delivered to other suppliers or utilized for the treatment of raw water and wastewater. Public-supply water also services public parks, pools, and facilities. All public-supply provisions in this report are considered freshwater.

Based upon the USGS Circular 1268 data, the average values for the Oregon demand measures are: ADD of 165 gpcd, MMD of 248 gpcd (1.5 × ADD), and MDD of 412 gpcd (2.5 × ADD). But these values are state averages, and the values for a specific community are affected by factors unique to that community and, consequently, can vary considerably.

Current Values of Lowell Water Demand Measures

Note: The "current values" of the water demand measures provided below are actually the *design values* selected for the Lowell WTP based upon water production data contained in the records for 2003–2005. These values, assumed to hold true in 2006, are projected into the future over the 25-year planning period, as detailed below.

Based upon an average daily demand of 112,000 gallons per day and a residential service population of 988 people, the average daily demand per person is 113 gpcd. Alternatively, the average daily demand per equivalent service account is 297 gpd/EDU. Besides average daily demand, maximum monthly and maximum daily demands are also provided in Table 5.2-1 (see Table 5.1-2). Since recent data indicates that 94% of the total metered usage has been for residential consumers, the usages for the non-residential consumers have little effect upon per-capita-based values of demand measures.

Table 5.2-1 – Current Values of Lowell Water Demand Measures

Demand Measure	Value in gal/day	Value in gpcd	Value in gpd/EDU
ADD	112,000	113	297
MMD	173,600	176	461
MDD	308,000	312	817

Based upon 2003–2005 Lowell Water Usage Records and Estimated Service Population.

Projected Values of Lowell Water Demand Measures

Similar to the projections for population and EDU-values in Section 2 of this study (see Table 2.4-3), the water demand measures are projected in Table 5.2-2 based upon the selected design values appearing in Table 5.1-2. The objective of projecting future demands is not to necessarily construct larger facilities to support excessive water consumption, but rather to assess existing facility capabilities, identify immediate deficiencies, recommend performance improvements, and "size" potential new facilities for reasonable future water demands. The current values for normalized water demand measures (gpcd and gpd/EDU) are reasonable in comparison to the values taken from the two references cited above. By projecting the residential population, total system EDU-value, and system water demand measures at the same AAGR, the *normalized* water demand measures are preserved.

Assuming a 3.30% AAGR for the planning period, the current population of 988 people is projected to reach 2,533 people in 25 years. If the proportions of total water usage for the residential and various non-residential consumer groups remain constant over this time period, then the EDU-values will increase at the same growth rate. It is possible that EDU-values could grow faster than the population if significant commercial/industrial development occurs. It is also possible that population growth will not maintain a 3.30% AAGR over this time period. For these reasons, the total system EDU-value at any time is always the best indicator of water needs at that time.

Table 5.2-2 – Projected Values of Lowell Water Demand Measures (3.30% AAGR)

Year	Residential Population	Total System EDUs	ADD (gpd)	MMD (gpd)	MDD (gpd)
2006	988	376.8	112,000	173,600	308,000
2007	1,021	389.2	115,696	179,329	318,164
2008	1,054	402.1	119,514	185,247	328,663
2009	1,089	415.3	123,458	191,360	339,509
2010	1,125	429.1	127,532	197,675	350,713
2011	1,162	443.2	131,741	204,198	362,287
2012	1,200	457.8	136,088	210,936	374,242
2013	1,240	472.9	140,579	217,897	386,592
2014	1,281	488.6	145,218	225,088	399,350
2015	1,323	504.7	150,010	232,516	412,528
2016	1,367	521.3	154,961	240,189	426,142
2017	1,412	538.5	160,074	248,115	440,204
2018	1,459	556.3	165,357	256,303	454,731
2019	1,507	574.7	170,814	264,761	469,737
2020	1,557	593.6	176,450	273,498	485,238
2021	1,608	613.2	182,273	282,523	501,251
2022	1,661	633.5	188,288	291,847	517,793
2023	1,716	654.4	194,502	301,478	534,880
2024	1,772	676.0	200,920	311,426	552,531
2025	1,831	698.3	207,551	321,704	570,764
2026	1,891	721.3	214,400	332,320	589,600
2027	1,954	745.1	221,475	343,286	609,056
2028	2,018	769.7	228,784	354,615	629,155
2029	2,085	795.1	236,334	366,317	649,917
2030	2,154	821.3	244,133	378,405	671,365
2031	2,225	848.4	252,189	390,893	693,520

Water demand measures are crucial for proper analysis and design of existing and future water supply systems. The current and 25-year-projected MDD values, in gallons per minute (gpm), are 214 and 482 gpm, respectively.

Design Criteria & Service Goals

Section 6

6.1 Design Life and Planning Period

In the design of municipal projects and facilities, two time frames exist: design life and planning period. A discussion of these elements in relation to the design of a municipal water system is given below.

The design life of a water system component is the expected time period over which the component will function as planned. The design life also may be referred to as the "useful life" or "service life". The selection of a design life for a particular component is a matter of engineering judgment, based upon such factors as purpose and intensity of operation, quality of fabrication materials, quality of manufacture, quality of installation, and regularity of maintenance. For any component, the design life and actual life can differ, depending on variations in and adherence to these factors. The determination of a design life yields a rational estimate of service duration, upon which a meaningful economic analysis of potential capital improvements can be based.

The planning period is the time frame over which the recommended water system is expected to provide an adequate water supply to meet the needs of all anticipated customers. The required system capacity is based upon population and water-demand projections, as well as land-use considerations. The planning period is affected by the ability and/or desire of a community to finance necessary improvements. The duration of the selected planning period must be short enough for current customers to derive the benefits of system improvements yet long enough to provide reserve capacity for the water demands associated with future growth.

Ordinarily, the planning period for a water system master plan is 20 years. However, recognizing that several years may pass before recommended improvements are implemented and new facilities become operational, the projections will be carried out 25 years, ending in 2031.

The planning period for a water system and the design life for its components may not be identical. For instance, a properly maintained steel storage tank may have a design life of 60 years, but the projected fire flow and consumptive water demand for a planning period of 25 years determines the tank size. At the end of the initial 25-year planning period, water demand may be such that an additional storage tank is required; however, the existing tank with a design life of 60 years still would be useful and could remain in service for another 40 years. Typical design lives for various system components are discussed below.

Structures and Pumping Equipment

Major structures and buildings should have a design life of approximately 50 years. Primary pumps and related mechanical equipment usually have a useful life of 15 to 20 years. When additional capacity is not required, the service life of some equipment can be extended if properly maintained. Flow meters typically have a design life of 10 to 15 years. Valves usually need to be replaced after 15 to 20 years of routine operation.

Transmission and Distribution Piping

Water transmission and distribution piping easily should have a useful life of 40 to 60 years if quality materials and assembly practices are employed during their construction and pipes are adequately sized.

It is common for buried steel piping, extensively utilized during the 1950s and 60s, to exhibit significant corrosion and leakage within 30 years. In contrast, cement-mortar-lined ductile iron piping can last up to 100 years if properly fabricated and installed. Manufacturers of PVC pipe claim a 100-year service life as well.

Water Storage Tanks

Distribution storage tanks should have a design life of 60 years (for painted steel construction) to 80 years (for reinforced-concrete construction). When lined with a glass-fused coating, steel tanks will possess a design life similar to that of concrete tanks. Again, the actual life will depend on materials, manufacture, installation, and maintenance factors. Several practices, such as cathodic protection, regular cleaning, and frequent painting can ensure or extend the service life of steel reservoirs. The useful life of steel tanks is greatly reduced if not periodically repainted as needed.

6.2 Sizing and Capacity Criteria and Goals

The 25-year projected water demands form the basis for sizing system components in the recommended improvements. Various elements of system demands are considered. The methods and demands utilized are discussed below.

Water Supply Source

At the very least, a water supply source should be sufficient to meet the projected maximum daily demand (MDD) of the system on a continual basis over many years of service. If possible, raw water availability should meet the ultimate build-out needs in a small community, especially when surface water rights are the only option. Currently, the MDD is 0.308 Mgd or 0.477 cfs. At the end of the 25-year planning period, the projected MDD is 0.694 Mgd or 1.074 cfs.

The total water supply available to the Lowell water system is 1.90 cfs (1.00 cfs from Dexter Lake, and 0.90 cfs from Wells #1 and #3) or 1.23 Mgd, although it is doubtful that these wells could yield their allotted supplies, based upon remarks provided in the 1998 water system master plan. Nonetheless, the City of Lowell should pursue certification of Well #3 in order to secure the formal water rights to this supply source. The other supply sources are certificated (see Table 4.3-1). With the presence of three lakes nearby, additional surface water rights should be procured for anticipated water demands beyond the 25-year planning period.

Design Goal: 1.074 cfs or 482 gpm over 24 hours(693,520 gpd) for the 25-year planning period.(or 525 gpm for 22 hours)

Water Treatment Plant

Water treatment plant equipment and components such as intake and distribution pumps, and clearwells, are usually sized to provide for the 25-year MDD. The actual plant capacity should be slightly increased to allow for the maximum daily demand to be met without requiring the plant to operate 24 hours per day. This increase is necessary because most plants cannot continuously operate 24 hours per day since filter backwashing and other down-times are required in order to produce safe finish water. The goal is to have the capacity to produce the 25-year MDD within 22 hours or less of continuous operation. As a minimum then, the design value for the 25-year MDD must be set to 0.757 Mgd ($22 \times 0.757 \approx 24 \times 0.694$).

Design Goal: 0.757 Mgd or 525 gpm treatment capacity (based upon only 22 hours of plant operation per day)

Treated Water Storage

The total storage capacity in a distribution system includes reserves for system equalization, emergency protection, and fire suppression. Each of these elements is described below.

- System Equalization Reserve – Accommodates the differences between supply and demand flow rates (which continually occur) in an active water system.
- Emergency Protection Reserve – Accommodates a sudden total loss of water supply due to events such as power outages, broken distribution pipes, plant breakdowns, or source contaminations.
- Fire Suppression Reserve – Accommodates an elevated and sustained water demand caused by fire suppression efforts.

In larger communities, it is common to provide a total storage capacity equal to the sum of the system equalization reserve and the larger of the emergency protection and fire suppression reserves. In smaller communities, especially those which are more isolated, it is recommended that the total storage capacity be the sum of all three reserves. This approach is considered prudent since fire dangers tend to intensify during emergency situations.

The system equalization reserve is typically taken as 20–25% of the MDD in order to balance out the difference between peak demand and supply capacity, and thereby ensure a sufficient allocation for the fulfillment of routine demands. When the peak hourly demand (PHD) is known, the system equalization reserve may be selected as the difference between PHD and MDD multiplied by 8 hours. Unique system equalization requirements occur when storage reservoirs are utilized as a tool to manage a marginal water supply source.

At the very least, the emergency protection reserve should be one MDD, or the ADD multiplied by a factor of 2.5 to 3 (the MDD *peaking factor*).

The fire suppression reserve is based upon the maximum flow rate and the duration of the flow required to confine a major fire. The guidelines provided in the "Fire Suppression Rating Schedule", published by the Insurance Services Office (ISO®), are often utilized to determine the required flow rate and reserve. Typically, flow rates of 1,000 to 1,500 gpm are sufficient for one or two-family dwellings not exceeding two stories in height. In general, business, industrial, or institutional buildings will require higher flows.

The ISO also classifies fire suppression capabilities on a quantitative basis, called the Public Protection Classification (PPC), with Class 1 denoting exemplary protection and Class 10 denoting subminimal protection. The PPC for a particular community is determined from a complex analysis of the capability to receive and respond to fire alarms, the strength of the fire department, and the adequacy of the water supply system. Further analysis of the water supply system is accomplished by considering:

- supply system capabilities
- type, size, and installation of fire hydrants
- operating condition of fire hydrants

For a PPC Class 8 rating or better (i.e., Classes 1–8), fire suppression reserves should support necessary fire fighting duration and flow rates as follows: 2 hours if less than 3,000 gpm; 3 hours if between 3,000 and 3,500 gpm; or 4 hours if greater than 3,500 gpm.

For typical residential areas, the minimum recommended fire suppression reserve is 180,000 gallons in order to provide a fire flow rate of 1,500 gpm for 2 hours. When substantial non-residential structures are present, with fire protection requirements larger than those for residential areas, greater fire suppression reserves are warranted.

Another important design consideration for distribution reservoirs is elevation. All reservoirs within the same pressure zone should be situated at the same elevation whenever practicable. Since a narrow range of water surface levels is maintained within each reservoir, the need for level-control valves, pressure-reduction valves, booster pumps, and other control devices may be reduced or eliminated. Furthermore, the reservoirs should be situated at an elevation that provides acceptable water pressure throughout the system, meaning sufficient pressures at higher elevations but inexcessive pressures at lower elevations. An ideal range of pressures for the system would be 40 to 80 psi.

For subdivisions at elevations higher than allowed within the main pressure zone, booster-pump-supplied storage tanks should be employed rather than hydropneumatic stations (which utilize a pressure vessel and pump configuration). In general, the tank size is determined on a case-by-case basis as part of the design review effort. The minimum tank size should be based upon a 120,000-gallon fire suppression reserve (1,000 gpm for 2 hours) plus a reserve equal to the EDU-normalized MDD multiplied by the total subdivision EDU-value. Fire suppression pumps with a capacity of 1,000 gpm should be available when a storage tank is not present.

For the Lowell water system, the emergency protection reserve should be one 25-year MDD. The system equalization reserve should be 25% of the 25-year MDD (since data for the PHD was not available from the water records, and a hydraulic analysis of the existing distribution system was not conducted). A fire suppression reserve of 360,000 gallons (2,000 gpm for 3 hours) was assumed in previous studies for this community and is acceptable to the local fire chief. Since the value of the 25-year MDD determined for this study is 0.694 Mgd or 693,520 gallons, the total storage capacity of the distribution system over the planning period should be at least 1,226,900 gallons, as detailed in Table 6.2-1 below.

While Table 6.2-1 shows the projected design requirements for storage, it is important to emphasize that an existing reserve shortfall of around 245,000-gallon based on existing water demands.

Design Goal: 1.227 Mgal storage capacity.

Table 6.2-1 – Treated Water Storage Capacity Requirement (projected)

Reserve Type	Description	Reserve Amount (gal)
System Equalization	0.25 × 25-Year MDD	173,380
Emergency Protection	1.00 × 25-Year MDD	693,520
Fire Suppression	2,000 gpm for 3 Hours	360,000
Total		1,226,900

Water Distribution System

Water distribution piping lines, or "mains", are typically sized to accommodate potential fire flows along with projected consumer demands. The mains should be at least six inches in diameter to provide the expected fire flow rates. All pipelines should be large enough to sustain a minimum line pressure of 25 psi during peak flow periods. The State of Oregon requires that a water distribution system be designed and installed to provide a pressure of at least 20 psi to all service connections (at the property line) at all times. The layout and sizes of system pipelines must be adequate to meet peak hourly demands, and to accommodate fire flows during high demand periods while maintaining system pressure.

Water transmission mains should be sized on the basis of a 50 to 60-year planning period, especially if these mains have significant length and would be difficult to repair or replace. In general, transmission mains are sized for the peak fire flow rate plus average daily demand.

In addition to the above design criteria, the following guidelines are recommended for the design of water distribution systems. Ultimately, a hydraulic analysis based upon peak consumer demands and potential fire flows may result in pipe sizes larger than the minimums indicated below:

- **Six-Inch (6") Diameter Lines** – minimum size for lateral mains in a gridiron (looped) system, and for short (less than 250 ft) dead-end mains which are not to be extended.
- **Eight-Inch (8") Diameter Lines** – minimum size for permanently dead-ended mains supplying fire hydrants, and for minor trunk lines.
- **Ten-Inch (10") and Larger Diameter Lines** – required for trunk (feeder) mains as determined by hydraulic analysis.

The lateral mains in a distribution system should be looped whenever possible. A lateral main is defined as a line whose diameter does not exceed eight inches and which is installed to provide water service and fire protection in a localized area of the distribution network. The normal size of lateral mains for single-family residential areas is six inches in diameter. However, lateral mains with diameters of eight inches or larger may be required to meet both domestic and fire protection needs of a particular area.

Both the installation of permanent dead-end mains and the dependence of relatively-large areas on single mains should be avoided. For the placement of a fire hydrant on a permanently dead-ended main, the minimum size for such a main is eight inches in diameter. Six-inch diameter mains may be utilized for a stub-out not exceeding 500 feet in length which is supplying a single fire hydrant not on a public street, and for internal fire protection. For new developments, the minimum size of lateral mains supplying fire hydrants in public ways is six inches in diameter, provided these mains are adequately looped with each other and neighboring mains within the distribution network.

Ordinarily, a computational model of the existing distribution system is created in a water system master plan study. The model is based upon actual pipe sizes, materials, and configurations, as well as the pipe junction and storage tank elevations. This model is utilized to perform a hydraulic analysis of the system by means of commercially-available software programs, such as WaterCAD® by Haestad Methods, Inc. The computational results are subsequently examined to determine whether or not the system is capable of simultaneously providing the necessary fire flow rate (FFR) and 25-year MDD at proper pressures.

The City of Lowell elected to not perform a hydraulic analysis of the existing distribution system for this study. As a result, the fluid-mechanical characteristics of the system has not been quantitatively assessed in a detailed manner.

Typical Goal: A distribution capacity for the peak fire flow rate plus the 25-year MMD (with a residual pressure of at least 20 psi)

Fire Flow Demands

In typical communities, the requirements for fire suppression efforts at any point in the service area can vary between 500 to 12,000 gpm for a single fire event. Multiple fire events will place greater demands upon a distribution system. A municipality is responsible for adequately serving its residents, businesses, industries, and institutions during a fire event. The ISO recommends that the fire protection system have the ability to operate with the remainder of the potable water system operating at the MDD.

For one or two-family dwellings not exceeding two stories in height, the ISO has adopted the following necessary fire flow capacities (see Table 6.2-2 below):

Table 6.2-2 – Necessary Fire Flow Capacities

Distance Between Adjacent Buildings (ft)	Necessary Fire Flow Capacity (gpm)
> 100	500
31 to 100	750
11 to 30	1,000
1 to 10	1,500

Source: 2005 ISO Fire Suppression Rating Schedule

Further Conditions:

- When a building is covered with a wood-shingle roof type that the ISO concludes will contribute to fire promotion, 500 gpm is added to the necessary fire flow rates indicated above.
- For other kinds of inhabitable buildings, the maximum fire flow rate is 3,500 gpm.

For other kinds of structures, the ISO utilizes a formula to determine the necessary fire flow, which can result in higher flows when compared to those for residential dwellings. Most insurance requirements are specified in terms of maintaining a flow rate of Q over a time period T, where Q is on the order of thousands of gallons per minute and T is less than or equal to 10 hours.

Fire hydrants should be spaced at a maximum distance of 500 feet. Ordinarily, they are located at street intersections so that they can be accessed from multiple directions of approach.

The goal for the City of Lowell is to provide at least 1,000 gpm to each fire hydrant in the system, with at least 3,000 gpm available for protection of larger buildings for businesses, industries, or institutions.

Design Goal: A fire flow capacity of at least 1,000 gpm per hydrant, with at least 3,000 gpm available for larger buildings.

6.3 Basis for Cost Estimates

The cost estimates presented in this plan generally include four elements: construction cost, engineering cost, contingency reserve, and legal/administrative cost. Each of the cost elements is discussed below. These estimates are preliminary and based upon the level and detail of planning considered in this study. The construction costs are based upon competitive bidding prices submitted for public works projects. As projects begin to proceed, and as site-specific information becomes available, the estimates may require an update adjustment.

Construction Cost

The estimated construction costs in this plan are based upon actual construction bidding results from similar projects/efforts, published cost guides, and other construction cost resources or experience. For the determination of construction quantities, references included as-built drawings, system maps of the existing facilities, elevations of reservoirs and major system components, and locations of distribution lines. Where required, estimates will be based upon preliminary layouts of the proposed improvements.

Future changes in the cost of labor, equipment, and materials may justify comparable changes in the cost estimates presented herein. For this reason, common engineering practices usually tie the cost estimates to a particular index that varies in proportion to long-term changes in the national economy. Regarding public infrastructure projects, the Engineering News Record (ENR) construction cost index (CCI) is most commonly utilized. This index is based upon an arbitrary value of 100 for the year 1913. Average index values are displayed in Table 6.3-1 for the past 16 years.

Table 6.3-1 – Average Construction Cost Index Values

Year	Index Value	% Change from Previous Year
1990	4,732	2.54
1991	4,835	2.18
1992	4,985	3.10
1993	5,210	4.51
1994	5,408	3.80
1995	5,471	1.16
1996	5,620	2.72
1997	5,826	3.67
1998	5,920	1.61
1999	6,059	2.35
2000	6,221	2.67
2001	6,343	1.96
2002	6,538	3.07
2003	6,694	2.39
2004	7,115	6.29
2005	7,446	4.65
2006 (Sept.)	7,704	3.46

Source: Engineering News Record Construction Cost Index History (1990-2006)

Note: CCI for 2006 reflects only data for **January through September**.

The cost estimates provided in this plan are based upon the present (September 2006) worth of U.S. dollars with an ENR CCI of 7704. For construction performed in subsequent years, costs should be projected based upon the then-current year ENR CCI by means of the following formula:

Updated Cost Estimate = Plan Cost Estimate × (Current ENR CCI Value) ÷ 7704

Contingency Reserve

A contingency reserve of 20% of the estimated construction costs has been added to the estimated overall construction costs presented in this plan. Recognizing that cost estimates are based upon conceptual planning, allowances must be made for variation in final quantities, bidding market conditions, adverse construction conditions, unanticipated investigations and specialized studies, and other obstacles which cannot be foreseen in advance but may tend to increase final costs. Upon completion of the final design in any construction project, the contingency reserve can be reduced to 10%, but at least a 10% reserve should always be maintained.

Engineering Cost

The cost of engineering services for major projects will typically include:

- investigations and specialized studies
- predesign analyses and reports

- surveying and measurement
- preparation of contract drawings and specifications
- bidding services
- construction management
- periodic worksite inspection
- construction staking
- start-up services
- preparation of operation and maintenance manuals

Depending on the type and scope of the project, engineering costs may range from 18% to 25% of the overall project cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical/electrical/structural systems. The higher percentage applies to small or complicated projects. Engineering costs for design and construction services presented in this plan are generally based upon 18% of the estimated construction cost.

Legal/Administrative Cost

An allowance of 3% of the construction cost has been added for legal and administrative services. This allowance is intended to include internal project planning and budgeting, grant administration, liaison efforts, interest on interim loan financing, legal services, review fees, legal advertising, and other related expenses associated with the project which could be incurred.

Land Acquisition Cost

Certain projects may require acquisition of additional rights-of-way, or even property, for construction of a specific improvement. The necessity and cost for such expenditures is difficult to predict and must be monitored and/or reviewed as a project develops. Efforts were made to include costs for land acquisition, where anticipated, within the cost estimates for this plan.

Improvement Alternatives

Section 7

7.1 Background

In this section, needed improvements to various components of the Lowell water system are discussed, and options for implementing such improvements for a 25-year planning period are presented. Section 4 described the existing water system; Section 5 identified the production demands placed on that system; and Section 6 discussed the design criteria and service goals to which the existing system capabilities are compared in order to determine the needed improvements.

When feasible, the following steps should be utilized for evaluation of water system alternatives relative to particular system components:

- Identification of alternatives
- Elimination of non-viable alternatives
- Analysis of viable alternatives
- Comparison of viable alternatives
- Selection/Prioritization of alternatives for recommendation

The primary consideration in the evaluation, comparison, and selection of alternatives is cost. Both the capital improvement costs and the on-going operation and maintenance (O&M) costs must be carefully included in the analysis. It is not uncommon for an alternative with a higher capitalization cost to be a more cost-effective choice if the anticipated O&M cost will be lower over the design life of a component or system.

Affordability must also be considered. A system or component improvement which is not affordable (i.e., which exceeds the ability of the stakeholder to finance) is a non-viable alternative. Furthermore, such non-cost factors as ease of implementation, risk to operators, acceptance by public, and impact upon the environment must be taken into account as well.

Because the scope of this planning effort has been limited to source, treatment, and storage facilities, the following major sections provide a discussion on each of these elements of the overall water system.

7.2 Water Supply (Raw Water Source)

As developed in Section 6.2, the City of Lowell has a planning goal for their raw water supply of 1.074 cfs or 482 gpm (693,520 gpd) at the end of the 25-year planning period. The city must become capable of supplying raw water through a combination of ground and surface water sources. A discussion of the capabilities of each of the city's available water supplies is provided below.

Ground Water Supplies

The city holds water rights for two ground water sources (Wells #1 and #3) for a total supply level of 0.90 cfs or 404 gpm. However, records indicate that the sustained yield of the two wells is not expected to provide much more than 100 gpm (0.22 cfs) over any extended period of usage.

Also, there is a history of detection of high arsenic levels in water produced by the city's wells, which originally led to the development of the city's current surface water treatment system. In order to use the existing well supplies, treatment infrastructure will have to be developed and operated for the effective removal of arsenic from the ground water supplies.

Based on preliminary budgets from arsenic removal equipment suppliers, it is estimated that a removal system for existing well yields (approximately 100 gpm) would cost between $100,000 and $150,000. For the small amount of water which can be gained from these wells, it is unlikely that development of the wells to supply additional water for the city would be a viable alternative for the future.

Surface Water Supplies

The City of Lowell holds surface water rights on Dexter Lake for a total of 1.0 cfs. According to the planning goal established in Section 6.2, the city will require 1.074 cfs (693,520 gpd) to satisfy the 25-year MDD. Figure 7.1-1 below indicates the projected MDD over the planning period and beyond, the capacity of the existing treatment facilities, and the existing 1.0 cfs (646,272 gpd) surface water right.

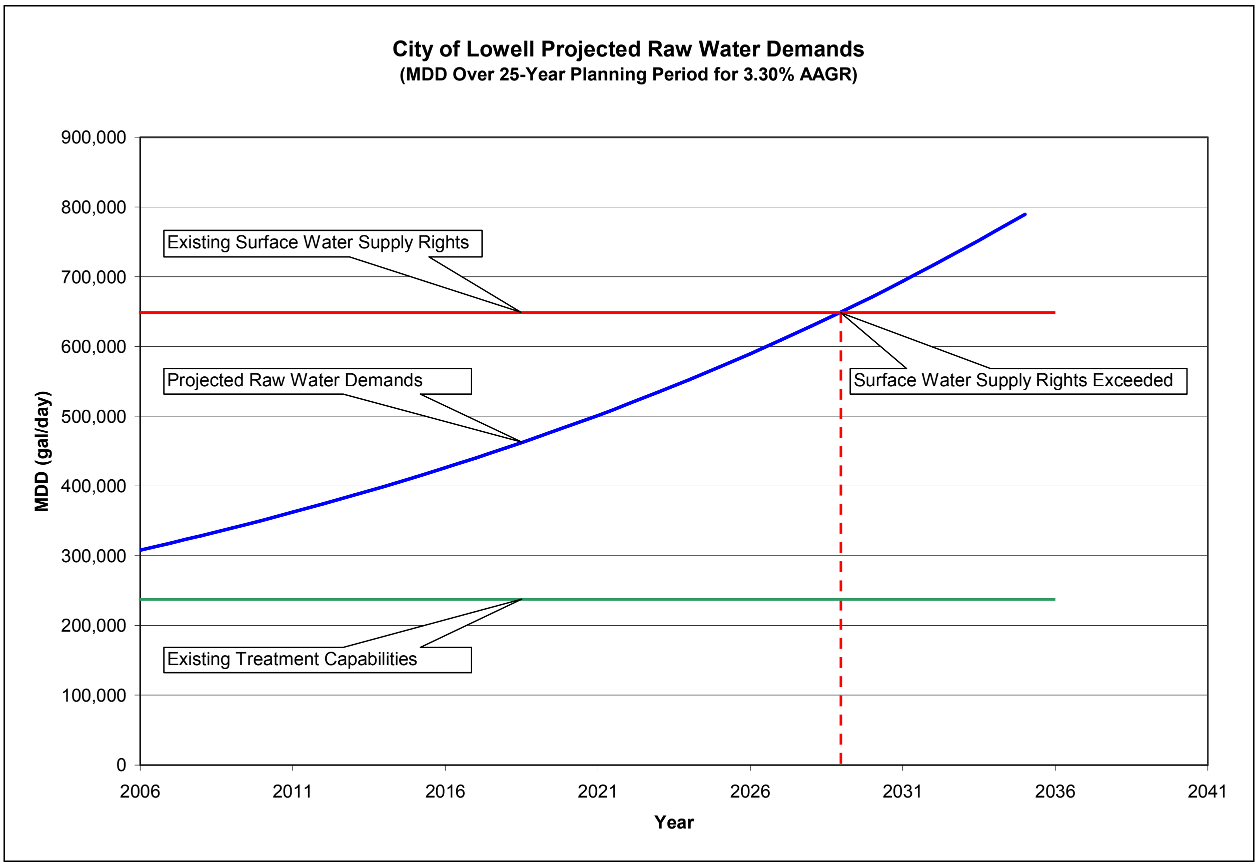


Figure 7.1-1 – Water Supply and Treatment Projections

According to the usage trend indicated in Figure 7.1-1, the city has adequate surface water supplies to meet the maximum daily demands through (approximately) the year 2029. After that, the city will not have adequate surface water rights to meet the needs of the community.

The city should begin planning for the eventuality that they will only be able to provide water for a little more than 20 years at the projected growth rate assumed in this plan (3.30%). The projected maximum service population in the year 2029 is about 2,100 persons or 800 EDUs. Any growth beyond this level will require the acquisition of additional surface water supplies.

During the preparation of this master plan, the U.S. Army Corps of Engineers was contacted to determine if any additional water was available for acquisition from Dexter Lake for the purpose of supplying a municipality. According to representatives from the Corps of Engineers (COE), opportunities may exist for the city to apply for additional water rights on Dexter Lake. Although the COE would not guarantee that additional water rights would be granted to Lowell, they did describe a "water supply reallocation process" that might lead to additional water rights for the City. The contact at the COE was Mr. Erik Peterson, who can be reached at (541) 937-2131. The COE could not comment on the potential cost of obtaining additional water rights at this time.

Blended Flow Alternative (Surface and Groundwater)

As it is projected that the city will require additional water before the end of the planning period, it may be possible to blend groundwater with surface water to provide additional flows.

Some communities have found a benefit in blending raw surface water and ground water together as the two sources typically differ in pH, hardness, alkalinity, and other qualities. When blended, the combined raw water can be easier to treat, though this varies on a case by case basis.

However, due to the relatively high levels of arsenic that have been observed in the City's wells in the past, it is difficult to recommend further use of the wells, through blending or otherwise. As discussed in this section, treatment facilities could be constructed to effectively remove the arsenic, but the cost of the treatment facilities compared to the expected yield is not reasonable.

While many of the treatment options discussed later in this section may provide some level of removal for arsenic, it is most likely that a dedicated arsenic treatment system would be required to ensure that the water supply is safe for consumption. Therefore, it is not recommended, at this time, that the City pursue any option where well and surface water is blended.

Depending on the treatment process that is selected, it may be appropriate, at a later date, to undertake some pilot work to determine the level of arsenic currently present in the well water and the effectiveness of the treatment process in removing the arsenic. If it is found that arsenic levels can be reduced to reasonable levels with standard treatment, the City may, in the future, consider a raw water blending option.

Conclusions and Recommendations – Water Supply

The City of Lowell is slightly deficient for surface water supplies during the planning period by an amount of 0.074 cfs or 33 gpm (about 48,000 gpd). While this shortage is relatively small, the city should begin considering how to acquire more water should their demands increase as projected in this plan.

As mentioned above, one option is that the city could invest in their existing wells and provide treatment facilities for the removal of arsenic. Other well upgrades may be possible to improve both the quality and quantity of water available from the wells. However, if the existing wells are only capable of a sustained production rate of 100 gpm, then it may be unwise to further invest in these facilities.

Since the possibility exists to obtain additional water rights on Dexter Lake, it is recommended that the city immediately begin the water supply reallocation process with the COE. The findings and information provided in this master plan should serve as support for the request. If the city is successful in obtaining additional water rights, it will not be necessary to develop systems to improve the ground water supplies in Lowell. While there is likely to be a cost associated with obtaining these additional water rights, that information is not currently available. As the City progresses further into the reallocation process and identifies the cost of the additional water rights, these costs should be added to the CIP and included in any SDC methodology calculations.

Recommendation: Seek to obtain at least an additional 1.0 cfs of surface water rights on Dexter Lake through the water supply reallocation process with the U.S. Army Corps of Engineers.

7.3 Water Treatment Facilities

According to the criteria established in Section 6.2, the City of Lowell should seek to develop treatment facilities capable of processing 0.757 MGD or 525 gpm over 22 hours of operation to satisfy the 25-year MDD.

Figure 7.1-1 above shows the existing treatment plant capacity to be about 211,200 gpd (160 gpm for 22 hours). This capacity is clearly shown to be inadequate to satisfy the current MDD production levels. Therefore, at the present time, the existing treatment facilities are already deficient.

The treatment plant must be upgraded to a capability of processing 525 gpm for 22 hours in order to satisfy the demand projections developed in this master plan. The remainder of Section 7.3 will address individual components or functions of the treatment process and will investigate various alternatives to consider for each element of the treatment process as well as complete treatment alternatives.

Raw Water Facilities

For the purposes of this analysis, the raw water facilities will be defined to include the intake structures, raw water pumping systems, and other equipment for the delivery of raw water to the treatment plant.

Intake Box and Screen – As described in Section 4, the city utilizes a submerged, screened intake box which is located in the main channel of the lake near the covered bridge portion of the causeway. The available information suggests that the screen provides approximately 17 square feet of screening area. Since the sizes of the screen openings are unknown, the velocity of the water approaching the intake box and through the screen cannot be estimated. However, with 17 square feet of area, it is unlikely that velocity issues will be a concern. Therefore, no improvements to the intake box or screen are anticipated during this planning period.

Intake Piping – The intake piping consists of approximately 2,500 feet of 10-inch PVC pipe that connects the intake box to the plant. For a flow rate of 525 gpm, the average velocity within the pipe would be about 2.1 fps. This velocity is acceptable for the flows projected over the planning period. Therefore, no improvements are necessary for the intake piping.

Raw Water Pumps – Section 4 remarked that the existing (newer) pump is capable of producing about 200 gpm at 37 feet TDH. The current practice of operations personnel is to throttle the raw water flow with a butterfly valve to about 160 gpm in order to optimize (as determined by experience) the water production throughput. An older pump, which is currently disconnected, is reported to have a capacity of about 400 gpm.

Improvements to the raw water pumping system should include at least two pumps, each being capable of providing the total design capacity for the treatment facilities. The capacity needed for the 25-year MDD is 525 gpm at approximately 40 feet TDH. The duty cycle for these pumps should be to alternate daily at the initiation of plant start-up.

It is possible that the existing raw water pumping area will to need be expanded, or that a new raw water pump station should be constructed to house the new pumping equipment, fittings, and valves. It also may be possible to remove the interior walls of the chlorine room and obtain additional space for new electrical or pumping equipment in this area, thereby retaining the existing structural configuration. The course of action taken will ultimately depend on the final treatment alternative selected.

The new pumping system should include variable-frequency drives (VFDs) for each pump so that plant operators have the ability to vary the flow without throttling fully-loaded pumps. The VFDs will provide this flexibility as well as reduce electrical power costs by operating the pump motors in an “off-phase” condition.

Final recommendations and costs for the raw water pumping system will be offered later in this section under the discussion of complete treatment alternatives.

Existing Primary Treatment Facilities

For the purposes of this analysis, the primary treatment facilities will be defined to include facilities that directly accomplish water treatment prior to filtration. Processes associated with these facilities include chemical pre-treatment, mixing, flocculation, sedimentation, and possibly other processes.

Chemical Coagulation – The city currently utilizes polyaluminum chloride sulfate (SternPAC™) to accomplish coagulation. This substance chemically binds together smaller impurities that are present in the raw water until they form larger, heavier particles which can be more readily removed. The city has experimented with several brands and formulas and has found a product that works well for their unique water conditions. There is no need to consider other alternatives for chemical coagulation at this time.

Mixing – An existing 6-in static mixer enables rapid mixing. This mixer is adequate for existing flows, but replacement of the mixer should be considered during the upgrade process if chemical coagulation is be continued. A slow mix unit is employed for mechanical stirring at approximately 50 rpm within the existing flocculation chamber, which is located adjacent to the sedimentation/clarification tank. Should the existing configuration be retained for the upgrade, it would be prudent to replace this mixing unit as well.

Sedimentation/Clarification – The existing treatment process incorporates a sedimentation/clarification component which is intended to be operated as a solids-contact clarifier. However, the geometry of the clarifier is not of a typical design for traditional contact clarifiers. Notwithstanding, the city has been able to operate the clarifier effectively and has been able to generally produce well-clarified water prior to their filtration process.

As discussed in Section 4, the existing clarifier has a cross-sectional area of about 104 sq ft. At the design capacity of 525 gpm, a horizontal velocity of 0.67 ft/min would be expected. A maximum horizontal velocity of 0.5 ft/min should be utilized for design purposes.

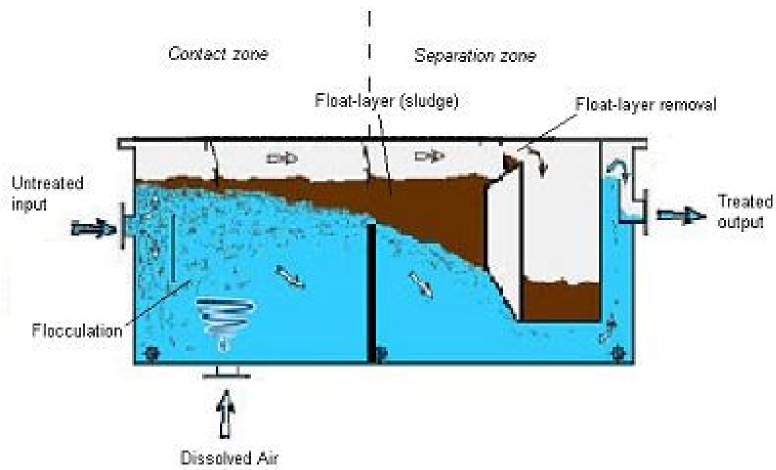
The existing clarifier utilizes tube settlers to improve the performance of the clarifier. The surface area of the tube settlers is about 217 sq ft. At the design flow rate of 525 gpm, the upflow rate through these settlers is about 2.42 gpm/sq-ft. A range of 1 to 3 gpm/sq-ft should be considered for design. During projected operation, these settlers will function in the upper part of the acceptable range but may provide only limited benefit to the treatment process at the future flow rate.

While detention time in a clarifier can vary widely, it is generally accepted that a sedimentation basin or clarifier tank should provide at least 1.5 to 4 hours of detention time in order to be effective with the longer detention time providing better clarification performance. At the future flow rate, the existing clarifier will only provide about 38 minutes of detention time.

Based upon these analyses, it is unlikely that the existing clarifier will provide adequate or effective sedimentation or clarification for the projected design flows. If chemical coagulation is to be followed by a clarification step, additional system components will be required.

Sedimentation/Clarification Alternatives

As it is likely that new sedimentation or clarification facilities will be required, this section will seek to discuss various alternatives for consideration.



DAF Clarification. Dissolved Air Flotation is a relatively new technology that utilizes a curtain of fine micro-bubble air to lift contaminants to the top of the tank rather than waiting for the heavier floc particles to settle to the bottom of a tank.

Chemical coagulation is still typically utilized to cause smaller particles and dissolved matter to form larger particles. The bubbles in the water cause these particles to become very buoyant and quickly float to the surface of the tank where they are removed and channeled to a waste stream. The fine curtain of bubbles also serves to oxygenate the water which provides a level of oxidation that will enhance pretreatment without the formation of harmful byproducts.

DAF systems are typically provided as a packaged unit that includes the tank, aeration equipment, waste removal, and effluent components. The systems are typically skid mounted and can be inserted quickly into the treatment process. DAF systems can also be installed in cast-in-place concrete tanks. Typically concrete DAF units are constructed in round clarifier-type tanks.

DAF systems are particularly effective in removing algae from raw water streams as the algae is typically very light and difficult to remove through a settling process. Also, the highly oxygenated or aerated raw water will help in reducing taste and odor issues due to the natural oxidation process, though it is not as effective as common activated carbon methods at removing taste and odor problems.

Table 7.3-1 – Cost Estimate for Packaged DAF System

DAF - Steel Tank Option (packaged)					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$50,000.00	$50,000.00
2	Construction Facilities/Temporary Systems	ls	1	$40,000.00	$40,000.00
3	Concrete pad for packaged DAF system	sf	800	$20.00	$16,000.00
4	Packaged DAF Equipment	ls	1	$300,000.00	$300,000.00
5	Site piping, fittings & misc. appurtenances	ls	1	$60,000.00	$60,000.00
6	Boost pump to lift flows into existing plant	ls	1	$20,000.00	$20,000.00
7	Electrical improvements	ls	1	$75,000.00	$75,000.00
8	Misc. appurtenances, simple roof structure, etc.	ls	1	$75,000.00	$75,000.00
		Construction Total			$636,000.00
		Contingency (20%)			$127,200.00
		Subtotal			$763,200.00
		Engineering (18%)			$137,376.00
		Administrative costs (3%)			$22,896.00
		Total Project Costs			**$923,472.00**

Conventional Sedimentation Tank. Clarifiers can be constructed in many shapes and configurations including round and rectangular. Generally, the most cost effective tank will be a long and narrow rectangular tank designed to provide good mixing, flocculation, detention time, and easy removal of settled solids.

While parameters and recommended sizing criteria vary widely by source, the Ten States Standards provide reliable sizing criteria. Based on these stands, the following criteria should be used to layout a new rectangular sedimentation tank:

- Length to width ratios of 4:1 to 5:1 (i.e. a 10-foot wide basin should be between 40 and 50 feet long)
- Basin depth and horizontal velocity: Such depth that the horizontal velocity is less than 0.5 ft/min. Therefore, if the tank is 10 feet wide, it should be around 14 feet deep.(525 gpm = 70 cu ft/ min, 70 cu ft/min / 0.5 ft/min = 140 square feet, 10 feet x 14 feet = 140 sq ft)
- If tube settlers are utilized, around 250 square feet (minimum) of tube settlers should be provided for the projected design flow.
- Outlet weirs: overflow rate should be less than 20,000 gpd/lf of launder. At the projected flow rate of 525 gpm, the existing clarifier would be at around 15,400 gpm/lf.
- Detention time should range between 1.5 to 4 hours depending on the performance requirements and raw water quality of the specific water source. (The existing clarifier to provide only 38 minutes of detention time at the projected flow rate.)
- Two-stage flocculation chamber with at least 30 minutes detention time in the flocculation process

Based on the above criteria, the following theoretical rectangular sedimentation basin is described as an alternative for the City of Lowell:

- Overall basin dimensions: 12' wide x 74' long x 14' deep
- Includes a 12' wide x 14' long x 14' deep flocculation chamber with over 30 minutes detention time at design flow and two stage flocculation
- Includes a 60'long x 12' wide x 14' deep sedimentation basin
- Sedimentation basin to include approximately 300 square feet of tube settlers, at least 40 lineal feet of finger launders, and other necessary appurtenances

A cost estimate for the above described sedimentation basin is provided below:

Table 7.3-2 – Cost Estimate for Reinforced Concrete Sedimentation Basin

Reinforced Concrete Sedimentation Basin					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$50,000.00	$50,000.00
2	Construction Facilities/Temporary Systems	ls	1	$30,000.00	$30,000.00
3	Site prep and Foundation Stabilization	ls	1	$10,000.00	$10,000.00
4	Reinforced Concrete Tank	cy	300	$800.00	$240,000.00
5	Flocculation equipment	ls	1	$45,000.00	$45,000.00
6	Baffling	ls	1	$10,000.00	$10,000.00
7	Tube settlers and launders	ls	1	$25,000.00	$25,000.00
8	Catwalks, stairs, sidewalks, and other appurtenances	ls	1	$50,000.00	$50,000.00
9	Electrical Improvements	ls	1	$25,000.00	$25,000.00
10	Site piping, fittings & misc. appurtenances	ls	1	$50,000.00	$50,000.00
		Construction Total			$535,000.00
		Contingency (20%)			$107,000.00
		Subtotal			$642,000.00
		Engineering (18%)			$115,560.00
		Administrative costs (3%)			$19,260.00
		Total Project Costs			**$776,820.00**

Existing Secondary Treatment Facilities (Filtration)

For the purposes of this study, secondary treatment facilities will be defined as any treatment processes utilized after preliminary treatment measures. This generally includes filtration measures.

The City of Lowell utilizes two filter cells, each with a total filtration area of 57.5 square feet. The two filters utilizes block underdrains and a dual media filtration cross section. Surface washers aid in backwash with a reported backwash rate (upflow) of around 1,000 gpm (17.4 gpm/sf) for each filter backwash. A third filter bay was constructed at the plant for future expansion though the under drain and filter media were not installed. All together, the City has the potential for a total of 172.5 square feet of filter area.

Like the sedimentation process, design parameters vary depending on source and application. However, the following criteria should be considered when designing or analyzing a conventional filter process:

- A filtration rate under 3 gpm per square foot should be utilized for most applications,
- A backwash rate of no less than 15 gpm per square foot should be used with rates of 20 gpm

preferred for optimum bed expansion.

- If surface wash is used, a rate of around 0.5 gpm per square foot should be used at a pressure of at least 45 psi for rotating arm washers.

- If air scour is to be used, between 3-5 cubic feet of air per square foot of filter is required for optimal operation. A program designed to combine different amounts of air and water at different times during the backwash sequence is required.

Based on these parameters, the City's filters will be able to produce a total of around 518 gpm of filtered water with all filters operating at one time. This is close enough to the design flow (525 gpm) to be acceptable.

With a backwash rate of around 17.4 gpm/sf, the upgrade may consider increasing this flow slightly if it is determined that additional expansion is required.

Conclusion: *The existing filters are adequate to meet projected demand flows assuming that changes are made to the plant so that all three filters can operate simultaneously. Existing backwash rates are adequate but could be increased. It may also be beneficial to upgrade the backwash system to utilize air scour rather than surface wash.*

Alternative Secondary Treatment Facilities (Membranes)

In recent years, the use of membrane treatment technology has surpassed conventional filtration in both new and upgrade projects. As a result, a water provider should consider the potential benefits which membrane technology can provide for their community over conventional water treatment.

Membrane treatment utilizes a physical barrier (membrane) with microscopic pores or openings that allow water to pass through but trap debris and impurities on the raw water side of the membrane barrier. By utilizing a physical barrier for a filter, a virtual guarantee is provided that no impurities (including most bacteria and viruses) can pass through the membrane and into the drinking water supply.

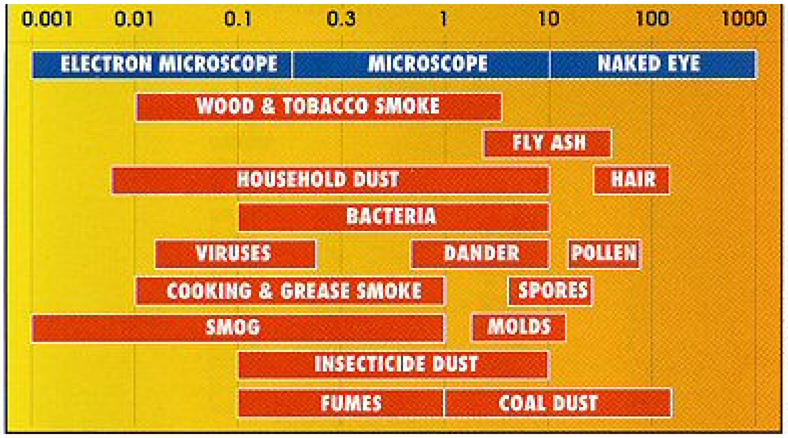
Membrane treatment, in the municipal water treatment industry, is currently divided into two main categories: microfiltration and ultrafiltration. The two categories differ in the size of the membrane openings and, therefore, the quality of treatment obtained. The opening sizes are further described as:

- Microfiltration – 0.1 micron to 1 micron (0.1 nominal)

- Ultrafiltration – 0.002 to 0.1 micron (0.01 nominal)

While ultrafiltration clearly provides a higher level of treatment, it may not be necessary as most bacteria and viruses are larger than 0.1 microns and will be removed with microfiltration. What is not removed is, theoretically and statistically, removed through disinfection measures. However, many communities have elected to make the investment in equipment to provide treatment at the ultrafiltration level.

The following figure provides a comparison of some common items and how they fit onto the micron scale. The chart indicates that bacteria falls between 0.1 and 10 micron while viruses fall between 0.01 and around 0.25 micron. Dust, molds, pollen, and other debris are much larger.

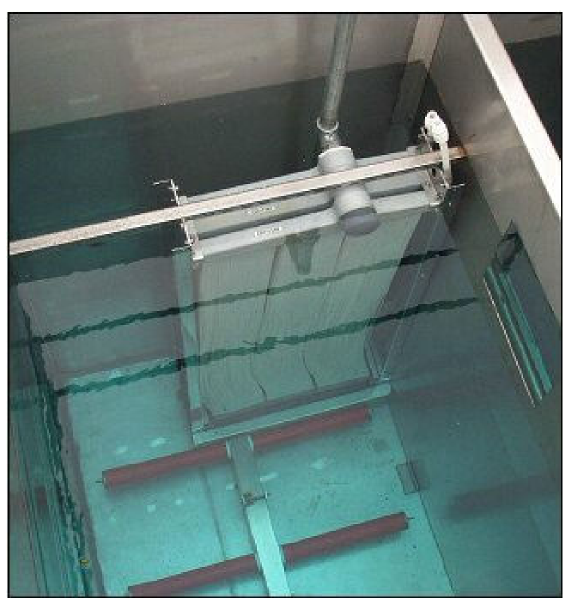


Within the two main performance categories, membranes are further divided into pressure and vacuum systems. The two types of membrane processes simply differ on how the water is induced across the membrane; either through the use of pressure to push water across the membrane or through the use of a vacuum to pull it across. Other differences in equipment include configuration, orientation, and fit and finish.

For the City of Lowell, two main options should be considered. These include the use of submerged membranes in the existing filter bays or the use of a skid mounted packaged membrane system. A third alternative is presented wherein the City would utilize a packaged conventional treatment system. These alternatives will be further developed below.

Submerged Membranes (in existing filter bays). Zenon Corporation manufactures membranes that can be retrofitted into many existing filter bays to replace conventional filtration processes. The old filter media and underdrains are simply removed and discarded and the new membrane units are fitted into the empty bays. While there are some geometric limitations, many systems have found it practical to retrofit their existing conventional plants with membrane units with little or no changes to the treatment process.

The advantages of this type of system are that existing tanks and equipment can be reused. Also, the process flow remains essentially unchanged with only the filtration step being greatly enhanced and improved. Backwash troughs and other existing systems can be reused with little or no changes. Zenon equipment also operates at an ultrafiltration level of quality so a very high level of performance and treatment can be expected from these membranes.



The disadvantages of using this type of system are usually cost related. Often, the cost to demolish and retrofit existing systems can be greater than using standard skid mounted systems to establish a membrane treatment process.

Zenon was contacted and asked to provide a proposal for the City of Lowell project. However, it was determined that a minimum of 8 feet of depth would be required in the existing filter bays to install the

Zenon equipment. According to available drawings, little more than 6 feet is available in the basins, even with the removal of the underdrains. Therefore, a submersible membrane in the existing filters bays will not be considered further.

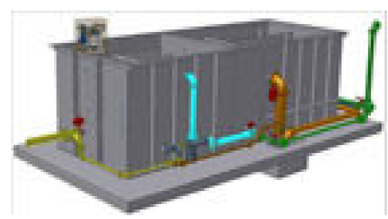
Packaged Skid Membrane Units. A number of equipment suppliers manufacture and supply skid mounted and packaged membrane treatment units in a variety of configurations and sizes. The skid mounted equipment can be provided in both vacuum and pressure configurations and in both ultra and microfiltration performance levels.

Use of a skid membrane system will, most likely require the construction of a small building to house the new equipment. As additional chlorine contact volume is likely to be required, the City may construct the new building above the new clearwell (see discussion below on disinfection).

Pretreatment may be provided by operating the existing clarifier unit at an accelerated rate, or by installing a DAF system in front of the membrane system, though most manufacturers reported that pretreatment would not be mandatory given the relatively high water quality levels available to the City of Lowell throughout the year.

The cost of packaged membranes can vary widely. Costs for packaged membranes will be discussed later in this section as part of the complete alternatives analysis.

Packaged Conventional Treatment Units. As with membrane systems, several companies offer packaged conventional treatment equipment capable of treatment a wide range of flows.



One of the leading equipment choices in this category is the Microfloc Trident packaged treatment unit. This equipment includes a proprietary flocculation process, a clarifier, and a filtration process all packaged within a single tank and within a relatively small footprint.

Packaged plants are often limited by the quality of the raw water. Turbidities over 20 NTU are typically difficult to treat. However, as Lowell has relatively good raw water quality throughout the year, a packaged plant would most likely provide good treatment service.

A conventional plant will not produce finished water at the high level of quality that is possible with membranes. Conventional plants also require more chemicals and operator involvement to ensure that flocculation, chemical balance, and overall treatment is operating well. Membrane plants will produce essentially the same quality of water regardless of the raw water quality or chemical balance (if chemicals are used at all).

Disinfection Facilities

The City of Lowell must utilize disinfection (through the use of some form of chlorine) to disinfect their drinking water supplies. Disinfection facilities are to include the disinfectant itself, feed equipment, and chlorine contact vessels.

The City currently utilizes a gas chlorination system with a capacity of around 4 ppd. According to data from the past several years, the City typically uses in excess of 2 ppd with peaks as high as 3 ppd of chlorine.

Based on a dosage of 1 mg/L for the projected MDD, the City must be prepared to deliver at least 5.8 ppd of chlorine to their system. Therefore, the existing system is inadequate to provide for the projected chlorine capacity requirements. It is recommended that the upgraded system be capable of handling at least double the amount projected or around 10 to 12 ppd of chlorine.

Chlorine Disinfection Systems. While gas systems tend to be reliable and the operators in Lowell are well versed in the use of chlorine gas, it remains one of the most deadly and dangerous chemicals that water treatment plant operators can be exposed to. For this reason and others, many of the small water systems in the state have converted over to an alternative disinfectant such as liquid hypochlorite or another variation of liquid chlorine.

Liquid hypochlorite is generally available for water treatment facilities in a nominal 12% strength solution. The solution can be diluted or fed neat using simple chemical feed pumps. The disadvantage to liquid hypo systems is that the chlorine solution rapidly degrades. In other words, a 12% nominal solution may only be 10% upon delivery. Before it can be used, it may have degraded (or off-gassed) to 8%. This requires regular feed rate adjustments to ensure that a consistent residual is met with the inconsistent chemical strength.

Another alternative that has gained widespread acceptance is the use of on-site generation systems to generate a low-strength solution (typically between 0.4% and 0.8%) that is fed directly into the treatment stream. This low strength solution has proven to have a steady strength that does not degrade like the stronger solutions. Also, because the solution is generated on site, it is not stored for very long before it is used. It has also been found that on-site generated disinfectant products have a more robust and long-lasting residual in the distribution system.

On-site generation systems utilize typical water softener grade rock salt, water, and an electrical current to generate disinfection chemicals. The most dominant disinfectant species is sodium hypochlorite though it is likely that other species are generated in smaller amounts.

Another advantage to using on-site generation is the obvious safety improvement as opposed to using chlorine gas. The only raw material stored on site is common rock salt. There are no alarms, gas sensors, or hazardous spaces that are required. No ventilation or breathing apparatus are needed at the plant. On-site systems also eliminate the need for transporting gas cylinders on the roads and through the community.

On-site generations systems are available from a number of equipment manufacturers. A number of suppliers were contacted to provide a proposal for the City of Lowell plant upgrade effort.

Alternatively, the city could upsize their existing gas system. While this would have a lower capital cost, it does not address the safety and risk issues associated with gaseous chlorine in a community. For budgetary purposes, it is estimated that the gas system could be upsized for approximately $7,500 including equipment, design work, and any appropriate contingency.

Table 7.3-3 – Cost Estimate for the Installation of On-site Chlorine Generation Equipment

Onsite Chlorine Generation - 10 ppd					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$3,500.00	$3,500.00
2	Construction Facilities/Temporary Systems	ls	1	$2,200.00	$2,200.00
3	On-Site Chlorine Generation Equipment - Miox	ls	1	$28,500.00	$28,500.00
4	Chemical Metering Pumps and Equipment	ls	1	$3,000.00	$3,000.00
5	Salt, spare parts, and Misc. Appurtenances	ls	1	$1,000.00	$1,000.00
		Construction Total			$38,200.00
		Contingency (20%)			$7,640.00
		Subtotal			$45,840.00
		Engineering (18%)			$8,251.20
		Administrative costs (3%)			$1,375.20
		Total Project Costs			**$55,466.40**

Chlorine Contact Vessel. (CT) Chlorine must be physically in contact with the drinking water for a period of time in order for the oxidation reactions to be complete to ensure that all inactivation of viruses and bacteria has been accomplished. As these reactions are time dependent, vessels must be provided for the water to reside until it can be assured that the reactions have had adequate time.

The State has adopted tables (CT Tables) that have been established by the EPA to determine the amount of chlorine contact time (CT) that is required for a particular water system. The amount of time required varies based on temperature, pH, the amount of disinfection inactivation required based on their treatment effectiveness and the amount of chlorine used in the disinfection process.

According to a March 2003 CT Tracer Study (HBH Consulting Engineers, Inc.), the existing clearwell was determined to provide around 44 minutes of contact time at the current flow rate of 160 to 170 gpm (160 gpm into the clearwell and approximately 170 gpm out of the clearwell). At this flow rate, the baffling or efficiency factor of the clearwell was determined to be around 0.365 or 36.5% effective.

If the same efficiency factor is used, and the flow increased to the projected rate of 525 gpm, the existing clearwell is estimated to provide around 16 minutes of contact time. The CT study also utilized approximately 600 lf of 10-inch transmission piping prior to the first user. At 525 gpm, this piping accounts for an additional 4 ½ minutes of contact time. Together, it is estimated that the City may have a total of 20.5 minutes of contact time. Because of the shape of the existing clearwell and the fact that the operating range is typically between the top 2 feet of the clearwell, increasing the normal operational depth of the clearwell may not alone be a viable option to increase contact time at the projected design flow rate.

According to the 2003 Tracer Study, typical worse case conditions for the City of Lowell are:

- Typical pH ~ 7
- coldest temperatures ~ 8°C
- typical chlorine residual in clearwell ~ 0.8 mg/L

Under these conditions, the City must have the following contact times for each level of inactivation (using the conservative 5°C Table):

- For 1-log reduction: CT Value of 49, equal to 61.25 minutes
- For 0.5-log reduction: CT Value of 24, equal to 30 minutes

As the existing plant has been credited with a 2.5-log removal, the City is already only required to obtain an additional 0.5-log removal through disinfection. Therefore, we must assume that, at the conditions described above, that the City will need to provide at least 30 minutes, if not more, contact time to meet their CT requirements.

Therefore, as only around 20-minutes of contact time is available, additional chlorine contact vessels or elements must be added as part of the plant upgrade. This can be accomplished in one of two ways; additional volume in a new clearwell, or, through dedicated piping to the reservoir.

Additional Clearwell Volume. The first option to consider would include the construction of a new clearwell to add volume to the existing clearwell and increase the contact time available at the plant site. When determining the volume required, it is recommended that the City plan to obtain all necessary contact time on the plant site and not rely on transmission piping for contact time.

It was previously estimated that, at 525 gpm, the existing clearwell would provide around 16-minutes of contact time. To be conservative, the new clearwell should provide an additional 20-minutes of contact time prior to pumping the finished water into the system.

At 525 gpm and assuming an average baffling efficiency factor of 0.75 (75%), a clearwell volume of 14,000 gallons would be required to obtain the 20 minutes of contact time. This additional contact volume must be available under the same worst case level conditions of the existing clearwell as the two will be hydraulically connected. It will be assumed that the contact volume will be obtained in the bottom 48-inches of the clearwell depth. Additional volume will be available above this depth as is the case with the existing clearwell.

While the final dimensions of the clearwell may vary, a reasonable size to consider might be 24'x 20' x 4' deep. The overall basin depth should be around 6 feet deep with a foot or two of freeboard (say 8' deep to overflow). The basin should include internal serpentine baffling and special inlet and outlet conditions to obtain the efficiency factor of 0.75 as assumed above. The new clearwell should be hydraulically connected to the old clearwell with piping so that the water level in both clearwells is always the same.

While the final cost will vary depending on the final treatment process and configuration selected, the following cost estimate is provided for the construction of a new clearwell as described above.

Another option to add additional clearwell volume would be to construct a steel, above-grade reservoir somewhere on the treatment plant site to be used for contact volume. For this to be possible, treated water would have to be pumped, first, into the steel tank then allowed to flow by gravity into the existing clearwell where it would have to be pumped a second time into the distribution system. This additional pumping would result in additional operating costs (electrical), a second set of pumping facilities (capital cost) and increased sophistication.

While a below grade clearwell can be constructed in such a way as to provide very efficient and effective chlorine contact volume, a prefabricated and erected above ground tank cannot generally provide the same level of efficiency. Therefore, an above-ground tank typically must be larger. If we assume that we can effectively baffle an above ground tank, it would likely need to be capable of providing an additional 30,000 gallons of contact time to meet CT requirements.

Another advantage of a below-grade concrete clearwell is that other facilities can be constructed over the clearwell. If space on the site is limited, placing the clearwell under the footprint of a building is crucial. As there is little space immediately around the existing treatment facility, an above ground tank would have to be constructed further from the plant, thus requiring more site piping and pumping costs to transfer water to and from the tank.

Estimated costs are provided below for both the below grade and above grade clearwell options.

Table 7.3-4.a – Cost Estimate for Reinforced Concrete Subgrade Clearwell

Reinforced Concrete Clearwell					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$18,000.00	$18,000.00
2	Construction Facilities/Temporary Systems	ls	1	$10,000.00	$10,000.00
3	Excavation	cy	350	$15.00	$5,250.00
4	Reinforced Concrete Construction	cy	115	$800.00	$92,000.00
5	Backfill & Sitework	cy	150	$20.00	$3,000.00
6	Interconnecting piping & inlet/outlet construction	ls	1	$20,000.00	$20,000.00
7	Misc. Appurtenances	ls	1	$50,000.00	$50,000.00
		Construction Total			$198,250.00
		Contingency (20%)			$39,650.00
		Subtotal			$237,900.00
		Engineering (18%)			$42,822.00
		Administrative costs (3%)			$7,137.00
		Total Project Costs			**$287,859.00**

Table 7.3-4.b – Cost Estimate for Above Grade Steel Tank Clearwell

Above Grade Steel Tank Clearwell					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$25,000.00	$25,000.00
2	Construction Facilities/Temporary Systems	ls	1	$15,000.00	$15,000.00
3	Foundation and earthwork	ls	1	$10,000.00	$10,000.00
4	30,000 gallon glass to steel tank	ls	1	$80,000.00	$80,000.00
5	Internal baffling system	ls	1	$15,000.00	$15,000.00
6	Site piping and appurtenances	ls	1	$35,000.00	$35,000.00
7	Pumping system to lift water into tank	ls	1	$25,000.00	$25,000.00
8	Controls and instrumentation	ls	1	$5,000.00	$5,000.00
7	Misc. Appurtenances	ls	1	$50,000.00	$50,000.00
		Construction Total			$260,000.00
		Contingency (20%)			$52,000.00
		Subtotal			$312,000.00
		Engineering (18%)			$56,160.00
		Administrative costs (3%)			$9,360.00
		Total Project Costs			**$377,520.00**

The benefits of the below grade clearwell are as follows:

- Little or no maintenance on the concrete (permanent)

- Less costly to construct and far less costly to operate as additional pumping will not be required.
- Clearwell can be hidden beneath new building which will solve space problems on the limited site.
- Piping costs are less as the new clearwell will be very close to the old clearwell.
- Baffling and design can be completed to maximize efficiency and effectiveness of the contact volume.

For these and other reasons, it is recommended that a below-grade concrete clearwell be utilized for the complete treatment alternative considerations.

Dedicated Piping to Reservoir. An alternative to constructing an additional subgrade clearwell is to take advantage of the contact volume available in the existing reservoir. However, the City must be capable of delivering finished water to the reservoir without having to provide water service to any customer prior to the water passing through the reservoir. As there are many services on the existing piping prior to the reservoir, a new and dedicated transmission pipe would need to be installed to deliver water from the treatment plant site to the existing 500,000 gallon tank.

The dedicated transmission line could be placed within existing rights of ways, in many cases paralleling existing distribution piping that is servicing the customers. At 525 gpm, the dedicated transmission piping should be an 8-inch C-900 or HDPE piping section. With a total length of 3,800 ft (depending on the final pipe routing), the transmission piping itself would provide up to an additional 19 minutes of contact time. This would alone provide enough contact time for the projected operating conditions. However, in addition to the time that would be available in the dedicated main, the reservoir itself could provide in excess of 90 minutes of contact time.

A cost estimate for the dedicated transmission piping option is provided below.

Table 7.3-5 – Cost Estimate for Dedicated Transmission Piping to Reservoir

Dedicated Piping to Use Reservoir for CT					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$30,000.00	$30,000.00
2	Construction Facilities/Temporary Systems	ls	1	$18,000.00	$18,000.00
3	8" PVC Dedicated Transmission Main to Reservoir	lf	3800	$50.00	$190,000.00
4	AC Pavement Replacement	lf	3000	$10.00	$30,000.00
5	Interconnections at plant and reservoir	ls	1	$20,000.00	$20,000.00
6	Miscellaneous appurtenances	ls	1	$20,000.00	$20,000.00
		Construction Total			$308,000.00
		Contingency (20%)			$61,600.00
		Subtotal			$369,600.00
		Engineering (18%)			$66,528.00
		Administrative costs (3%)			$11,088.00
		Total Project Costs			**$447,216.00**

Taste and Odor System Upgrades

The City has struggled historically with taste and odor issues brought on by summertime algae blooms. The taste and odor is most likely a result of dead or dying algae and the chemicals contained within the algae being taken in through the City's raw water intake in the lake. This condition appears to happen only during the hottest summer months and appears to be worse during years when there is little runoff from snow melt in the mountains causing lower flows through the lake.

The materials in the water causing the taste and odor problems are not likely to be removed through any normal treatment process utilizing chemical coagulation, sedimentation, or filtration. Rather, a process must be incorporated that will remove the offending materials through adsorption. The following alternatives are provided for consideration:

Powder Activated Carbon (PAC). The City currently utilizes powder activated carbon in an effort to control taste and odor concerns when they arise. The current method is to add PAC to a drum of water and constantly mix the drum to form a PAC slurry. The slurry is then injected into the raw water stream where the offending materials adhere (adsorb) to the PAC particles. The PAC then settles out in the clarification process and is removed to the backwash lagoon when the sludge blanket is drawn down.

The use of PAC is relatively messy and inconvenient for the operators though it provides relatively effective results. The capital equipment required is relatively inexpensive and the application is not sophisticated.

More sophisticated feed systems can be installed utilizing a dry hopper feed design. These systems, while more automatic, continue to be messy to use and tend to have high maintenance costs due nature of the gritty material being fed.

Granular Activated Carbon (GAC). Like PAC, granular activated carbon removes offensive taste and odor materials through adsorption. GAC differs from PAC in that GAC is generally comprised of much larger pieces of activated carbon. GAC can be added to the top of a conventional multi-media filter as a special taste and odor removal layer. Some communities elect to install a GAC cap on the filters in place of typical anthracite media. If using GAC in a filter, accommodations must be made to prevent the GAC from being washed out of the filter during backwashes. This is especially the case with air-scour backwash systems. If the City of Lowell elects to continue using conventional filtration, a GAC cap could be installed in each filter, assuming there is adequate available head room. Maintenance of the GAC cap simply requires the addition of a maintenance amount of GAC each year or two, such as is the case with regular anthracite.

Another way of utilizing GAC is through the use of a GAC tower or GAC packaged filter. These systems operate like a pressure filter but utilize GAC for the media. Typically, a GAC filter would be used at the end of the treatment process just before disinfection and before the finished water is chlorinated.

The estimated price to add a layer of GAC in three filters is between three and six thousand dollars depending on the amount of anthracite or other media that must be removed from the existing filters.

In Lowell, it would be difficult to utilize GAC pressure filters after the conventional filters as the filter effluent would have to be pumped through GAC pressure filters. However, it is possible to utilize a pump to push the finished water through the GAC pressure filter and then on to the clearwell.

A better application of a GAC pressure filter would be to use it after a membrane process where the filtrate is already under pressure enough to force it through the GAC filter. The water coming out of the membranes is very clean so the GAC filters would not foul with debris. However, the pressure filters would adsorb any remaining offending taste and odor materials prior to the disinfection process. This would eliminate the current practice of using a messy PAC slurry.

The cost of installing a GAC pressure filter system ranges from $35,000 to $50,000 depending on whether or not additional pumping is required to push the finished water through the filters and whether the filters will be pretreatment or final treatment.

Miscellaneous Plant Upgrades

A number of other plant deficiencies should be improved as part of an overall project. They are mentioned in this section for information and costs will be included below in the overall alternatives cost estimates as appropriate.

New Raw Water Pumps. As discussed above, new raw water pumps must be included to provide the design flow into the plant. New valves, fittings, and appurtenances may also be required for the installation of larger pumps. VFD's should be installed to operate the new pumps to allow for flexibility in operation.

Flow Metering Equipment. As mentioned previously, new flow meters should be installed on the raw water piping, finished water piping, and backwash piping.

New Valves and Actuation. Many of the existing valves and actuators have proven to be a maintenance problem. For any upgrade alternative, new valves and actuation equipment should be included. Quality valves and actuators should be installed with any upgrade to reduce the amount of maintenance issues related to these critical components.

Controls Upgrade. The existing control system should be expanded to include recording and trending and other common SCADA features. Information about historical plant operation should be permanently stored on the control computer to be recalled at any time.

Instrumentation Sampling Upgrade. The last plant upgrade included the installation of new turbidimeters, chlorine analyzers, pH analyzers, and other instrumentation equipment. Small booster pumps were installed on sample lines coming from each filter outlet piping. Relatively large (1" diameter) copper sample piping was plumbed around the treatment plant to the instrumentation. The larger diameter sample piping and the length of pipe between the sample point and the instruments, coupled with the very small sample flow rate required by each instrument have created a situation where there is a tremendous "lag" between the sample point and the instrument. Also, due to the low velocity in the sample piping, small amounts of sediment can deposit themselves in the piping. Then, when an operator "opens up" the sample line in the control room sink or elsewhere in the plant, the sediment is swept down the piping and into the instrumentation (i.e. turbidimeters). This often results in a high instrument reading and usually causes the plant to alarm and shut down, thinking that a violation has occurred.

To correct this problem, the plant upgrade should consider relocating the instrumentation sensors closer to the sample points and/or using smaller diameter piping for the sample lines to increase the velocity in the sample piping.

Depending on the final treatment process selection and configuration, costs should be included to correct this deficiency in the existing instrumentation.

Reuse of Existing Components. Should it be determined that membrane treatment is preferred over the existing conventional filtration process, efforts should be made during design to identify system components that can be reused or recycled into the new treatment process. This is especially true with instrumentation, pumps, and other system components.

New Finished Water Pumps. To match the influent flow to the plant, new pumps will be required to lift finished water into the system. These pumps will most likely be vertical turbine style pumps with VFD's to allow for flexibility in operation.

Complete Treatment Alternatives

After considering all of the treatment process component alternatives above, complete treatment system alternatives must be considered that combine various individual components resulting in an overall treatment process to provide for the projected treatment needs of the City.

The following complete treatment alternatives were assembled for consideration.

1. Complete Treatment Alternative No. 1: Use of Existing Conventional Filters with new Clarification. Under this alternative, the plant would be upgraded utilizing conventional treatment technologies and using as much of the existing facilities as possible. Included in this alternative scenario is:

- New raw water pumps and a new building to house equipment including raw water pumps, new finished water pumps, on-site chlorine generation equipment, chemical storage and metering equipment, and other general equipment. Chemical injection and rapid mixing to take place in building.

- New piping and metering from raw water pumping equipment to new sedimentation basin.

- New concrete sedimentation basin including two-stage flocculation, tube settlers, launders, and related appurtenances.

- New piping to provide gravity flow from new sedimentation basin to existing clarifier. Clarifier to provide a quiescent zone prior to water entering the filters.

- The two existing filters will be upgraded to provide air-scour backwash for a more effective backwashing system. The existing third filter bay will be constructed and all three filters will receive a GAC cap for taste and odor issues. Special fins will be required on the backwash troughs to reduce the amount of loss of GAC and anthracite material. A blower will be installed in the existing raw water pumping room.

- Existing and new piping will transmit filtrate to the existing clearwell. Disinfectant from new on-site chlorine generation housed in the new building will be injected in this piping run prior to the clearwell.

- Gravity piping will carry water from the existing clearwell to the new clearwell located under the new building slab. The new clearwell will also be baffled to provide long residence time.

- New high-level system pumps will be installed in the new building to lift water into the system. These pumps will be vertical turbine pumps each capable of delivering in excess of 525 gpm.

- Project to include upgrades to controls, electrical improvements, piping elements, valves, actuators, and some metering equipment.

Figure 7.3-1 below illustrates the process for Alternative 1. The diagram shows, in blue, the existing flow patterns and existing flow components. New flow patterns and system components are shown in red.

Table 7.3-6 below presents the project costs for development of Alternative No. 1. Note that the project cost estimate includes engineering, contingency, and other related project costs in addition to construction costs.

Table 7.3-6 – Complete Treatment Alternative No. 1 Cost Estimate – Conventional Filtration & Sedimentation

Alternative 1: Complete Treatment Alternative - Conventional Filtration					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$100,000.00	$100,000.00
2	Construction Facilities/Temporary Systems	ls	1	$75,000.00	$75,000.00
3	Addition of third filter	ls	1	$15,000.00	$15,000.00
4	Conversion of plant to air scour backwash & blower	ls	1	$50,000.00	$50,000.00
5	Construction of conventional sedimentation basin	ls	1	$535,000.00	$535,000.00
6	Piping Improvements in plant and on site	ls	1	$80,000.00	$80,000.00
7	Controls & Instrumentation upgrades	ls	1	$50,000.00	$50,000.00
8	Installation of a GAC Cap in filters	ls	1	$5,000.00	$5,000.00
9	Construction of a new concrete clearwell	ls	1	$198,250.00	$198,250.00
10	New raw water & finished water pumping upgrades	ls	1	$35,000.00	$35,000.00
11	On-site chlorine disinfection system	ls	1	$38,200.00	$38,200.00
12	Flow metering and valve upgrades	ls	1	$50,000.00	$50,000.00
13	Electrical Improvements	ls	1	$100,000.00	$100,000.00
14	Small building over new clearwell for misc. equipment	lf	500	$150.00	$75,000.00
		Construction Total			$1,406,450.00
		Contingency (20%)			$281,290.00
		Subtotal			$1,687,740.00
		Engineering (18%)			$303,793.20
		Administrative costs (3%)			$50,632.20
		Total Project Costs			**$2,042,165.40**

Fig 7.3-1 – Alternative No 1 – to be inserted from AutoCAD print

2. Complete Treatment Alternative No. 2: Use of Packaged Membrane Filtration. Under this alternative, the plant would be upgraded to utilize membrane treatment technology as the best available technology. Under this alternative, new sedimentation equipment is not required. The following project components are included within this alternative:

- New raw water pumps and a new building to house equipment including raw water pumps, on-site chlorine generation equipment, chemical storage and metering equipment, and other general equipment. Chemical injection and rapid mixing to take place in building.

- New piping to distribute flow to the existing clarifier. Clarifier to provide a quiescent zone prior to water entering the membranes. Additional piping to return water from clarifier back to new building. Piping and valves should be set up so that the clarifier can be bypassed when water quality allows.

- Coarse screening (Amiad or similar) should be provided to provide removal of large debris from the raw or partially clarified water and to protect the membrane from damage. Depending on the configuration and equipment selected, a booster pump may be required to increase pressure through the pre-filtration equipment.

- New membrane treatment equipment provided in packaged, skid-mounted configurations. The equipment will be self contained on one or two skids depending on the equipment that is selected.

- New piping will transmit flows from the membrane equipment to a GAC filtration unit for taste and odor. This component is optional and could be substituted with a powder activated carbon feed into the raw water stream. If GAC is used, accommodations must be made in the pumping systems to provide adequate head through the GAC filters.

- New piping will transmit filtrate to the new baffled clearwell. Disinfectant from new on-site chlorine generation housed in the new building will be injected in this piping run prior to the clearwell.

- Gravity piping will carry water from the new clearwell to the existing clearwell located under the new building slab. The new clearwell will also be baffled to provide long residence time.

- New high-level system pumps will be installed in the existing building to lift water into the system. These pumps will be vertical turbine pumps each capable of delivering in excess of 525 gpm.

- Project to include upgrades to controls, electrical improvements, piping elements, valves, actuators, and some metering equipment.

Figure 7.3-2 (following) illustrates the process flow of water under the Alternative 2 plan using membrane technology. As before, existing system components are shown in blue and new components are shown in red.

A cost estimated is for the packaged membrane treatment alternative is provided below in Table 7.3.7.

Table 7.3-7 – Complete Treatment Alternative No. 2 Cost Estimate – Membrane Treatment

Alternatives No. 2 - Membrane Treatment w/ GAC Pressure Filters					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$75,000.00	$75,000.00
2	Construction Facilities/Temporary Systems	ls	1	$50,000.00	$50,000.00
3	Constructing of new concrete clearwell	ls	1	$198,250.00	$198,250.00
4	New Building to house membrane and other equipment	sf	1000	$150.00	$150,000.00
5	New raw and finished water pumping equipment	ls	1	$35,000.00	$35,000.00
6	New pre-filtration equipment (Amiad or similar)	ls	1	$40,000.00	$40,000.00
7	Membrane packaged treatment equipment	ls	1	$500,000.00	$500,000.00
8	Piping improvements in plant and on site	ls	1	$60,000.00	$60,000.00
9	Controls and instrumentation upgrades	ls	1	$35,000.00	$35,000.00
10	GAC pressure filters after membrane	ls	1	$40,000.00	$40,000.00
11	Onsite chlorine generation equipment	ls	1	$38,200.00	$38,200.00
12	Flow metering and valve upgrades	ls	1	$20,000.00	$20,000.00
13	Electrical improvements	ls	1	$100,000.00	$100,000.00
		Construction Total			$1,341,450.00
		Contingency (20%)			$268,290.00
		Subtotal			$1,609,740.00
		Engineering (18%)			$289,753.20
		Administrative costs (3%)			$48,292.20
		Total Project Costs			**$1,947,785.40**

Figure 7.3-2 Alternative 2 Membrane treatment flow process – insert cad drawing here

3. <u>Complete Treatment Alternative No. 3: Interim Improvement Alternative</u>. Under this alternative, only minor improvements would be made to expand the capacity of the treatment facility while not having to construct any significant new system components. Under this alternative, the following improvements would be required.

- New raw water pumps to be installed in existing raw water building. Fittings and piping to be upgraded as necessary to make installation. Use VFD's to allow for flexibility and control to balance incoming and outgoing flows.

- New piping to deliver flows to the existing clarifier. Clarifier to provide a quiescent zone prior to water entering the filters. Clarifier may not operate well at higher flow rates.

- Construct third filter in existing empty bay. Utilize surface wash for backwash. Add piping, valves, fittings, and other necessary components to complete the system.

- Add new or additional baffling and inlet components to the clearwell to improve CT. Change operating programming to maintain a fuller clearwell volume to increase the residence time in the clearwell.

- Add new finished water pumps with VFD's. Add new piping and fittings as necessary.

- Update controls to provide for recording and enhanced SCADA capabilities. Change program to facilitate to operation.

Figure 7.3-3 on the following page illustrates the process flow of water under the Alternative 3 plan using interim upgrades to increase the capacity of the plant. As before, existing system components are shown in blue and new components are shown in red. A cost estimate for this option is provided as follows:

Table 7.3-8 – Complete Treatment Alternative No. 3 Cost Estimate – Interim Treatment Improvements

Alternative No. 3 - Interim Treatment Measures					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$35,000.00	$35,000.00
2	Construction Facilities/Temporary Systems	ls	1	$20,000.00	$20,000.00
3	Construction of third filter w/ surface wash	ls	1	$20,000.00	$20,000.00
4	Addition of GAC cap	ls	1	$5,000.00	$5,000.00
5	On-site chlorine generation equipment	ls	1	$38,200.00	$38,200.00
6	Clearwell upgrades	ls	1	$30,000.00	$30,000.00
7	Piping, valve, actuator, and metering improvements	ls	1	$60,000.00	$60,000.00
8	Controls & Instrumentation upgrades	ls	1	$40,000.00	$40,000.00
9	Electrical upgrades	ls	1	$35,000.00	$35,000.00
10	New raw and finished water pumping & related equip.	ls	1	$35,000.00	$35,000.00
		Construction Total			$318,200.00
		Contingency (20%)			$63,640.00
		Subtotal			$381,840.00
		Engineering (18%)			$68,731.20
		Administrative costs (3%)			$11,455.20
		Total Project Costs			**$462,026.40**

Figure 7.3-3 – Complete Treatment Alternative No. 3 – Interim Improvements – Insert CAD drawing here

4. <u>Complete Treatment Alternative No. 4: Use of Packaged Conventional Treatment</u>. Under this alternative, the plant would be upgraded to utilize a packaged conventional treatment process. The following project components are included within this alternative:

- New raw water pumps and a new building to house equipment including raw water pumps, on-site chlorine generation equipment, chemical storage and metering equipment, and other general equipment.

- New piping to distribute flow to the existing clarifier. Clarifier to provide a quiescent zone prior to water entering the packaged plant. Additional piping to return water from clarifier back to new building. Piping and valves should be set up so that the clarifier can be bypassed when water quality allows.

- Coarse screening (Amiad or similar) should be provided to provide removal of large debris from the raw or partially clarified water prior to the packaged treatment system.

- New packaged conventional treatment equipment provided in a packaged, skid-mounted configuration. The equipment will be self contained on one or two skids depending on the equipment that is selected.

- New piping will transmit flows from the packaged equipment to a GAC filtration unit for taste and odor treatment. This component is optional and could be substituted with a powder activated carbon feed into the raw water stream. If GAC is used, accommodations may be required such as a simple pumping system to provide adequate head through the GAC filters.

- New piping will transmit filtrate to the new baffled clearwell. Disinfectant from new on-site chlorine generation housed in the new building will be injected in this piping run prior to the clearwell.

- Gravity piping will carry water from the new clearwell to the existing clearwell located under the new building slab. The new clearwell will also be baffled to provide long residence time.

- New high-level system pumps will be installed in the existing building to lift water into the system. These pumps will be vertical turbine pumps each capable of delivering in excess of 525 gpm.

- Project to include upgrades to controls, electrical improvements, piping elements, valves, actuators, and some metering equipment.

The layout of the packaged conventional option would be similar to the layout presented for the packaged membrane equipment described under Complete Alternative No. 2 with conventional equipment substituted in for the membrane equipment.

A preliminary cost estimate is provided in Table 7.3-9 for Alternative 4.

Table 7.3-9 – Complete Treatment Alternative No. 4 Cost Estimate – Packaged Conventional Treatment Alternative

Alternatives No. 4 - Packaged Conventional Treatment Process					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$80,000.00	$80,000.00
2	Construction Facilities/Temporary Systems	ls	1	$55,000.00	$55,000.00
3	Constructing of new concrete clearwell	ls	1	$198,250.00	$198,250.00
4	New Building to house new treatment equipment	sf	1200	$150.00	$180,000.00
5	New raw and finished water pumping equipment	ls	1	$35,000.00	$35,000.00
6	New pre-filtration equipment (Amiad or similar)	ls	1	$40,000.00	$40,000.00
7	Conventional Packaged Treatment Equipment	ls	1	$400,000.00	$400,000.00
8	Piping improvements in plant and on site	ls	1	$60,000.00	$60,000.00
9	Controls and instrumentation upgrades	ls	1	$35,000.00	$35,000.00
10	GAC pressure filters for taste and odor	ls	1	$40,000.00	$40,000.00
11	Onsite chlorine generation equipment	ls	1	$38,200.00	$38,200.00
12	Flow metering and valve upgrades	ls	1	$20,000.00	$20,000.00
13	Electrical improvements	ls	1	$100,000.00	$100,000.00
		Construction Total			$1,281,450.00
		Contingency (20%)			$256,290.00
		Subtotal			$1,537,740.00
		Engineering (18%)			$276,793.20
		Administrative costs (3%)			$46,132.20
		Total Project Costs			**$1,860,665.40**

Additional alternatives were considered but were not found to be viable. Various sub-alternatives are possible within the two main alternatives presented. These sub-alternatives should be reviewed in more detail during final design for the project.

Summary and Recommendations – Treatment

As illustrated above, the City has a number of alternatives available to them when considering improvements for individual components as well as complete treatment alternatives. The following summarize the three complete alternatives presented along with the pros and cons of each alternative.

Alternative 1 – Existing Conventional Treatment: In the first alternative, the project would utilize the existing filters plus a third filter to meet the projected demands for the system. However, to meet these demands, a relatively large investment will have to be made to add a sedimentation basin and a new clearwell at the plant site in addition to the third filter. The estimated total project cost for this alternative is around $2.04-million dollars.

The pro's of this alternative is that the existing conventional process is familiar to the City's operators and has proven to provide adequate treatment for the City's water supply. This alternative also makes the fullest use of the existing facilities though several new treatment components are required to obtain the projected flow rates needed for future water demands.

The con's of this option include the fact that conventional filtration does not represent the best available technology or highest level of available treatment equipment available today. It may be unwise for the City to commit their resources for several decades for a treatment technology that is not the best current technology. As water quality requirements have increased, other technologies have emerged as better treatment alternatives. This alternative (Alternative 1) is also the most expensive, though only slightly more than the estimate prepared for Alternative 2.

Alternative 2 – Membrane Treatment: In the second alternative, a complete treatment process was developed centered around utilizing membrane treatment. This alternative will utilize much of the existing treatment system components while adding a new clearwell and membrane filtration system. The estimated total project cost for this alternative is around $1.95-million dollars.

The pro's of this option are that the best available technology is being utilized for a long-term investment in the City's water treatment process. Membrane treatment is widely used and provides the highest level of treatment performance available in the market today. Through this option, the City would also be utilizing nearly all of their existing treatment system components with the exception of the two existing 6'x10' conventional filters. Therefore, it is not as though the City would be throwing away its recent investments in the existing plant.

The con's of this alternative are few except that this alternative, while less expensive than the first alternative, still represents a significant investment in the City's water treatment plant.

Alternative 3 – Interim Improvements: In the third complete alternative, a plan is provided to increase the flow through the plant while making a minimum amount of improvements. In this alternative, the third filter is added along with improvements to the clearwell and changes to the programming. Through these efforts, it is estimated that between three and four hundred gallons per minute could be produced by the plant under normal operating conditions. Based on water demand projections, this project would allow the City to produce water for between 10 and 15 years into the planning period. At which time, if growth has occurred as projected, new and major improvements would be required to produce water at a higher rate. The estimated cost for this project is around $462,000.

The pro's of this alternative obviously include a significantly lower project cost. Existing water production (~160 gpm) can be more than doubled for a relatively small cost. Also, nearly all previous investments in the plant can continue to be used under this alternative.

The con's of this alternative must include a lower level of confidence in the treatment process. As discussed previously, the City's clarifier is not of a typical or conventional design. Therefore, the performance of the clarifier at higher flow rates is difficult to predict. It is possible that that clarifier will not function well at higher flow rates. This would place a greatly increased burden on the filters, resulting in increased backwash frequency and, potentially, reducing the capacity of the plant upgrade. Also, at some point within the planning period, the plant will require another upgrade. The future upgrade will be significant as it will require the construction of a sedimentation basin or the installation of membrane equipment as discussed in the first two alternatives. It also will require the construction of a new clearwell.

This alternative simply provides a way for the City to increase production while putting off a major plant upgrade for several years. This alternative should not be considered as a permanent solution.

Alternative 4 – Packaged Conventional Treatment Alternatives: In the fourth and final complete treatment alternative, a plan is provided to establish a treatment facility around a packaged conventional process such as the Trident Microfloc. This alternative will use the existing clarifier only for pre-settling and pretreatment seasonally as required.

The pro's of this alternative is that conventional treatment is relatively familiar to the operations staff. The equipment is also slightly less expensive than packaged membrane treatment equipment.

The con's of this alternative include the fact that the packaged treatment equipment is older technology and not capable of the consistent and full-proof high quality finished water that is possible with membrane equipment. Conventional equipment also will require higher chemical use (cost), more operational interaction from staff to ensure chemical balance, as well as the potential for filter break through and turbidity violations. Generally, conventional equipment is considered to be older and less effective technology when compared to membrane equipment.

Complete Treatment Recommendation: Ultimately the City must choose which path they wish to follow. All of the alternatives presented will provide a good level of treatment, through the third alternative will provide service for a shorter period of time and will require an additional upgrade in the future.

If the City wishes to invest a small amount in the plant now and increase the capacity of what they have without building new facilities, the third alternative is recommended. This recommendation comes with a warning that the existing clarifier may not operate well under some conditions and at higher flow rates. Therefore, increasing the flow through the existing clarifier bears some risk, though required treatment levels will likely be obtained through the three simultaneously operating filters.

Should the City wish to invest in their treatment facilities now for the entire planning period, Alternative 2 is the recommended option. Alternative 2 makes use of the best available technology and will meet the City's quality and quantity needs for the entire projected planning period (25 years). This alternative also makes good use of the existing facilities while still providing membrane filtration for the highest water quality.

Additional discussion on the recommendations for treatment and the total project costs is provided in Section 8.

Discussion of Comparable Operating Costs for Complete Treatment Alternatives

It is difficult to place exact dollar figures on the potential operating costs for each alternative without an effort that is beyond the scope of this planning project. However, an effort will be made to provide a discussion of the comparable operating costs for each option using the existing facilities as a measuring stick.

Operating Costs: Alternative 1- Expansion of Existing Conventional Equipment- The City has been operating their existing water plant for several years and has a good understanding of the operating costs that they currently operate under. Expansion of the existing facilities will not greatly change the operating costs from what they currently are. Potential impacts to the day to day operating costs include:

- Slightly higher chemical costs as more water will be put through the facilities as demands increase.
- Slightly higher electrical costs due to the addition of new pumps and additional pumping that may be required. However, new super efficient motors and shorter operating days may prove to actually reduce the electrical costs from current levels.
- Manpower costs should stay about the same or, potentially, be reduced as the more efficient treatment process will no longer be required to operate for long hours. This should reduce extended shifts, overtime, and weekend manpower costs.
- Costs for backwash water are likely to remain about the same. As there will be more filter area to backwash, more water will be required. However, the time between backwashes should be

increased due to decreased loading. The resulting change to the amount of water needed should be minor. If air scour backwash is introduced, the volume of water required for each backwash can be significantly reduced.

- More filters mean more filter media at the plant. Filter media is lost on an average of around 1" of anthracite per year. A slight increase in operating costs will be required to maintain proper media levels. Also, media should be replaced between 10 and 20 years of service depending on the loading of the filters and the operation and maintenance of the filters. With more filter area, replacement costs will be greater, though the cost is minor if amortized over more than 10 years.

Operating Costs: Alternative 2 – Packaged Membrane Equipment: Under this alternative, we find that small water systems that go from conventional treatment processes to new packaged membrane equipment typically find their operating costs remain about the same or actually fall below existing levels. Potential impacts and considerations for impacts to operating costs under this alternative include:

- Potential reduction in electrical costs due to the use of premium duty motors.

- Reduction in chemical costs with the potential to eliminate the use of coagulation chemistry completely.

- As is the case with filter media, membrane media must be replaced on a similar schedule. Also, as with filter media, if amortized over many years, the cost of membrane replacement is not significant.

- Manpower costs for membrane equipment should be reduced as an operator is not required to adjust chemical feed rates, rake filters, manually operate a backwash cycle, monitor turbidity, perform jar testing, or many other operational activities. A membrane treatment system should free up some of the operator's time to complete other tasks in the City. Also, the plant will be sized for shorter run times greatly reducing or eliminating overtime or extended work hour costs.

Operating Costs: Alternative 3 – Interim Improvements: Under this alternative, no significant changes are planned. However, the following considerations may result in minor changes to the operating costs for the plant:

- New pumps with premium efficiency motors could result in reduced energy costs along with shorter run times due to the higher flow rates.

- Chemical costs will increase slightly as the amount of water treated increases. This will, however, be a minor impact on costs.

- There should be a reduction in manpower costs in terms of the amount of hours the plant will have to operation. However, as the operations staff have many other duties to complete within the City, it is not likely that a real savings will be realized. It is likely that overtime and additional shifts could be reduced due to the increased capacity of the plant. This will, however, diminish as demands increase and approach the new plant capacity.

Operating Costs: Alternative 4 – Packaged Conventional Treatment Equipment: Under this alternative, operating costs will be similar to those experienced under the conventional alternative described by Alternative No. 1. Considerations that may affect operating costs under this alternative include:

- Improved motor efficiency and lower electrical costs for using premium duty motors.

- Greater chemical costs than the membrane option as coagulation chemicals are required for flocculation process.

- Manpower costs for conventional treatment will be greater than the membrane alternative as any conventional process will require more operator attention and time than a membrane process will require. However, as the city employs operators that have responsibilities at numerous city facilities, there will be no real appreciable savings for manpower costs.

Operating Cost Summary: There is no appreciable difference or advantage that will tip the scales to one alternative or the other based on operating costs alone. While some alternatives offer the opportunity for likely lower operating costs, the potential savings should not be significant enough to affect the selection of one process over another.

7.4 Treated Water Storage

As developed in Section 6.2, the City must develop reserves for the following major categories:

1. Equalization Reserves: accommodates for the difference between high and low water uses in the system on a day to day basis. This amount is typically set to 25% of the projected MDD.

2. Emergency Reserves: this storage accommodates for failures in the system, treatment facility, or raw water supply that would result in the City being unable to produce water and replenish their reserves. This amount is typically set to one full projected MDD.

3. Fire Reserves: this storage should be in the reservoirs at all times to fight a major fire in the community. This reserve should be considered over-and-above the reserves for emergency and equalizing storage. For this planning effort, this amount has been identified as 360,000 gallons (2,000 gpm for 3 hours).

With a projected MDD of 693,520 gallons, the required storage identified for this planning period is 1.23 MG (see Section 6.2). Existing reserves total 500,000 gallons leaving a reserve shortfall of 730,000 gallons. Additional reserves must be added to the system to satisfy planning requirement for the projected planning period.

The reservoir siting study completed in 2001 (Systems West Engineers, Inc.) identified a preferred site for a new reservoir on the west site of the City to the north of the existing reservoir site at an elevation of around 1,166 feet. As additional reservoir siting efforts were beyond the scope of this project, we have not identified other locations where other reservoirs or additional reservoirs could be constructed if locating a single large reservoir at the identified location proves to be impractical or improper.

Recommended Reservoir Sizing and Location

As discussed above, the City requires an additional 730,000 gallons of finished water storage to satisfy the projected storage planning requirements. This may be provided through one or multiple reservoirs.

However, locating a large reservoir at the new high level location presents some serious operational problems. These are summarized as:

1. Filling the larger tank will remove water from the lower tank and lower pressure level.

2. A PRV (pressure reducing valve) will be required if the upper system and the lower system are to be interconnected so that the larger upper reservoir can provide storage reserves for the lower system.

3. If the lower level tank level is reduced during the filling of the upper tank, flows from the upper tank will recirculate to fill the lower tank again. This will result in wasted energy use when energy is used to lift water from the lower tank to the upper tank only to have it flow by gravity back into the lower tank.

4. Complex controls could be installed to prevent recirculation, though they will likely create problems of their own. Fire, water line breaks, maintenance efforts, and other events not included in the complex control logic could result in problems with the reservoir systems.

5. A large reservoir in the upper system may not "turn over" often enough to maintain "fresh" water in the reservoir. In other words, the water may become stagnant.

Normally, a water system will utilize larger reservoirs in the lower or main pressure level and use booster stations to lift water into small upper reservoirs that are not able to flow by gravity into the lower system. This eliminates the problems described above and ensures that the majority of the water use, and therefore water turnover, takes place in the larger reservoirs in the lower system. The smaller reservoirs in the upper system are sized for their individual service areas and not to provide additional reserves for the system at large.

While a siting study was beyond the scope of this planning effort, it is important that potential reservoir siting scenarios are established.

Site Alternative No. 1: The preferred site for the new reservoir is on the opposite side (northwest) of the community from the existing reservoir on property currently owned by the Seneca Lumber Company. This property is currently being considered for expansion of the UGB to allow for future growth. Topographic maps indicate that there are several potential sites on the property that would provide the proper elevation for the construction of the tank on the Seneca property. Considerations for putting the tank on the Seneca property should include:

1. The tank should be constructed at the same elevation as the existing tank so the lower and main pressure level would benefit from the additional storage (550,000-gallon).

2. A smaller tank will still need to be constructed on the east side to service the upper pressure level there. This will also require a booster pump station.

3. Another smaller tank may be required in the upper pressure level in the area of the Seneca property to service develop at or above the level of the proposed reservoir. Another booster pump station would be required to service this smaller reservoir. The size of the reservoir should be determined once the development plan for the property is established. However, for now, it is safe to assume that a 180,000-gallon reservoir, such as is described for the eastern upper area, would be adequate for the upper pressure level on the Seneca property.

4. Additional piping may be required to properly connect the new reservoir (on the Seneca site) to the distribution system depending on the final location selected.

Site Alternative No. 2: An alternative site, but equally as appropriate, was identified in a siting study previously undertaken by the City. Figure 3.3 on Page 7-29 has been adapted from the City's previously completed reservoir siting plan and is provided on the following page to illustrate this siting alternative.

The following recommendations are indicated on the figure and are described below in more detail:

1. One large reservoir located at the selected "Site C" is not recommended. This would result in stagnancy issues, pressure control problems, and would not provide the level of service required by the new finished water reserves.

2. Rather, two reservoirs are recommended. A total reserve capacity of around 730,000-gallons is required. The upper reservoir (Site C) should be sized to provide service to the subdivision or upper level pressure area only. This upper tank should not be depended on to provide service to the lower service level. Therefore, it is recommended that this upper reservoir be sized at around 180,000 gallons. This should, however, be confirmed once the final development plans for the upper service area are known. However, this size of reservoir should be adequate for a significant upper level development.

3. With a 180,000-gallon reservoir in the upper pressure level, an additional 550,000-gallon reservoir should be located within the lower pressure level at the same elevation as the existing tank. The original siting study recommended a pipe be extended from 4th and Hyland to fill and empty a new reservoir at Site C. This approach would still be appropriate and is recommended for a dual-tank approach.

4. Therefore, a 550,000 gallon tank could be constructed on the originally proposed alignment at the same elevation at the existing 500,000 gallon tank. The two tanks would rise and fall together in the system and provide pressure and service to the lower pressure level where the majority of the water system is located.

5. A pipe would be installed out of the new 550,000 gallon tank and connected to a booster pump station that would boost water to the new 180,000 gallon tank at the original Site C. The pipe to the Site C tank would be for filling only and would not connect the tank back into the system though a manual bypass could be included to provide that option if it were required under emergency or special conditions. This will eliminate the need for a PRV vault between the upper and lower system.

Depending on the timing of the development, the availability of land, and the requirements for interconnecting the tank to the distribution system, it would be preferable to locate the new reservoir on the Seneca property. This will "spread" the City's reserves out rather than locating them all on the same side of town.

However, if conditions and timing do not coincide with the City's plans, siting the reservoir as discussed previously is also acceptable.

The recommended location for the first siting option (east side option) is shown on the following page on a figure labels Figure 3.3 as reproduced from the City's previously completed reservoir siting plan. The proposed siting recommendations are shown in the hand-drawn additions to the figure.

A separate figure has not been provided for the Seneca property siting as that is beyond the scope of this planning effort. Cost estimates for each part of the recommended system are provided hereafter.

Figure 3.3 from Reservoir Siting Study – Hand Drawn changes to siting figure – Insert Here

Reservoir Material Alternatives

Potable water reservoirs are constructed of a variety of materials and configurations. Tanks are generally cylinders though square tanks are occasionally constructed. Most tanks are above ground while some are constructed partially or fully underground.

In the northwest, reservoirs are typically constructed of reinforced concrete or steel. These two options are further discussed below.

Reinforced Concrete. Reservoirs constructed of reinforced concrete are typically characterized as having a very long life with little need for maintenance. However, if improperly designed or constructed, a reinforced concrete reservoir can be as big a maintenance problem and have as short of a life span as any other type of reservoir. For example, the existing reservoir in Lowell was constructed with a number of "cold joints" that formed when different batches of concrete arrived on the site too far apart and were not vibrated or otherwise mixed to integrate the batches. The resulting leaks at each joint are visible on the surface of the tank today. If left unchecked, the leakage has the potential to corrode the reinforcing steel in the wall sections, weakening the tank.

If designed and constructed appropriately, a reinforced concrete tank should be expected to have a useful life of 50 to 80 years if properly maintained.

The following cost estimate is for the construction of the new 550,000 gallon reservoir. Piping costs to the reservoir are included though costs for the booster pump station and piping to the upper reservoir are not included with this project. As it is not financially feasible to construct smaller reservoirs of reinforced concrete, a price for the smaller tank was not developed for construction the smaller reservoir of concrete.

Table 7.4-1 – Cost Estimate for Reinforced Concrete Reservoir (550,000-gallon)

Reinforced Concrete Reservoir - 550,000 gal					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$90,000.00	$90,000.00
2	Construction Facilities/Temporary Systems	ls	1	$65,000.00	$65,000.00
3	Site Preparation & Excavation	ls	1	$50,000.00	$50,000.00
4	Site Piping & Appurtenances	ls	1	$25,000.00	$25,000.00
5	Reinforced Concrete Reservoir (0.55 MG)	ls	1	$750,000.00	$750,000.00
6	Fencing	ls	1	$15,000.00	$15,000.00
7	Telemetry	ls	1	$10,000.00	$10,000.00
8	10-inch piping to reservoir	lf	2400	$65.00	$156,000.00
9	Roadway & Site improvements (crushed rock)	ls	1	$15,000.00	$15,000.00
		Construction Total			$1,176,000.00
		Contingency (20%)			$235,200.00
		Subtotal			$1,411,200.00
		Engineering (18%)			$254,016.00
		Administrative costs (3%)			$42,336.00
		Total Project Costs			**$1,707,552.00**

Glass-Fused-to Steel Reservoir. Steel tanks are commonly constructed across the northwest to be used for treated water storage. However, steel alone will corrode and fail very quickly without adequate protection. Many tank suppliers utilize epoxy and other coatings to protect the steel. The results of coated steel tanks vary depending on the coating material, the application, and the maintenance of the tanks over the years. Generally, coated steel tanks are assumed to have a useful life of 20-40 years, depending on the level of maintenance practiced with a particular tank.

Another steel tank option that has gained wide acceptance as a less costly yet highly reliable storage option is that of a glass-fused-to-steel (GFS) reservoir. In a GFS reservoir, fiberglass is bonded to steel plates that are used to construct the reservoir.

Characterized by blue tank walls and bright aluminum dome roofs, the GFS reservoirs are reliable, require little maintenance, and are more affordable then their concrete counterparts. While it is not likely that GFS reservoirs will enjoy 80-year life spans, it is likely to expect life spans up to 50 years if the tank is properly cared for. The GFS reservoirs resist corrosion, but unlike concrete reservoirs, they can and are regularly damaged by gunshots. Repair kits for this type of damage are available from most manufacturers.

Aqua-Store provides GFS tanks throughout the northwest that have had a history of reliable service. Aqua-Store was contacted and asked to provide a proposal for the City of Lowell.

Cost estimates are provided below for GFS reservoirs in both the 550,000 gallon and 180,000 gallon size. The 180,000 gallon reservoir cost estimate includes costs for a new booster pump station and the related appurtenances. A description of the booster pump station and required piping interconnections is provided on the following pages.

Table 7.4-2 – Cost Estimate for Glass-Fused-to-Steel Reservoir (550,000 gallon)

Glass Fused to Steel Reservoir - 550,000 gal					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$60,000.00	$60,000.00
2	Construction Facilities/Temporary Systems	ls	1	$35,000.00	$35,000.00
3	Site Preparation & Excavation	ls	1	$50,000.00	$50,000.00
4	Site Piping & Appurtenances	ls	1	$25,000.00	$25,000.00
5	Glass-Fused-to-Steel Reservoir (0.55 MG)	ls	1	$350,000.00	$350,000.00
6	Fencing	ls	1	$15,000.00	$15,000.00
7	Telemetry	ls	1	$10,000.00	$10,000.00
8	10-inch piping to reservoir	lf	2400	$65.00	$156,000.00
9	Roadway & site improvements (crushed rock)	ls	1	$15,000.00	$15,000.00
		Construction Total			$716,000.00
		Contingency (20%)			$143,200.00
		Subtotal			$859,200.00
		Engineering (18%)			$154,656.00
		Administrative costs (3%)			$25,776.00
		Total Project Costs			**$1,039,632.00**

Table 7.4-3 – Cost Estimate for Glass-Fused-to-Steel Reservoir & Booster Pump Station (180,000 gallon)

Glass Fused to Steel Reservoir & Pump Station - 180,000 gal					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$45,000.00	$45,000.00
2	Construction Facilities/Temporary Systems	ls	1	$25,000.00	$25,000.00
3	Site Preparation & Excavation	ls	1	$30,000.00	$30,000.00
4	Site Piping & Appurtenances	ls	1	$20,000.00	$20,000.00
5	Glass-Fused-to-Steel Reservoir (0.18 MG)	ls	1	$175,000.00	$175,000.00
6	Fencing	ls	1	$15,000.00	$15,000.00
7	Telemetry	ls	1	$7,500.00	$7,500.00
8	10-inch piping to reservoir	lf	1900	$65.00	$123,500.00
9	Booster Pump Station, Electrical & Appurtenances	ls	1	$100,000.00	$100,000.00
10	Roadway & site improvements (crushed rock)	ls	1	$10,000.00	$10,000.00
		Construction Total			$551,000.00
		Contingency (20%)			$110,200.00
		Subtotal			$661,200.00
		Engineering (18%)			$119,016.00
		Administrative costs (3%)			$19,836.00
		Total Project Costs			**$800,052.00**

Booster Pump and Piping to Reservoir

In addition to constructing a new reservoir at the City's preselected site, a new booster station and connective piping must be constructed to pump water to the new reservoir to fill it and connect the reservoir to the lower system in order to provide reserves to the entire system.

The booster pump station should be a smaller, packaged-type pump station capable of filling and maintaining the water levels in the upper (Site C) reservoir. Technically, the station should be sized to provide the peak hourly demand flows of the maximum number of customers in the upper level system served by the proposed upper reservoir (Site C). If we assume that the upper reservoir will provide service to around 100 homes, the pump station should be sized to operate at around 200 gpm or 2 gpm per household. This sizing should be confirmed once more accurate information is available on the upper pressure level development plan.

The booster pump station should make use of a skid mounted booster system with multiple pumps mounted on a manifold. The pumps should be sized to deliver the maximum performance required with at least one pump available as a reserve or backup.

The booster system should be connected into the outlet piping on the existing reservoir and into piping that will be used to fill and empty the new reservoir. A check valve should be used to prevent water from flowing backward through the pump station and into the lower reservoir.

As the station will be used only to fill the upper reservoir, variable speed operation is not required. Therefore, soft starts should be used to operate the pumps rather than variable frequency drives. All other controls should be included within a PLC-based control system within the pump station housing.

The new booster equipment should be located within a small building on the site of the new 550,000-gallon reservoir. The building can be wood frame or CMU block construction. The size of the building must include considerations for the pumping equipment, valves and fittings, electrical switchgear and control cabinets, and other system components. The building should include adequate doorway openings to perform regular maintenance on the equipment as required in the future.

The piping connecting the lower reservoir to the upper reservoir should be 10-inch piping providing for fire flows if the hillside between the lower and upper tanks is eventually developed for residential construction.

Costs for the booster pump station are included in the smaller reservoir option above.

Rehabilitation of Existing Reservoir

The existing reservoir was constructed in 1992. Due to problems with construction, the tank has numerous surface leaks, cold joints, and other surface blemishes on the entire surface today (see photo below). While the leaks are not serious in terms of the amount of water that is lost, they do result in a poor aesthetic condition and are likely to be reducing the structural life span of the reservoir. If water is leaking through the 18-inch thick walls, it is likely to be in contact with the reinforcing steel in the reservoir. This condition may result in the eventual corrosion and weakening of the reinforcement steel and the ultimate failure of the reservoir.


Lowell Tank

It is generally considered wise to eliminate leakage through a concrete reservoir in order to extend the useful and structural integrity of the reservoir.

Other communities have found success in rehabilitating leaking reservoirs or repairing reservoirs with cold joints through the use of various injection techniques. A recent HBH project in Rockaway beach was able to take a 25-year old tank with severe cold joint leakage and seal the tank so that practically all seepage was eliminated. The surface was then cleaned and coated to improve the aesthetics of the reservoir. A before and after photo of the Rockaway Beach reservoir is provided below.


Rockaway Beach Tank - Before

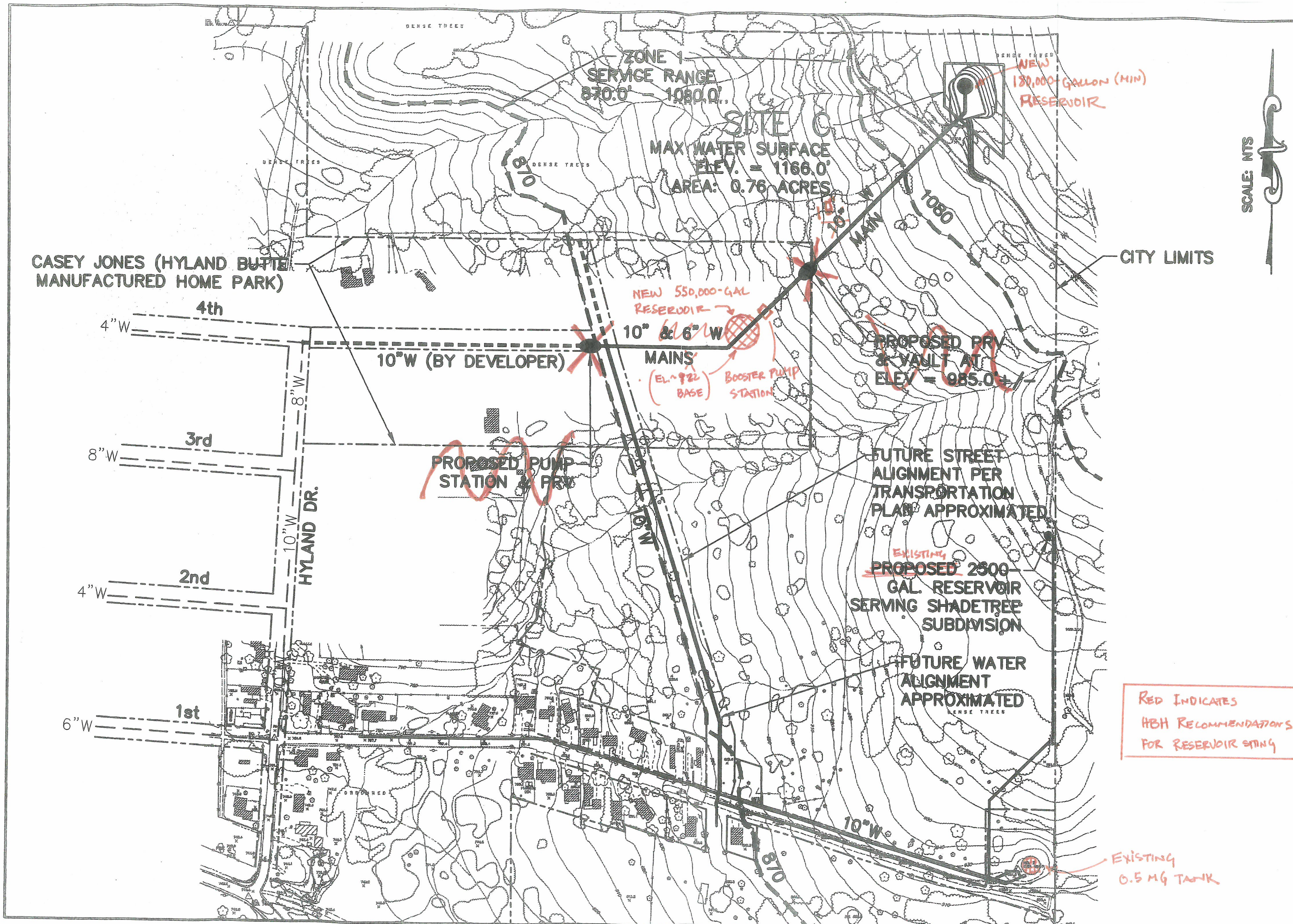

Rockaway Beach Tank – After

If a similar technique is used in Lowell, it is likely that the leakage in the tank can be eliminated, the aesthetics of the tank improved, and ideally, the useful structural life of the tank extended.

The contractor that completed the above work for the City of Rockaway Beach was asked to provide a proposal for the City of Lowell tank repair. The following budget estimate has been prepared based on the contractor's proposal.

Table 7.4-4 – Existing Reservoir Rehabilitation

Existing Reservoir Rehabilitation					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Overhead, Mobilization Costs	ls	1	$5,500.00	$5,500.00
2	Construction Facilities/Temporary Systems	ls	1	$8,000.00	$8,000.00
3	Surface Preparation	ls	1	$5,000.00	$5,000.00
4	Foam Injection	lf	500	$65.00	$32,500.00
5	Epoxy Injection	lf	100	$65.00	$6,500.00
6	Exterior Cleaning & Coating	sf	5440	$8.50	$46,240.00
		Construction Total			$103,740.00
		Contingency (20%)			$20,748.00
		Subtotal			$124,488.00
		Engineering (18%)			$22,407.84
		Administrative costs (3%)			$3,734.64
		Total Project Costs			**$150,630.48**



SYSTEMS WEST ENGINEERS, INC.
mechanical electrical civil
411 HIGH STREET
EUGENE, OREGON 97401-2427

CITY OF LOWELL
PROPOSED WATER STORAGE
SITE C

FIGURE 3.3

PROJECT: 7018.13
MARCH 2001

Section

8

Capital Improvement Plan

8.1 Summary of Recommendations

Several project alternatives were discussed in Section 7 for different parts of the City's water system. This section will summarize the recommendations that will serve as the City's Capital Improvement Plan

The following recommendations are provided for improvements to the City's water system during the planning period:

Water Supply

Based on projected demands, the City will run out of surface water rights before the end of the planning period. It is recommended that the City begin now to acquire new water supplies that will be required near the end of the planning period and beyond.

While some water can be acquired from the City's groundwater supplies, concerns over groundwater quality suggest that development of these resources would not be the best course.

Therefore, it is recommended that the City begin negotiations with the US Army Corps of Engineers to acquire an additional 1.0 cfs (minimum) of surface water rights on Dexter Reservoir. See Section 7.1 for more details on this recommendation. While there is likely to be a cost to complete this water rights acquisition, the Corps of Engineers was unable to provide a quotation at this time.

For the purposes of this plan, it is recommended that the City budget $100,000 for the acquisition and purchase of the additional surface water rights from the COE.

Water Treatment Facilities

The City must expand the existing water treatment capabilities from 160 gpm to 525 gpm to satisfy the planning requirements for this planning period.

To provide for high quality drinking water at the projected flow rate, it is recommended that the City utilize some of the existing treatment facilities and add some new facilities to provide a complete treatment system. In summary, the recommended improvements include:

- Construction of new concrete clearwell to provide for increase contact time. The clearwell will serve as the foundation for a new building to house other new equipment.
- Construct new building over clearwell to house new equipment.
- Install new raw and finished water pumps and VFD's capable of delivering the design flow to and from the treatment plant.
- Install piping and fittings to deliver raw water to the existing clarifier or directly to the coarse screening equipment and membranes.

- Install coarse screening equipment (Amiad or similar) to protect membranes and remove larger debris (50 to 100 micron screening).
- Install new packaged membrane treatment equipment in new building to treat up to 525 gpm. (Or alternatively a packaged conventional process, though the conventional process does not promise the high level of treatment quality and has, potentially, slightly higher operating costs.)
- Install granular activated carbon (GAC) pressure filters after the membranes to provide optional (seasonal) taste and odor removal.
- Install new on-site chlorine generation equipment to provide for disinfection.
- New electrical upgrades to provide for a complete system.
- New piping, fittings, valves, and metering as required for complete installation.
- Upgrades to controls and integration to provide for a complete system.
- All other appurtenances and improvements required to provide for a complete system.

The above recommendations will provide the city with the volume of water required for the planning period and will utilize the best available technology.

An alternative recommendation for the treatment facilities is developed in Section 7 around an interim approach that will increase the output of the plant, but not provide enough water to satisfy the demands of the entire planning period. While the interim approach will more than double the existing plant output, it may only provide water for just over 10 years into the planning period. This study is intended to provide recommendations that meet the 25-year planning period requirements. Therefore, the interim recommendation is not provided as the primary recommendation, though, it could be undertaken now with additional upgrades to be undertaken in the latter half of the planning period.

Treated Water Storage

It was found that the City is currently deficient for storage of treated water for the purposes of providing drinking water under maximum demand conditions in conjunction with fighting a major fire (see Section 7.4). Therefore, it is recommended that the City construct new treated water reserves as further described below:

It is recommended that the City first construct a 550,000-gallon reservoir in the lower pressure level. The preferred location for this reservoir would be on the northwest side of town on the Seneca Lumber property. The tank should be located at the same elevation as the existing reservoir.

An alternative location, referred to as “Site C” in a previously completed siting study, is equally as appropriate for the location of the new reservoir. The site C alternative would locate the new 550,000 gallon reservoir on the alignment (but within the lower pressure level) of the piping previously identified to service Site C as developed in the previous siting study (See Figure 3.3 from the siting study on page 7-35 of this plan).

The new reservoir should be a glass-fused-to-steel (GFS) type reservoir and enclosed on a secure site with chain-link fencing and other appurtenances as needed for a complete system. This reservoir should be constructed as soon as the City is able to secure adequate funding to do the project

A second reservoir should be constructed on or near Site C in the upper pressure level as identified in the City's previous reservoir siting plan. This second reservoir should have a volume of at least 180,000 gallons and should be a GFS type reservoir. The second reservoir should be constructed to service development in the upper pressure level. The timing of this reservoir will depend upon the development plan for the upper pressure level.

Another smaller reservoir may be required to service the upper elevations of the Seneca property at some point in the future. However, as the plan for development of the property is currently not known, no recommendations are provided at this time for this second upper level tank.

It is further recommended that the City undertake a project to rehabilitate the existing 500,000 gallon reservoir early in the planning period.

8.2 Project Cost Summary (CIP)

This section will summarize the project costs for the recommended projects summarized above. Taken together, these projects will form the City's CIP for the water system.

Table 8.2.-1 below summarizes the CIP for the City of Lowell water system for the planning period from 2006 to 2031. Project costs are represented in 2006 dollars and should be adjusted in the future as discussed in Section 6 of this plan.

Table 8.2-1 – City of Lowell CIP and Project Cost Summary

Project No.	Project Name and Description	Project Cost
1	Acquisition of 1.0 cfs Surface Water Rights	$100,000
2	Water Treatment Facilities Upgrades (based on membrane alternative)	$1,947,785
3A	New 550,000-gallon Reservoir Project	$1,039,632
3B	New 180,000-gallon Reservoir & Pump Station Project	$800,052
3C	Rehabilitation of Existing Reservoir	$150,630
	Total Project Costs	**$4,038,100**

It should be reiterated that an interim project could be substituted for the recommended water treatment system upgrades. However, the interim project will only provide adequate water service for a period in excess of 10 years. At which time, a major upgrade project would be required. At an estimated cost of $462,000, the interim cost could reduce the overall project cost to around $2,552,315.

8.3 Project Implementation

This section will provide a brief description of project timing (project schedule) and issues affecting the implementation of the project as well as the potential to phase the projects.

Project Schedule

The city is facing several immediate existing system deficiencies in their water system including issues with the treatment facilities and the finished water storage supplies. For these reasons, much of the recommended projects should be undertaken as soon as the City has adequate funding available to complete the work.

If the City immediately begins the process of securing funding, it is likely that design activities could begin some time in the 2007 year with construction taking place between 2008 and 2010. Because many of the deficiencies described in this plan are immediate, the City should proceed expeditiously toward solutions. As soon as funding is available for design, the City should begin the final design process for the projects that are to be undertaken.

Project Phasing

While many of the projects require immediate or relatively swift attention, there are opportunities for the city to phase-in some of the improvements. A brief summary is provided below on the timing and potential phasing for each of the projects included on the CIP list in this section:

- **Project 1- Surface Water Rights:** While the city will need to acquire additional water rights, the need for this is not imminent. However, the concern always exists as to whether water rights will be available in the future when they truly are needed. Therefore, the city should begin the process of investigating the availability of additional water from Dexter Reservoir. The City should determine what water is available and what will be the cost of acquiring new water rights. Once this is determined, it will be easier to place a priority ranking on this project as well as determining when the City should spend the funds to secure the water rights.
- **Project 2 – Water Treatment Upgrades:** Due to deficiencies with the existing treatment process, the City should undertake the treatment upgrades as soon as funding is available. The only feasible alternative available to phase the treatment upgrades is to undertake the interim upgrades (Alternative No.3 in Section 7.3) which will provide the City with water service for a period in excess of 10 years. If the interim project is undertaken early in the planning period, another, more significant project, must be undertaken later in the planning period.
- **Project 3A – 550,000 gal Reservoir:** As the City is currently deficient for potable water storage, this project should be undertaken as soon as funding is available. It is not feasible to phase this project and the cost of constructing multiple reservoirs over time is greater than constructing a single reservoir now.
- **Project 3B – 180,000 gal Reservoir and Pump Station:** The need for this project will depend greatly on the development pressures in the upper pressure level. As the plans for the upper pressure level develop, the City should require that the reservoir be constructed by the developers of the upper subdivision. SDC credits can be provided to cover some of the developer's costs or SDC revenues can be used to build the reservoir should the City be able to and choose to undertake the project. Therefore, this project can be phased and constructed on an as-needed basis later in the planning period as required.

- **Project 3C – Rehabilitation of Existing Reservoir:** This project can be undertaken at a later date and as funds are available. However, the life of the reservoir can be extended if the rehabilitation efforts are completed in time to protect the structural steel.

Project Prioritization

While all of the recommended projects are important, they should not be considered as having equal priority ratings. For the purposes of this plan, the following priority ratings system is provided:

1. **Priority 1** – Priority 1 projects should be undertaken immediately and as soon as the City has available funding. Priority 1 projects will correct existing deficiencies and provide capacity for the planning period. In this plan, projects 2 and 3A should be classified as Priority 1 projects.
2. **Priority 2** – Priority 2 projects should be undertaken when funding becomes available, but are not necessarily considered critical to address existing deficiencies. Priority 2 projects include important maintenance projects. In this plan, projects 1 and 3C should be considered as Priority 2 projects.
3. **Priority 3** – Priority 3 projects should be undertaken based on need and development pressures. These projects should be considered optional until development pressures require the project to be undertaken. In this plan, project 3B should be considered as a Priority 3 project.

Section 9

Financing Strategy

9.1 Improvement Costs

The estimated total cost for the improvements recommended in the water system master plan are:

- Priority 1 – $2,987,417
- Priority 2 – $250,630
- Priority 3 – $800,052

The total project costs for all project priorities is $4,038,100.

The proposed improvements to the supply, treatment, and storage elements of the Lowell water system will increase capacity from 160 gpm currently up to at least 525 gpm. This increase in capacity will correct existing deficiencies as well as provide for future growth in the system.

This section shall seek to provide some insight with regard to potential funding mechanisms for the city to consider paying for the improvements. This section will also make an attempt to identify the potential impact to rate payers.

The following sections will provide a brief summary of some potential sources of funding available for the recommended projects.

9.2 System Development Charges (SDCs)

While a complete SDC methodology was outside of the scope of work for this plan, an effort has been made to characterize the potential SDC charges that could be established based on the recommendations and planning information contained in the plan.

SDCs are utilized to collect funds from development and growth resources in order to offset the cost of developing infrastructure that is capable of supporting growth and development. Generally, SDCs are divided into two main categories:

1. **Reimbursement SDC:** A reimbursement SDC is used when a community builds a new facility that has excess capacity incorporated for future growth. The city "over sizes" the facility and reserves the extra capacity until such time as growth or development pressures require the use of the reserved capacity. At that time, the new development is charged a reimbursement SDC to pay for the cost of the reserved capacity.

2. **Improvement SDC:** An improvement SDC is collected for facilities that will be constructed in the future to accommodate growth. For example, a community may know that they will have to build a new treatment facility in 10 years to accommodate growth occurring over the planning period. In such a case, the city can assess an improvement SDC against growth today and save the funds to be used for the construction of the improvement in the future. If it were not for the growth being experienced today, the future facility would not be required. Therefore, in these

cases, development pays for a portion of the future facility through an SDC charge before it is constructed.

Typically, community SDC programs utilize a combination of reimbursement and improvement SDCs when calculating a total SDC charge.

In addition to the project-related SDC charges, a community is allowed, under State statutes, to include charges which are to be used to implement an SDC program. These charges, typically referred to as compliance costs, can be used to pay for the preparation of SDC methodologies, master planning, accounting costs, SDC administration costs, advertising costs, and other costs which are incurred when implementing an SDC program.

For this plan, an effort was made to identify the percentage of each project's SDC eligibility. This was accomplished by determining what the existing capacity needs were for each project and determining what percentage of each project was needed purely for future development or growth. Accordingly, the total SDC eligible costs were divided by the number of new EDUs projected to be added to the system over the planning period (see Section 5).

A summary of the preliminary SDC calculation is provided below in Table 9.2-1.

Table 9.2-1 – SDC Calculation Worksheet

Project No.	Project Name and Description	Project Cost	Percent SDC Eligible	SDC Eligible Costs
1	Acquisition of 1.0 cfs Surface Water Rights	$100,000	93%	$93,000
2	Water Treatment Facilities Upgrades (based on membrane treatment alternative)	$1,947,785	55%	$1,071,282
3A	New 550,000-gallon Reservoir Project	$1,039,632	55%	$571,798
3B	New 180,000-gallon Reservoir & Pump Station Project	$800,052	100%	$800,052
3C	Rehabilitation of Existing Reservoir	$150,630	0%	$0
	Total Project Costs	**$4,038,100**		**$2,536,132**
		Existing EDUs		377
		Future EDUs		848
		EDUs added over planning period		471
		Estimated SDC Charge per EDU		**$5,384.57**

Based on the above analysis, the city could charge an SDC of around $5,384.57 for their water system SDC. It is important to note that this SDC amount does not include accommodations for any SDC credits nor does it include charges for SDC compliance costs. In order to determine a final SDC amount, an SDC methodology update should be completed.

9.3 Loan and Grant Sources

The current State average non-metropolitan median household income is $41,230. According to the U.S. Census report in 2000, Lowell has a median household income (MHI) of $35,540 which is 86.2% of the State average. The percentage of low-to-moderate income persons residing in Lowell is around 47%. The current average residential water bill is $33.86 for a usage of 5,188 gallons (per EDU), and $44.50 for a usage of 7,500 gallons.

The following programs could be considered as potential funding sources for the City:

RUS/USDA Loan and Grant Program. The Rural Utilities Service (RDA) offers grant and loan packages to municipalities to fund infrastructure improvement projects. The RD packages typically include a combination of grants and loans with the award amounts varying based on financial need, total project costs, financial stability of the entity, and other factors. Interest rates are typically lower than market rates with grant amounts varying based on the MHI of the community.

Technical Assistance (TA) Grants. TA grants are available through the USDA/RUS program to be used for technical assistance projects, planning efforts, operator training, and other technical assistance purposes. The grants are typically limited to $10,000 to $20,000.

Community Development Block Grant. CDBG are available through the Oregon Economic and Community Development Department (OECDD). Funds come from the USDA and are intended to be used to develop water and wastewater infrastructure. Grants are available for up to $1-million dollars. Communities must satisfy several selection criteria for this competitive grant program including local financial criteria such as the percentage of low to moderate income households. According to OECDD, the percentage of low to moderate households in Lowell is approximately 47%. To qualify for the grant program, a city must have more than 50% low to moderate income households.

Based upon this information, Lowell will not likely qualify at present for a Block Grant from the Oregon Economic and Community Development Department since the low-to-moderate income requirements are not met.

Water/Wastewater Financing Program. This state funding program is administered through OECDD and is intended to assist communities with the development of water and wastewater infrastructure. The program typically includes a package of loan and grant monies to fund some or all of a project. Grants are only typically made available if loans are not financially feasible for a community. As with other programs, several criteria must be satisfied.

Drinking Water Program State Revolving Loan Fund (SDWRLF). It is also possible that a loan could be obtained through the Safe Drinking Water State Revolving Loan Fund. Typically, an existing compliance problem or health risk is required to qualify. Typically, if a community qualifies for funding, the low interest rate loans are an excellent source of funding for all or some of a project's costs. Special consideration is given for small communities (10,000 persons or less) as well as disadvantaged communities. The SRLF is administered through OECDD.

9.4 Other Funding Sources

In addition to grants and loans, local funding resources can be utilized. This can include general obligation bonds, revenue bonds, improvement bonds, capital construction or "sinking" funds, local improvement districts, ad valorem taxes, assessments and other local funding sources.

All funding options should be explored and considered by the city in preparation for undertaking major system improvement projects.

9.5 Potential Impact to Rate Payers

In Lowell, the typical household consumes an average of 5,188 gallons of water per month, which results in an average residential water bill is $33.86 per month (for 5,188 gallons, or 1 EDU). Funding agencies often cite a figure of 7,500 gallons per month to estimate the average water bill. For 7,500 gallons, the existing rate is $44.50. The rate structure consists of a base charge of $16.50 per month, plus $3.30 per 1,000 gallons for the first 5,000 gallons, plus $4.60 per 1,000 gallons for usages over 5,000 gallons.

For the calculations indicated below, it is assumed that the entire cost for each improvement project will be funded with a single 25-year loan at 4% interest so as to separately reveal the rate hike associated with each project. However, the rate hike is based upon the current system EDU-value (377 EDUs) in order to meet the payment schedule during the early stages of amortization.

If the Priority 1 improvements are adopted (project cost: $2,987,417), then an immediate increase in water system revenue of $15,769 per month is required, or about $41.83 per EDU.

If the Priority 2 improvements are adopted (project cost: $250,630), then an immediate increase in water system revenue of $1,323 per month is required, or about $3.51 per EDU.

If the Priority 3 improvements are adopted (project cost: $800,052), then an immediate increase in water system revenue of $4,223 per month is required, or about $11.20 per EDU.

While it is not the primary recommendation of this plan, should the City elect to undertake the interim improvements for an estimated cost of $462,026.40, an immediate increase of $2,438.75 per month, or about $6.47 per EDU. However, the interim approach is only an attempt to increase treatment capacity and does not satisfy the projections for the planning period nor does it attempt to correct the existing and projected treated water storage deficiencies.

9.6 Recommended Financing Plan

Based on the recommendations and information provided in this plan, this section is intended to provide a general financing plan for the city to follow to pursue and obtain the necessary funding to undertake the selected improvement project(s).

The recommended financing plan is as follows:

1. Immediately update the city’s SDC methodology and assessment to reflect the new CIP. Begin collecting SDC funds that can be contributed to the project.

2. Schedule a one-stop meeting where all potential and participating funding agencies can attend to discuss and potentially offer funding packages for the city’s improvement projects.

3. Begin the process of raising user rates in anticipation of a new loan. It is not necessary to make a very large increase, though the city should consider the likelihood of the funding package they will receive and develop a schedule of rate increases that can be implemented over a couple of years.

4. When the project costs and funding package become clear, the city should raise rates as required

to meet their obligations for the improvements.