Water System Master Plan

City of Warrenton, Oregon

CH2MHILL

July 1997

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Water System Master Plan

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JULY 1997

REGISTERED PROFESSIONAL ENGINEER
#13,248
OREGON
JULY 17, 1985
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Expires 12/31/97

The preparation of this report was funded in part with an award from the Oregon Economic Development Department

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CHAPTER 1

Executive Summary

Warrenton's *Water System Master Plan* will guide improvements to the water system over the next 20 years. It outlines a program to ensure that customers continue to receive a reliable supply of safe drinking water at an affordable price.

The city's last water master plan was prepared in 1979. Since that time, the city has experienced about 25 percent growth in water use. In addition to growth, other factors are directing the course of improvements for the water system. These include strengthening of drinking water and environmental regulations, and a greater awareness of the value of conservation.

Existing Facilities

The city's water system serves about 8,000 people in Warrenton, Gearhart and surrounding unincorporated areas. The system annually provides an average of 2,400,000 gallons per day, with use exceeding 5 million gallons per day during peak summer periods.

Warrenton's water supply comes from the Lewis and Clark River and three of its tributaries. The four intakes are located in the coast range northeast of Seaside. The supply system includes a 17 million gallon raw water impoundment, which is used to supplement the streams during low flow periods or as a substitute to the streams during storm events that cause high turbidities. The city holds water rights far in excess of projected needs. However, the reliable capacity of the system is limited by the drought yield of the streams.

Treatment consists of chlorine feed for disinfection and lime feed for reducing the corrosivity of the water.

Water is supplied by gravity to the entire service area. The 12 miles of transmission pipelines from the watershed include sections of 24, 20 and 18-inch pipe. The transmission capacity is adequate to meet system needs through the 20-year planning period.

The distribution pipe network includes over 20 miles of 8- through 18-inch diameter pipe. Most of this is concentrated in the Warrenton and Hammond areas. Connections off the transmission lines feed Gearhart and unincorporated areas in the Clatsop Plains. Distribution storage is provided by two tanks, a 1.6 million gallon steel tank located east of Camp Rilea, and a 200,000 gallon tank in the center of Warrenton.

Recommended Improvements

Chapter 8 presents a capital improvements plan for Warrenton's water system. The plan is summarized in Table 8-1. The attached maps, which follow Chapter 8, also provide a summary of the improvements. Table 8-2 lists non-capital recommendations.

Meters

The city has already initiated one of the primary recommended improvements, which is the installation of meters for all customer connections. This is a necessary prerequisite to account for all water use and to obtain outside funding for proposed improvements. Metering also instills an awareness of the cost of water and may result in lower per capita use.

Conservation

As an adjunct to system-wide metering, the master plan recommends implementing conservation measures. Depending on the water use patterns following the introduction of meter-based rates, the city may wish to target reductions in unaccounted for water and per capita use. By promoting conservation measures, the city may realize significant savings for its customers.

Water Treatment Facility

The largest single recommended project in terms of cost is the construction of a filtration plant. The city has used chlorine to disinfect the water supply for many years. This level of treatment is no longer considered sufficient to control microorganisms such as *Giardia* and *Cryptosporidium*. In the past few years, the city has investigated filtration alternatives and decided on a technology called slow sand filtration. Slow sand filtration provides a high level of reliability and appears to be the least expensive option.

Water Supply

Available records and other studies suggest that the average monthly drought yield of the city's present source waters is 3.7 million gallons per day. Present levels of demand will result in periodic water shortages. As growth occurs, there is potential for extended periods of water shortages. Two options are discussed in the master plan for expanding the city's supply. One is to add additional raw water storage in the watershed. The second is to add a supplemental groundwater supply in the Clatsop Plains area. Because of water quality, treatment costs and potential environmental impacts for the groundwater supply, the city favors the option of adding raw water storage in the watershed. A proposed size for a second impoundment is presented in the capital improvements plan, although it is based on very limited data on stream flows.

We recommend further investigation of supply improvements. In particular, additional groundwater quality data will better enable the city to compare costs between groundwater and adding a surface impoundment. Moreover, a second, similar sized impoundment will not meet long-term supply needs. A part of the recommended groundwater testing includes investigation into aquifer storage and recovery. This involves storing treated water in the Clatsop Plains aquifer in the winter, and withdrawing the same water in the summer. If the aquifer is appropriate for this technology, it may eliminate the need for costly iron and manganese removal treatment of the groundwater supply and make groundwater an attractive alternative for Warrenton.

Distribution Storage and Pumping

Another large project is the construction of a new finished water storage tank near the city. This tank, called the North Coast Reservoir, is needed to supply peak daily water use, provide improved fire protection, and to increase emergency storage. A 3.0 million gallon volume is proposed and the recommended elevation is nearly 50 feet higher than the existing tank. This will raise pressures and flows throughout Warrenton and Hammond. A booster pump station and new pipelines are elements of this improvement.

Distribution Piping

Additional pipelines are needed to boost pressures in the Hammond and Fort Stevens area north of downtown. Modeling of the city's distribution system demonstrated the existing pipelines are not capable of meeting peak summer demands and fire flows in this area. Pipeline improvements are also identified for other areas of the system, including the North Coast area and in Surf Pines.

Major Recommended Improvements:
Meter all customers
Install a slow sand filtration plant
Investigate supply expansions (surface and groundwater)
Install new storage tank and associated pipelines and pump station
Install pipelines to increase supply to north part of system

Costs for Improvements

Recommended projects and their costs are summarized in Table 1-1. The treatment plant, North Coast Reservoir and its pump station and some other projects are categorized as current needs. These improvements are necessary to bring the system into compliance with current regulations and recommended design criteria. The second category is projects to meet future needs. These are needed to accommodate projected growth. The capital improvements plan in Chapter 8 assigns dates for their implementation based on the water requirement projections of Chapter 2.

The estimated cost for currently needed improvements totals $11,260,000. The two main components are installation of a filtration plant and a new finished water storage tank and pump station. Distribution pipe upgrades also account for a large share of the capital improvements plan.

Improvements listed as future needs total $4,370,000. These include additional pipeline additions, a second new reservoir, and supply improvements.

Cost estimates are conceptual-level only. Although they include allowances for contingencies and engineering, unforeseen changes could result in actual costs that are higher. All estimates are given in June 1997 dollars.

TABLE 1-1
Water System Capital Improvements Plan Summary

Project Description	Need for Project	Estimated Cost
CURRENT NEEDS		
Install meters for all residential customers	Meters will allow for full accounting of all water that is used and encourage careful water use	$156,000
Construct a slow sand filtration plant	Filtration needed to achieve compliance with state and federal drinking water regulations	$4,900,000
Install a new 3.0 million gallon finished water tank plus pump station and connecting pipes	Additional storage will provide more reliable system operation for peak use periods and fire protection, and will improve pressure levels in north system	$2,820,000
Install cover on existing Clatsop Plains reservoir	A cover will protect water quality against airborne contamination and vandalism	$200,000
Perform pipe leak survey and repair leaks	Leak surveys and repairs are usually a cost-effective management approach	$50,000
Upgrade distribution system in Surf Pines area	To improve fire flows and develop looped system	$880,000
Upgrade distribution system by adding pipelines from new reservoir to north end	More carrying capacity is needed to improve fire service to Hammond and other areas in the north of Warrenton	$2,250,000
	SUBTOTAL	$11,260,000
GROWTH NEEDS		
Add 17 million gallon raw water impoundment in watershed	Current supply will not be capable of meeting system needs during drought conditions in a few years	$600,000
Perform water quality monitoring to investigate a groundwater supply	It may be necessary to supplement surface water supplies with groundwater	$15,000
Upgrade distribution system by adding pipelines	New pipelines are needed to improve peak use pressures and fire flows, particularly as growth occurs	$1,500,000
Install a second new 3.0 million gallon tank	More storage will be needed as growth occurs	$2,250,000
	SUBTOTAL	$4,370,000
	TOTAL	**$15,600,000**

CHAPTER 2

Water Requirements

This chapter provides estimates of future water requirements for Warrenton. It includes descriptions of historical water use data and of criteria for projecting future demands. This chapter also includes discussions on unaccounted-for water, conservation, and fire flow requirements.

Water use projections are central for planning improvements for the city's water supply, storage, and transmission facilities.

Chapter Summary

In 1996, an average of 2.4 million gallons per day of water were delivered to the water system through the source master meter. The yearly total production was 876 million gallons.

On the highest use day in the summer, the system supplied 5.3 million gallons. This value, called the maximum day demand, is important from a planning standpoint. The source water facilities, treatment system and transmission piping must be sized to meet the maximum day demand. If they are not, storage tanks within the system will fail to fill during the night. Successive days of at or near the maximum day demand value will result in a water shortage.

1996 water use
Annual average: 2.4 mgd
Maximum single day: 5.3 mgd

Projected use for year 2016
Annual average: 3.5 mgd
Maximum single day: 7.5 mgd

Residential users inside the city limits are not metered. In addition to the higher water use experienced with unmetered customers, this results in a record-keeping problem; it is not possible to accurately determine the amount of unaccounted for water. Unaccounted for water includes water lost through leaks and unmetered public uses such as hydrant use.

For Warrenton, the difference between production and metered consumption in 1994 was 49 percent. City staff have estimated that the average unmetered residential customer uses 10,000 gallons per month. Using this value, the amount of unaccounted for water is 34 percent. The city's actual rate cannot be determined until all customers are metered. Most utilities strive to keep this value within a maximum range of 10 to 15 percent.

Historical demands were put into per capita terms, and projected for the 20-year study period using population projections provided by the city's planner. The planner estimated growth within Warrenton and Hammond to occur at a rate of 2.95 percent per year, and growth outside the city to occur at 1.0 percent per year. If growth occurs at these rates and per capita water use remains at today's levels, the average use will grow from 2.4 to 3.5 million gallons per day in year 2016. Maximum use will grow from 5.3 to 7.5 million gallons per day in year 2016. These projections represent 50 percent increases over current use.

Since maximum day demands dictate the sizing of source and treatment facilities, we encourage the city to implement conservation measures. These measures should focus on reducing peak summer use in combination with reducing the level of unaccounted for water. The first step is metering all customers. Other conservation measures include leak detection and repair, public education programs, rate structures that encourage efficient water use, and plumbing code revisions to require low water use fixtures.

Chapter Recommendations:
Meter all customers
Track production and consumption
Determine unaccounted-for water rate
Carry out leak detection and repair program
Implement conservation measures

Definition of Terms

Demand is defined as the total quantity of water supplied (produced) for a given period of time to meet all system needs. Demand in this sense includes all consumption (residential, commercial, and industrial) plus public uses (e.g., fire fighting or hydrant flushing) and water lost to leakage or evaporation.

Demand Terms
Average Day Demand (ADD): total volume of water delivered in a year divided by 365
Maximum Day Demand (MDD): maximum water delivered in any single day of a calendar year
Peak Hour Demand (PHD): highest hourly use hour during the MDD

The most common units for expressing these demands are million gallons per day (mgd). One mgd is equivalent to 695 gallons per minute (gpm) or 1.55 cubic feet per second (cfs).

Peaking factors describe the ratio of maximum day to average day use, or peak hour to maximum day use.

Unaccounted-for water refers to the difference between metered production and metered consumption. Unaccounted for water includes unmetered residential use, unmetered hydrant use, and water lost to leakage or evaporation. Meter inaccuracies also contribute to unaccounted for water.

Historical Water Use Information

The historical water use data and population growth projections form the basis for projecting future water demands. Historical demand, population, and service connection data were provided by city engineering and planning staff. The historical demands for 1994 are summarized in Table 2-1. They were divided into sub-categories using meter book boundaries and water billing information.

Appendix A contains the detailed 1994 water use records provided by the city. Appendix B provides population planning information that was provided by the City Planner.

TABLE 2-1
1994 Warrenton Water Demands[1]

Area	ADD (mgd)	MDD (mgd)	MDD/ADD Peaking
Inside Warrenton and Hammond, without seafood processing	0.98	1.71	1.7
Seafood processing (inside Warrenton and Hammond)[2]	0.51	2.00	3.9
Outside Warrenton and Hammond	0.82	1.43	1.7
Total	**2.31**	**5.14**	**2.2**

[1]Demands estimated from 1994 meter book records.
[2]Seafood processing MDD based on 1995-96 water billing information, and equals 1.2 times peak season demand.

The total seafood processing water demands inside the city limits of Warrenton and Hammond include the four largest seafood processing industries. Table 2-2 presents the 1995-1996 water demands of the seafood processing industries.

TABLE 2-2
Seafood Processing Demands[1]
(In million gallons per day)

Company	Annual Average Use	Peak Season Use[2]
Pacific Coast Seafoods	0.266	0.661
Point Adams Packing	0.156	0.688
Protein Recovery Company	0.057	0.278
Bio-Products Company	0.033	0.047
Total Seafood Processing	0.512	1.674

[1] Estimated from 1995- 1996 billings.
[2] Peak season occurs in the summer months.

Discussion

For 1994, the system ADD was 2.31 mgd, and the MDD was 5.14 mgd. The maximum to average demand ratio was 2.2.

Industrial demands for seafood processing were separated from other system demands because they are a large portion of the total system demand. They represented about 22 percent of the ADD and 39 percent of the MDD for 1994.

Several peaking factors are listed in Table 2-3. They are useful primarily for performing the hydraulic modeling of the system.

TABLE 2-3
Peaking Factors

Factor	Value
Maximum to average day (without seafood processing)	1.7
Seafood industry maximum to average day	3.9
Peak hour to maximum day (without seafood industry)	1.5
Seafood industry peak hour to maximum day	1.2

Only limited data were available to estimate PHDs. Data from 1992 were analyzed for the April 1994 *Preliminary Filtration Evaluation Study*, and were presented in Figures A-1 through A-3 in that report. The peak hour to maximum day ratio averaged 1.5. This corresponds to information presented in *Distribution Network Analysis for Water Utilities Manual* (AWWA M32, 1989), which suggests that typical PHD/MDD peaking factors range from 1.3 to 2.0.

Unaccounted-for Water

Unaccounted-for water in 1994 was 49 percent of total production, based on billing and production records. However, this high rate is caused in part by the unmetered residential services within Warrenton's city limits. There were approximately 1,130 unmetered residential services in 1994. City staff have estimated that monthly use averages 10,000 gallons for these residential, unmetered services. When metered totals are corrected by this amount, the unaccounted for water rate is 34 percent.

A second major contributing factor is the continuous overflow practiced at the Clatsop Plains Reservoir. As described in Chapter 6, an average of approximately 150 gpm overflows the reservoir to protect against water quality degradation. This results in a loss of 216,000 gallons per day, nearly 10 percent of current ADD. The improvements presented in the capital improvements plan (the addition of new reservoir storage at a higher elevation and a booster pump station) will eliminate the need for this overflow.

A unaccounted for water rate of 34 percent is excessive. A rate of 10-15 percent is a reasonable target for Warrenton's system. We recommend the following steps to respond to the apparent high unaccounted for water rate:

1. Install meters for all customers. It is not possible to confirm the unaccounted for water rate without metering of all customers. Meters will encourage responsible use, help the city to track growth and leakage, and are a necessary prerequisite for outside funding of system improvements. The installation of meters is also a prerequisite if the city applies for additional water rights to increase supply.
2. Once meters are installed, carefully track unaccounted-for water to determine areas of high leakage and other losses. Leak detection and repair could commence prior to the installation of meters, but would be carried out more efficiently once meters are installed.
3. Install the new reservoir and booster pump station included in the capital improvements plan to eliminate the need for overflow at the existing Clatsop Plains Reservoir.
4. Contract with a firm that specializes in leak detection to assist the city in locating problem areas of the system. Include a budget item in the capital improvements plan for this leak detection program and for repair of those pipes contributing to the problem. Depending on the outcome of the leak survey and the level of unaccounted-for water, the city may wish to add an annual budget amount for pipe replacement.
5. Continue and improve the city's program for meter calibration. Check and calibrate annually all meters 2-inches and larger. Annually calibrate at least 5 percent of all meters.

Water Demand Projections

Projections for future water demands were developed by applying population growth projections to historical per capita demands. Table 2-4 summarizes population projections.

A lower growth rate, 1.0 percent, was assumed for the seafood processing industry. Industrial demands depend on the type and size of future industries. A 1.0 percent annual growth provides a modest allowance for future seafood processing industrial growth. Annual growth should be monitored to verify projections and make necessary adjustments to the demand projections.

TABLE 2-4
Population Growth Projections

Area	Projected Annual Growth Rate
Inside Warrenton and Hammond[1]	2.95%
Outside Warrenton and Hammond[1]	1.0%
Seafood processing industry	1.0%

[1]Provided by City Planner

TABLE 2-5

Demand Projections

Not including reductions for conservation measures

Year	Average Day Demand (mgd)				Maximum Day Demand (mgd)				Peak Hour Demands (gpm)			
	Inside[1]	Seafood[2]	Outside[3]	Total	Inside	Seafood[2]	Outside	Total	Inside	Seafood[2]	Outside	Total
1994	1.0	0.5	0.8	2.3	1.7	2.0	1.4	5.1	1,780	1,670	1,490	4,940
1995	1.0	0.5	0.8	2.4	1.8	2.0	1.4	5.2	1,830	1,690	1,500	5,020
1996	1.0	0.5	0.8	2.4	1.8	2.0	1.5	5.3	1,890	1,700	1,520	5,110
1997	1.1	0.5	0.8	2.4	1.9	2.1	1.5	5.4	1,940	1,720	1,530	5,190
1998	1.1	0.5	0.9	2.5	1.9	2.1	1.5	5.5	2,000	1,740	1,550	5,290
1999	1.1	0.5	0.9	2.5	2.0	2.1	1.5	5.6	2,060	1,760	1,570	5,390
2000	1.2	0.5	0.9	2.6	2.0	2.1	1.5	5.7	2,120	1,770	1,580	5,470
2001	1.2	0.5	0.9	2.6	2.1	2.1	1.5	5.8	2,180	1,790	1,600	5,570
2002	1.2	0.6	0.9	2.7	2.2	2.2	1.5	5.9	2,250	1,810	1,610	5,670
2003	1.3	0.6	0.9	2.7	2.2	2.2	1.6	6.0	2,310	1,830	1,630	5,770
2004	1.3	0.6	0.9	2.8	2.3	2.2	1.6	6.1	2,380	1,840	1,650	5,870
2005	1.3	0.6	0.9	2.8	2.4	2.2	1.6	6.2	2,450	1,860	1,660	5,970
2006	1.4	0.6	0.9	2.9	2.4	2.3	1.6	6.3	2,520	1,880	1,680	6,080
2007	1.4	0.6	0.9	2.9	2.5	2.3	1.6	6.4	2,600	1,900	1,700	6,200
2008	1.5	0.6	0.9	3.0	2.6	2.3	1.6	6.5	2,670	1,920	1,710	6,300
2009	1.5	0.6	1.0	3.1	2.6	2.3	1.7	6.6	2,750	1,940	1,730	6,420
2010	1.6	0.6	1.0	3.1	2.7	2.4	1.7	6.7	2,830	1,960	1,750	6,540
2011	1.6	0.6	1.0	3.2	2.8	2.4	1.7	6.9	2,920	1,980	1,760	6,660
2012	1.7	0.6	1.0	3.2	2.9	2.4	1.7	7.0	3,000	2,000	1,780	6,780
2013	1.7	0.6	1.0	3.3	3.0	2.4	1.7	7.1	3,090	2,020	1,800	6,910
2014	1.8	0.6	1.0	3.4	3.1	2.4	1.7	7.2	3,180	2,040	1,820	7,040
2015	1.8	0.6	1.0	3.4	3.1	2.5	1.8	7.4	3,280	2,060	1,840	7,180
2016	1.9	0.6	1.0	3.5	3.2	2.5	1.8	7.5	3,380	2,080	1,850	7,310

[1] Inside city limits of Warrenton and Hammond

[2] Seafood processing inside city limits of Warrenton and Hammond

[3] Outside city limits of Warrenton and Hammond

Figure 2-1
Demand Projections

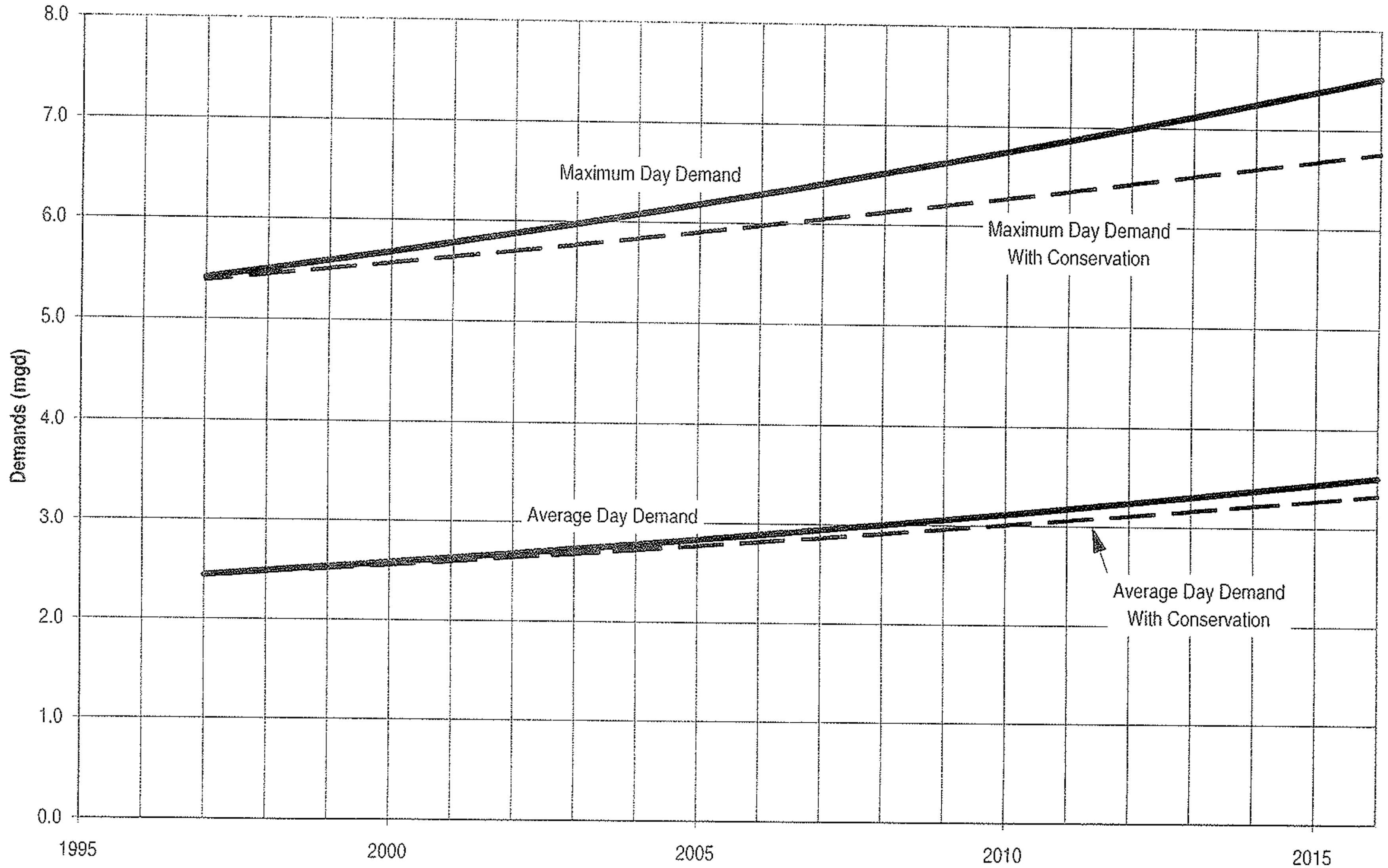


Table 2-5 and Figure 2-1 show demand projections for the next 20 years, based on these growth rates. Average day use is expected to grow from a current level of 2.4 mgd to 3.5 mgd in year 2016. Maximum day use is expected to grow from 5.3 mgd to 7.5 mgd. These projections represent 50 percent increases over current use. A maximum day demand of 7.5 mgd will exceed the city's supply capacity of 6.0 mgd.

These projections assume that per capita use and the unaccounted for water rate remain at current levels. The addition of meters will reduce per capita use, although the rate of reduction is uncertain. Combined with leak repair and conservation measures (discussed later in this chapter), a maximum day demand reduction of 10 percent by the end of the twenty year period is a realistic target for planning purposes. This reduction will not be achieved all at once, but will gradually phase in as conservation measures are implemented. Once meters are installed and the city records two or three years of demand data with meters in use, the true unaccounted-for water rate can be determined. The city can then decide what level of per capita MDD reduction to target. The target level should bring the unaccounted-for water rate down to 15 percent or less.

Table 2-6 summarizes maximum day demand projections, with and without reductions from conservation. Conservation is phased in, so that the target level of 10 percent is achieved at the end of the 20-year planning period. As shown in Figure 2-1, conservation could conceivably delay the time when the source capacity of 6.0 mgd is exhausted from year 2003 to year 2006.

A reduction in per capita MDD will have a corresponding reduction in ADD, although the percent of reduction in ADD will be less.

TABLE 2-6
Maximum Day Demand Projections

Year	Using Current Per Capita Levels	With a Phased 10% Reduction in Per Capita Use
2001	5.8	5.6
2006	6.3	6.0
2011	6.9	6.4
2016	7.5	6.8

Fire Flow Requirements

Fire flow demand is the amount of water required to fight a fire for a specified period. The Insurance Services Office (ISO) classifies cities for insurance rating purposes on the basis of a maximum fire flow requirement of 3,500 gpm. Fire flow requirements that are larger than 3,500 gpm are evaluated individually and not used by the ISO in determining the public protection classification of a city water system.

The recommended required fire flow durations, according to the Fire Protection Handbook (National Fire Protection Association, 1989), are presented in Table 2-7.

TABLE 2-7
Recommended Fire Flow

Required Fire Flow (gpm)	Duration[1] (hours)
2,500 or less	2
3,000 to 3,500	3
4,000 to 12,000	4

[1] Source: Fire Protection Handbook, 1989

The 3,500 gpm maximum fire flow requirement was used in hydraulic modeling of the distribution system. Fire protection is not dependent on the water distribution system alone; required fire flows could be reduced by individual fire suppression systems, such as a sprinkler or chemical system, and by alarm systems.

The last ISO fire flow survey of the Warrenton water system was completed in March 1996. It was given a Class 5 fire insurance classification, an improvement from the prior Class 6 classification. The Warrenton water system was graded with 24.95 points out of a possible 40 points. Ten locations in the water system were tested for fire flow availability in the 1996 survey. The results of the ISO survey are summarized in Table 2-8. The ISO fire flow requirements and their locations shown in Table 2-8 were the criteria used to analyze the fire flow capability of the Warrenton water system under 1996 and 2016 demand conditions. The 1996 ISO survey report is included in Appendix C.

TABLE 2-8
ISO Fire Flow Survey, 1996

Location	Fire Flow Avail @ 20 psi (gpm)	ISO Required Fire Flow (gpm)	Duration (hrs)
S. Main and Cemetery Rd. (Comm./Res.)	2,500	3,500	3
S.W. Cedar and S.W. 9th (Comm./Res.)	1,500	7,000[1]	4
Airport Rd. and C.G. Rd. (Comm.)	700	6,000[1]	4
S. Main and S.W. 2nd (Comm.)	2,400	2,250	2
Skipanon Dr. and Harbor Pl. (Comm.)	1,100	1,500	2
Warrenton Dr. and N.W. 13th (Comm.)	3,300	4,500[1]	4
Heceta Pl. and Pacific Dr. (Comm.)	1,100	5,000[1]	4
Pacific Dr. and Lake Dr. (Comm.)	950	1,750	2
Russell;; Dr. and Russell Pl. (Comm./Res.)	750	2,000	2
Hwy. 101 and Harbor St. (Comm.)	3,500	2,250	2

[1] Fire flow requirements that are larger than 3,500 gpm are evaluated individually and not used by the ISO in determining the public protection classification of a municipality.

Water Conservation

Figure 2-1 illustrates the projected maximum day demand through year 2016, and the impact of a 10 percent reduction phased in over 20 years. Maximum day use is the most important factor for Warrenton's system. A reduction in the MDD will delay costly expenditures to expand source and treatment facilities. For this reason, we recommend that conservation measures implemented by Warrenton focus on reducing peak summer use.

In general, conservation programs can be categorized into supply side and demand side management programs. Supply side measures are directly controlled by the city, and include any measures that reduce water loss between the source and delivery to customers. The primary supply side measure is locating and repairing leaks. Other supply side measures include controlling production to reduce or eliminate overflow from storage tanks, and selecting an efficient treatment system that minimizes wash water use.

Although demand side conservation measures depend on customer response, there are a number of programs that promise significant results for the city. Example demand side measures include:

1. Install meters for all customers.
2. Incentive based water rates. Provide an economic incentive to reduce water use by means of an increasing block rate or peak summer differentials.
3. Plumbing codes. Adopt city plumbing codes for new construction that require the use of water efficient fixtures.
4. Shower head and faucet retrofits. Provide and install water efficient shower heads and faucets at low cost or free.
5. Audit of large demand customers. Train a staff member in performing water efficiency audits and provide audits of large water users to determine if they have inefficiencies.
6. Public information and education. Promote careful water use by billing stuffers, displays at fairs and community events, and through presentations at schools.
7. Odd/even watering schedules. Encourage or require that outdoor watering be limited to an odd or even date, depending on whether the property address is odd or even. Note that some waterworks professionals regard odd/even watering restrictions as an emergency curtailment measure rather than a conservation measure.
8. Landscape irrigation. Provide a display and information on low water use landscaping.

The first and most important step for Warrenton is to meter all customer connections. This should reduce water use and enable the city to track future use patterns. Following installation of meters, the city can implement a combination of the above conservation elements to further reduce maximum day water use.

The rate evaluation portion of this master plan study and the city's plans for installing a filtration plant also offer opportunities for the city to implement conservation measures. We recommend that the city's rate be structured to reduce peak summer use. The process of funding and constructing the filtration plant will provide a platform for public education and how conservation can provide significant economic benefits for customers.

Curtailment

Along with developing a conservation program, we recommend that Warrenton develop a curtailment plan. Curtailment refers to measures the city takes to achieve immediate reductions in water use during a crisis period. The crisis may either be weather-related, an unusually long hot and dry period, or it may be an emergency shortage caused by failure of a major system component such as a pipeline or pump station.

A workable curtailment plan should provide specific answers to the following questions:

- Who makes the decision to implement the plan?
- When is the plan implemented? What are the trigger points?
- What curtailment measures will be implemented? Are the restrictions voluntary or mandatory?
- How will the need for curtailment be communicated to the customers?

It is common for a curtailment plan to include two to four stages for dealing with progressively worsening shortages. For example, the first stage might consist of voluntary requests for limited outdoor watering and is triggered by demands that reach 90 percent or higher of the supply capacity. A later stage might prohibit outdoor watering. It is valuable to develop a plan now, before a crisis occurs, so that consequences of curtailment can be more fully considered. One question to address is how will various user groups shoulder the burden of reduced water use. Will it mainly fall to residential users so that the fish processors are not financially impacted? Or, will industries face mandatory restrictions as well?

CHAPTER 3

Supply Planning

Chapter Summary

Warrenton faces a potential water shortage. A comparison of projected demands to the yields expected during a drought reveals that periodic shortages will occur. In the near term, these will probably last for only a few days. If demands reach the levels projected in Chapter 2, the possibility exists of a month-long shortage in year 2016.

A yield analysis was not included as part of the scope of this master plan. Therefore, we have relied on the city's recent experiences and conclusions in other studies to estimate drought yields. It appears that during a drought, the city's present surface sources may produce as little as 3.7 million gallons per day (mgd) when averaged over a month's period. This would likely occur in August, September, or October. Yields are known to have fallen below 3.0 mgd for shorter periods of time. In past years, demands have reached as high as 5.3 mgd on a maximum use day and 3.3 mgd for a maximum monthly average. By year 2016, we have projected demands to reach 7.5 on a maximum day and 4.8 mgd for a maximum monthly average. Using these numbers, the system could be short 1.1 mgd for an entire month's time in year 2016.

The above analysis assumes that the city is allowed to capture 100 percent of streamflow. While it appears that the city's water rights allow this, it is not certain that the city should rely on such practice given state and nationwide concerns for salmon restoration.

To meet the shortfall, the city could 1) add more raw water storage in the watershed, 2) develop a groundwater supply, or 3) implement a formalized and rigorous conservation program. A combination of two or three approaches may be required. The time to plan and construct either a new impoundment or a groundwater supply may be three to five years or longer.

Demands

Chapter 2 presents water demand projections for the City of Warrenton. The crucial demand value for source planning is the MDD. This is the amount of water needed to supply all system requirements (all metered use plus unaccounted for water) on the peak use day of the year. The MDD will usually occur in the summer when outdoor watering and seafood processing are at their peak.

As described in Chapter 2, available data indicate that the MDD in 1994 was 5.14 million gallons per day (mgd). Data for 1995 and 1996 were not available, but their values were estimated as 5.2 and 5.3 mgd, respectively. Chapter 2 presented projections for future demands, and according to these projections, the MDD will increase to 7.5 mgd by year 2016.

Factors that influence demands include population growth, industrial growth (or decline), and the impact that residential metering and conservation might play in reducing per capita use. Uncertainties in all categories suggest that the city should carefully track demands and recognize that new source capacity may be required even prior to the projections of this plan.

Weather also influences MDDs. An unusually hot and dry period in the summer may contribute to a higher-than-projected MDD for any given year. This is particularly true if the hot and dry period coincides with high levels of water use in the fish processing industries. Therefore, it is advisable to maintain a reserve in excess of projected MDDs to account for unexpected annual variations.

Existing Supply

Warrenton's water supply comes from the Lewis and Clark River and three of its tributaries. The supply system includes a 17 mg off-channel raw water storage impoundment located in the watershed, downstream of all intakes. Any or all of the four intakes and the impoundment can contribute water at any given time, based on manual valve operation by the water system operators.

The city can also draw from a 10-inch diameter, emergency intertie pipe connected to the City of Seaside's water system. By agreement with Seaside, this is only to be used for emergencies, and therefore, is not considered an addition to Warrenton's supply.

Water Rights

The city holds water rights to 25 cfs (16.2 mgd) in the Lewis and Clark River, and a total of 12 cfs (7.7 mgd) from the three tributaries. The 23.9 mgd total far exceeds the 2016 MDD projection of 7.5 mgd. There is no indication that other more senior rights will challenge the city's rights, although an examination of water rights was not included in this study. We recommend that such an evaluation is commissioned by the city.

Fish Passage

City staff indicated that there have been periods in each of the last six years when the entire flow of the Lewis and Clark River was diverted by the city's intake and no water passed over the fish ladder around the intake impoundment. These periods, occurring in late summer and early fall, have lasted up to two months at a time. Although the city is acting within their water right, they are concerned about the environmental consequences of this practice.

The time when flow is needed over the fish ladder depends on the fish species that are present and their migration times. If the species are coho salmon and steelhead, the key periods may be limited to November and December for in-migration, and May and June for out-migration. It may not damage these fish runs if the city diverts the entire river flow during August and September. This is only speculation, and is subject to Oregon Department of Fish and Wildlife's evaluation. We recommend that Warrenton open discussions on this topic with ODFW. It is an especially important issue in light of the Governor's Salmon Initiative that was begun in 1997.

Yield Analysis

The scope for the master plan did not include a yield analysis for the city's surface sources. City staff suggested a value of 6.0 mgd for the drought yield, primarily based on the previous master plan.

No flow records are available, since neither the mainstem of the Lewis and Clark River nor its tributaries have been gaged. Two other studies have estimated the drought yield by comparison to nearby watersheds where flow records are available.

- The city's 1979 master plan estimated the reliable drought yield to equal 6.5 mgd. A subsequent (November 1979) memo from Donald Lampi, Administrative Assistant for the City of Warrenton at the time, questioned this value. He believed that the drought yield might be as low as one-third the 6.5 mgd. He further questioned the validity of assuming that the city could withdraw all of the streamflow during a drought flow situation, leaving non for fisheries.

- A March 1997 draft of a report prepared by Woodward-Clyde Consultants for the Clatsop County Regional Problem Solving Pilot Project presents yield estimates for Warrenton's supply (personal correspondence with John Davis). They estimate that the average monthly yield will equal 3.7 mgd for one month a year one time every 17 years. The second lowest monthly average yield once every 17 years was estimated to equal 7.4 mgd. They developed these estimates using a 17-year period of record that was available for the Necanicum River, and comparison of the two watersheds. The period was for October 1977 through September 1995, with the exception of October 1991 through September 1992.

Although the city does not have flow records for the watershed, city staff did record a video of the stream conditions for all four intakes during the drought that occurred in September 1991. On the particular day recorded, demands were less than 3 mgd and all streamflow was withdrawn from the watershed. Over a seven-day period, system demands averaged 3.1 mgd and average withdrawal from the 17 mg raw water impoundment was 0.4 mgd (a 2-inch drop in depth over the 6.5-7.0 acre area of impoundment). A portion of the reservoir loss can be attributed to evaporation. Assuming that portion is small, the seven-day yield averaged approximately 2.7 mgd (3.1 mgd - 0.4 mgd).

Recommended monthly average drought yield = 3.7 mgd

The city's experience in 1991 suggests that a monthly average drought yield of 3.7 mgd could indeed occur. Until better flow records are developed, we recommend using a monthly average drought yield of 3.7 mgd as the basis for planning. The accuracy of this value is limited because it represents a comparison with another watershed and is based on a relatively short period of record. This approach also is based on capture of all water from the streams, which may or may not be acceptable in practice in the future, as described previously under the section, *Fish Passage*. If the true drought yield on a monthly basis is 3.7 mgd, it is expected that shorter periods (1 week or 1 day) will have significantly less supply than 3.7 mgd. This was the city's experience in 1991, when flows during an entire week averaged 2.7 mgd.

Supply Need

Figure 3-1 illustrates a drought yield of 3.7 mgd compared to projected maximum day demands. This chart illustrates that the city's demands, even at current levels, will exceed supply during a drought year.

As Figure 3-2 illustrates, the average monthly demands are significantly less than the MDDs. In 1994, the average demand during August was 3.3 mgd, whereas the maximum day use, which occurred in August, was 5.1 mgd. Assuming the same ratio for year 2016, the average monthly demand for August will equal 4.8 mgd and for July, 4.5 mgd. If a drought occurred in August, the deficit between supply and demand would total 34 mg. This amount is twice the 17 mg of existing raw water storage.

Further evaluation beyond the scope of the master plan is necessary to reach more definitive conclusions on supply needs. The preceding discussion clearly identifies the need for Warrenton to begin immediate planning for expanding its supply.

Supply Planning
Warrenton is facing a near-term supply shortage
Watershed yields have not been quantified
Uncertainties in estimating yields and projecting demands suggest the need for a conservative approach in supply management planning

Supply Expansion Alternatives

Although the timing and sizing for additional supply are uncertain, the need is well-enough defined to begin planning specific improvements. The two most likely approaches are to expand the existing surface water supply by adding more raw water storage, or the installation of a groundwater supply in Clatsop Plains. Either alternative will require several years to implement, because of the planning, permitting, design and construction requirements. Conservation provides a third approach to meeting the pending supply deficit. A formalized, rigorous conservation program may succeed in delaying the need for source development for many years.

Surface Water Supply Addition

The city currently uses a 17 mg raw impoundment. It appears that the volume in this existing impoundment is marginally adequate to meet current needs, and will not be capable of meeting the city's needs in just a few years if demands grow as projected and a drought occurs.

Figure 3-1
Source Needs

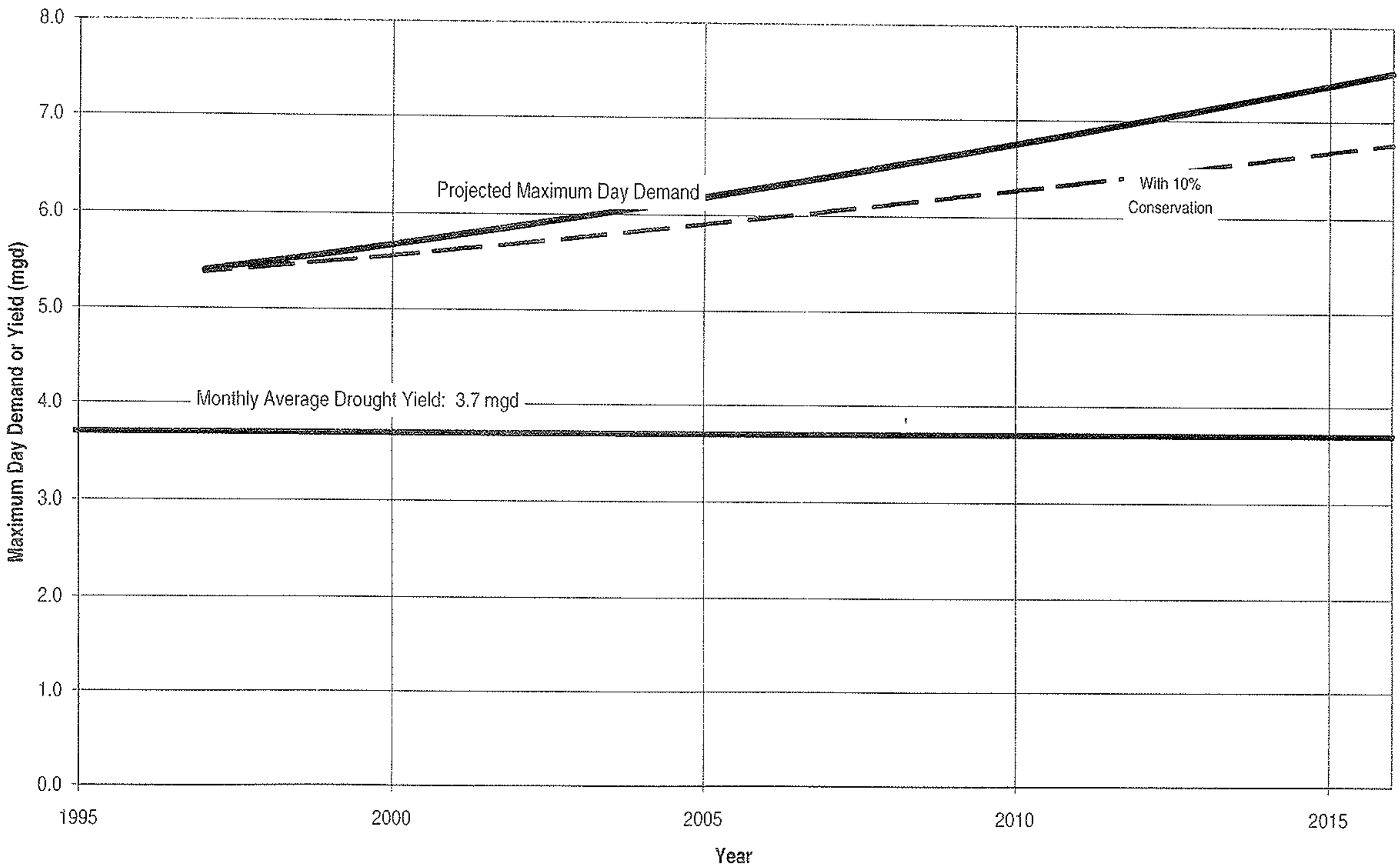
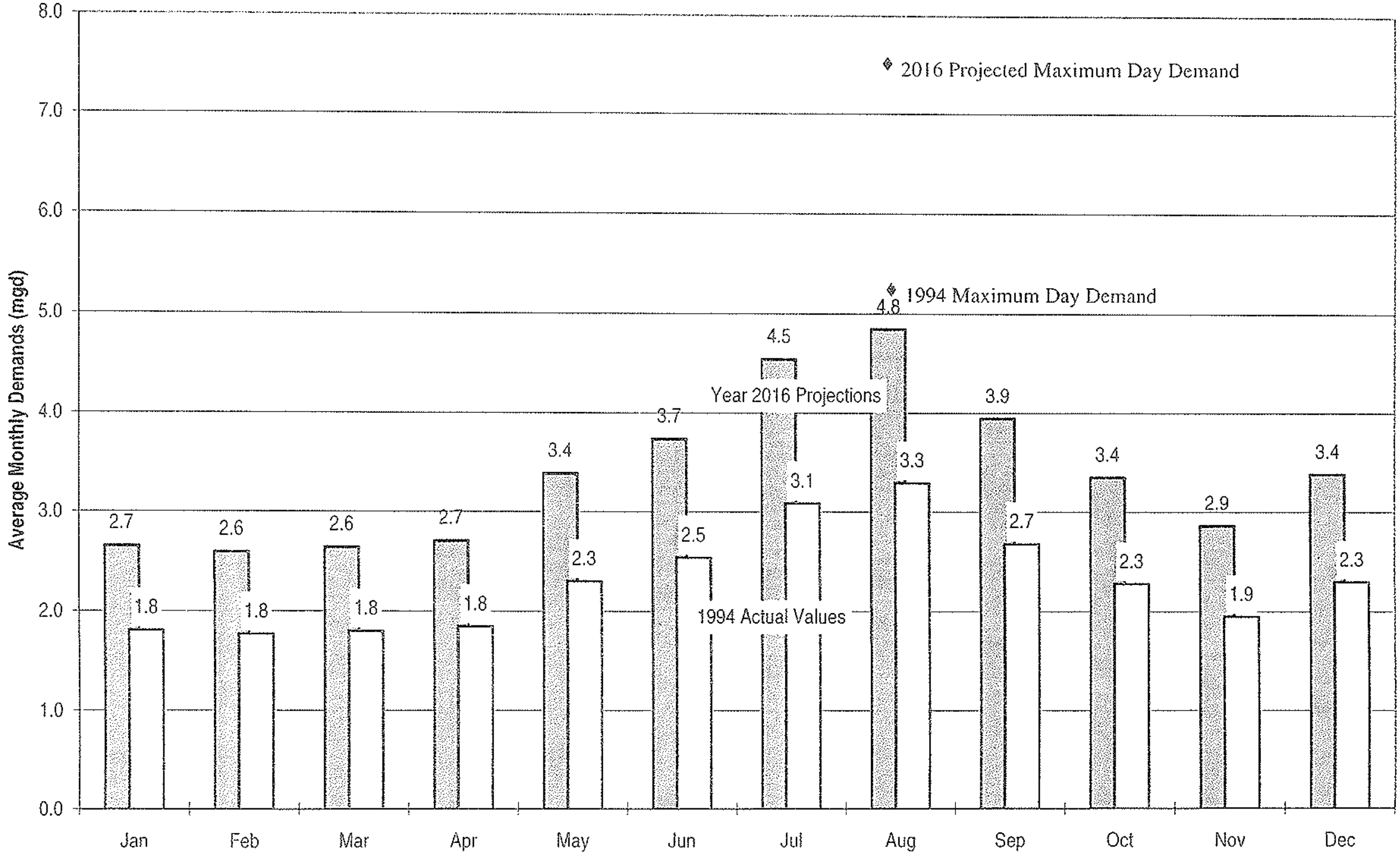


Figure 3-2
Average Monthly Demands for 1994 and 2016



Adding a second impoundment of similar or larger size is one means of addressing the projected supply deficit. It appears that land adjacent to the existing impoundment could accommodate another off-channel impoundment. Costs to install a second 17 mg impoundment are included in the capital improvements plan, although further evaluation is warranted to determine the appropriate size (and to determine if a second surface impoundment is preferable to developing a groundwater supply). A second 17 mg impoundment cannot be expected to provide sufficient storage beyond the 20-year planning period.

The 1979 master plan presented the alternative of constructing an in-stream dam on the South Fork Tributary. The proposed location was 365 feet upstream from the existing impoundment. By constructing a dam 38 to 50 feet high, a total of approximately 20 mg could be stored over a surface area of 3 acres. This gives an average depth of 20 feet. The study proposed an overflow elevation of 412 feet. Apparently, no additional work since that time has been performed in investigating this site.

Groundwater

Developing an alternative supply such as groundwater is a second approach to addressing the supply deficit.

There are several obstacles to developing a groundwater supply from Clatsop Plains, as noted in Chapter 4. During preparation of this master plan, city staff evaluated these constraints and determined that adding a second surface impoundment was more likely than adding a groundwater supply. However, further evaluation of the groundwater alternative may be warranted depending on a more accurate determination of the magnitude of the supply deficit and the costs to develop a groundwater supply. Groundwater offers the advantage of an independent, secondary source, making Warrenton's system more reliable. The potential for using aquifer storage and recovery is also discussed in Chapter 4. Its use may eliminate the need for costly iron and manganese treatment, making a groundwater alternative much more attractive.

If a groundwater supply is used, individual wells could be brought on line in small increments to meet growing demand needs. Rather than the one large expansion, a groundwater supply could be added well by well, thereby delaying capital expenditures.

Source Planning Recommendations

Because Warrenton faces a potential water supply shortage, it is important that specific source planning measures be commenced immediately. Several years will be needed to realize benefits from conservation measures or to implement either a groundwater or surface source addition.

Chapter 4 describes the evaluation criteria for a Clatsop Plains groundwater supply. Some of the steps include investigating water quality (especially for nitrates), siting the wellfield, determining safe well yields, permitting (including water rights and land use), designing the wellfield and treatment, and constructing the facilities. There are also preliminary steps identified for assessing the potential for using aquifer storage and recovery. We recommend beginning the water quality investigation process in 1997, along with participating in land use planning discussions through the Clatsop Plains Regional Problem Solving project.

Recommendations

Carefully track daily water use throughout the summer and early fall
Track watershed yield during low flow periods by installing stream gaging stations
Confirm seniority of water rights
Evaluate need for fish ladder flows with ODFW
Consider adding another raw water impoundment as part of the plant construction
Initiate early planning steps for a groundwater supply
Evaluate the potential for aquifer storage and recovery
Prepare a Water Management Plan

Bringing a new surface water impoundment on line involves permitting as well. Further investigation of either the proposed site for a second impoundment or a dam on the South Fork Tributary, and contact with the permitting agencies will be needed to identify the permitting needs, constraints and implementation time frame. Three or more years may be required for environmental studies, public review, mitigation steps, and obtaining water rights.

Either a new impoundment or groundwater supply requires a water rights permit. One component of a new water right is developing and submitting a Water Management Plan to Water Resources Department. The intent of the plan is to demonstrate that the city is and will manage its use of the water resource in a sound fashion. The city's commitment to conservation measures is necessary for plan adoption.

CHAPTER 4

Groundwater Availability

This chapter describes a preliminary evaluation of the Clatsop Plains groundwater aquifer as a potential supplemental supply for the City of Warrenton. This was identified as the next increment of supply to be developed for the city in the 1979 master plan.

Chapter Summary

The Clatsop Plains Aquifer may be a feasible source for producing 1.5 to 2 mgd of water to supplement the existing City of Warrenton drinking water supply. Previous studies of the dunal aquifer system along the Oregon coast suggest that potable water in sufficient quantity and quality can be obtained from aquifers of coastal dune sands. However, there are a number of important issues that must be resolved regarding saltwater intrusion, elevated concentrations of nitrate and iron at some locations, impacts to dunal lakes caused by lowering the water table, and land use compatibility. Developing a water supply system in the Clatsop Plains sand dune area will require addressing these issues during the planning stages as well as during operation of this new system.

The most serious issue is the potential for high nitrate levels. We do not recommend development of a supply with raw water nitrate levels above the drinking water standard, unless absolutely necessary to meet Warrenton's supply needs. Data from monitoring wells near the proposed wellfield location suggest that nitrate levels are not problematic, but further testing is warranted. Groundwater from other locations in the Clatsop Plains aquifer has shown elevated nitrate levels.

Treatment for high iron and manganese will almost certainly be required. However, these two constituents have only secondary, non-health related standards. Treatment for iron and manganese is common for drinking water applications, though not inexpensive.

An alternative approach to using the aquifer is termed aquifer storage and recovery (ASR). ASR involves storing treated water from the city's existing surface supplies in the aquifer during the winter months, and withdrawing the same water during the summer or an emergency. ASR may address some environmental concerns with using the Clatsop Plains aquifer as a direct groundwater supply and it may eliminate the need for expensive iron and manganese treatment. However, further technical evaluations are needed to confirm that ASR is feasible for this aquifer.

Background

There are several previous studies of the Clatsop Plains sand dune area as well as two currently ongoing investigations. The most significant previous studies include a water supply study conducted in the late 1960s by the United States Geological Survey (*Water-Supply Paper 1899-A*, by J. Frank, 1970), and a septic leachate carrying capacity study of the dunal aquifer by Environmental Geology and Groundwater Company (R. Sweet 1977).

Current investigations being conducted in the area include a water level and water quality monitoring program by the Oregon Department of Environmental Quality (DEQ), and a regional land use planning effort funded by the Land, Conservation and Development Department (LCDC). The following section is a summary of technical data from these previous and current investigations.

A vicinity map of the area is provided in Figure 4-1.

Geologic and Hydrologic Setting

The productivity of an aquifer is directly related to the geology of the water-bearing units, the recharge and discharge conditions, and the climate and physiography of the area. These characteristics of the geologic formations in the Clatsop Plains area are described below.

Geology

The bedrock underlying of the northwest coastline of Oregon is a low permeability, fine grained sandstone and shale (Astoria formation). These rocks can be seen in the edge of the coast range east of the Clatsop Plains area. This formation yields small quantities of water that has poor quality. This bedrock underlies the Clatsop Plains sand dune deposits along the shoreline at a depth of over 100 feet.

The coastal dunal formation overlies the eroded surface of the bedrock. This water-bearing unit is over 100 feet thick and contains loose and unconsolidated fine to medium sand. The sand at and near the surface is wind-deposited material, and the unit at depth probably contains interbedded layers of ocean deposited and wind-blown sand material. The dunal formation also contains some discontinuous silty layers within the sand deposit.

Hydrology

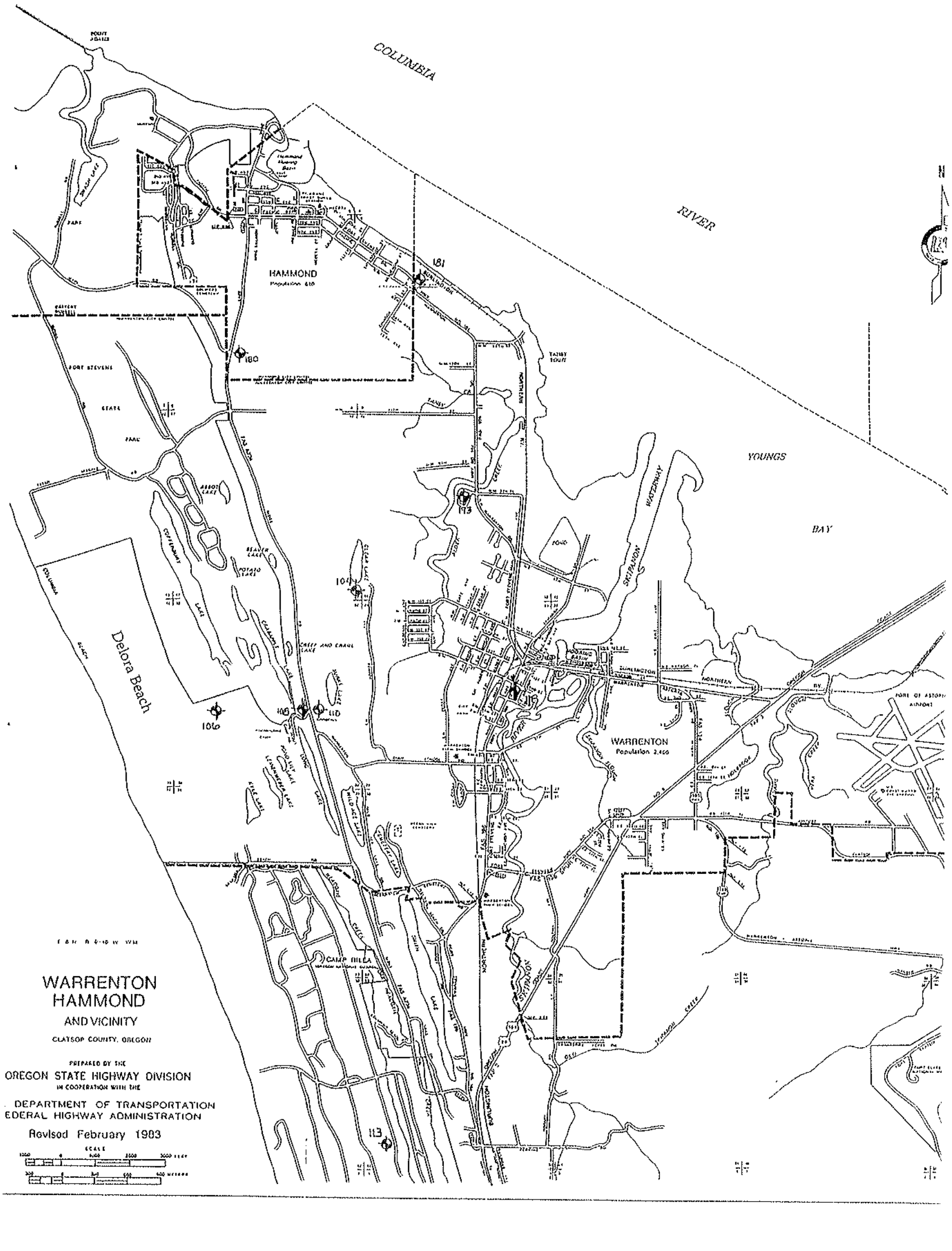
The Clatsop Plains Aquifer is an unconfined aquifer that has a water table contour that roughly coincides with the sand dune topographic contours of the area. The aquifer receives recharge mainly through infiltration of up to 60 inches of precipitation per year. Because there is little runoff, most of this precipitation infiltrates downward through the sand and recharges the aquifer. This amount of recharge is more than adequate to support long term pumping of 2 mgd without causing significant drawdown. Discharge of water from the aquifer is mainly through underflow to the Pacific Ocean, seeps, and small localized streams. The dunal aquifer is also directly connected to the dunal lake system. The aquifer study conducted by the USGS concluded the aquifer transmissivity was a minimum of 27,000 gpd/ft (gallons per day per foot). This value indicates that the aquifer can readily transmit water to wells.

Potential Yields

The potential yield of the dunal aquifer system will require maintaining a balance between the recharge, the natural groundwater discharge, and the pumping system. This balance will maintain the water levels in the dunal lakes and minimize the encroachment of salt water into the currently fresh water portion of the aquifer system. The yield that will maintain the balance point may be dynamic over time and will have to be evaluated on a continual basis.

Figure 4-1
Vicinity Map
Groundwater Monitoring Wells

DEQ Monitoring Well



We have estimated the potential yield of the Clatsop Plains Aquifer using previous measurements and calculations by Frank (1970), and by considering CH2M HILL's recharge evaluations for a similar aquifer near Coos Bay. Our preliminary determination is that the aquifer could support 200 gpm wells spaced 1,000 feet apart. This level of production would keep pumping interference to an acceptable level, and the recharge capability of the system would sustain this pumping on a long-term basis. This means that 5 to 7 wells are needed to meet the production goal of 1.5 to 2.0 mgd (1,000 to 1,400 gpm). These values are preliminary. Additional assessment of the total aquifer system combined with a review of current land uses is needed to refine these pumping rates.

Issues Affecting Groundwater Development

Saltwater Encroachment

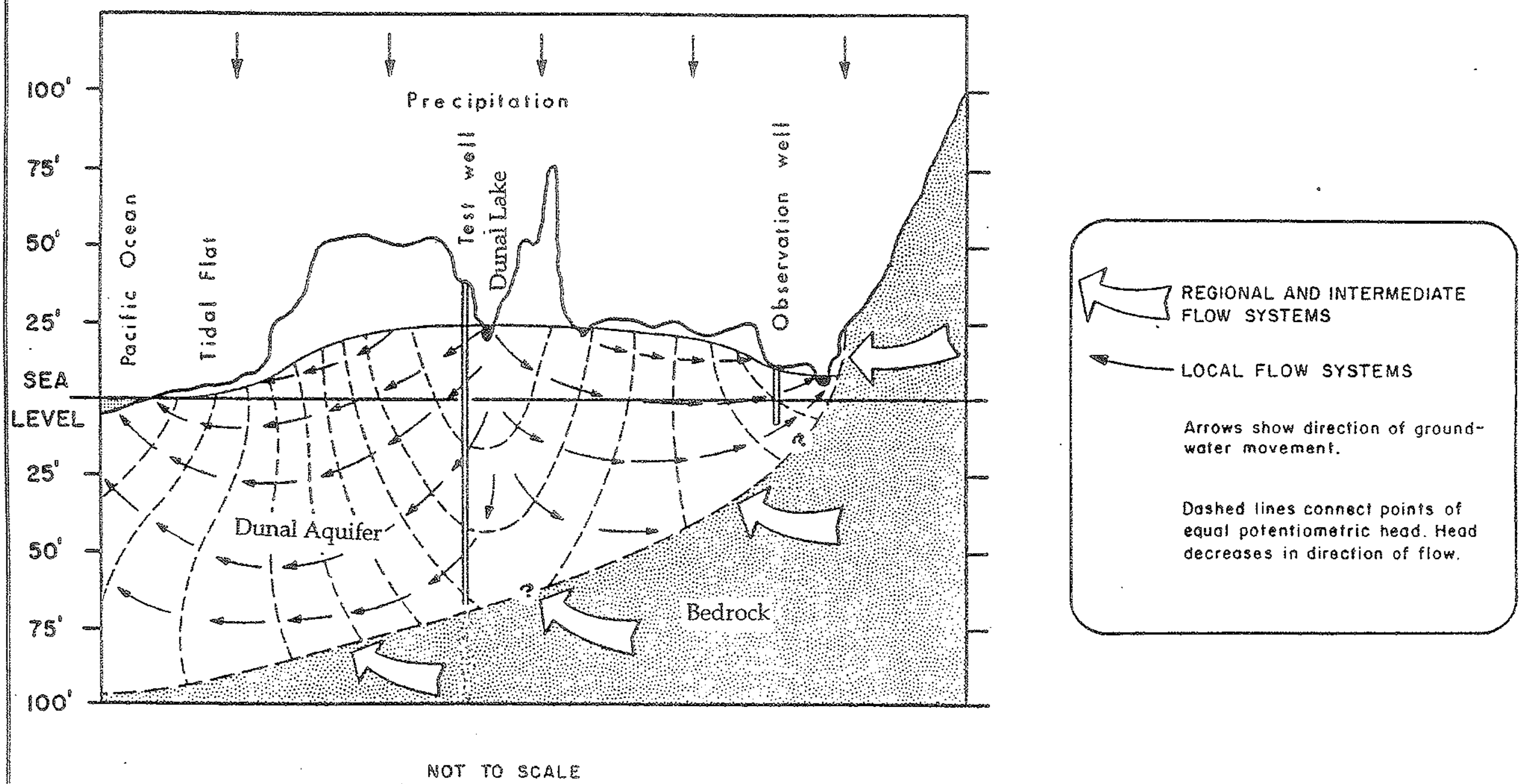
Saltwater (seawater) intrusion is a significant factor constraining aquifer development in many coastal aquifers. Saltwater intrusion can occur by over-pumping the fresh water aquifer which causes a landward migration of the interface between saltwater and fresh groundwater. Saltwater encroachment from aquifer pumping can be avoided through proper evaluation of the aquifer system and careful management of pumping rate and well locations. CH2M HILL's computer modeling of the Coos Bay-North Bend dunal aquifer, one that is similar to the Clatsop Plains Aquifer, concluded that a conservative pumping rate of approximately 200 gpm per well would not cause migration of the saltwater interface. Further study of the Clatsop Plains area will be needed to evaluate the location of the interface and the rate at which wells can be pumped without affecting the interface. Pumping from the dunal aquifer will require long term monitoring of the saltwater/fresh water interface location for possible movement.

Impacts to Dunal Lakes

The extent of hydrogeologic connection between the numerous dunal lakes and the groundwater flow patterns was not examined in previous studies. Figure 4-2 shows a schematic example of the relationship between groundwater flow patterns that may exist in the dune system. Lowering the water table near the dunal lakes by pumping groundwater may promote a decrease in lake levels and affect wetland vegetation. The dunal lake levels and their relationship to the groundwater flow pathways will need to be evaluated so that well locations and pumping rates can be selected to minimize adverse impacts to the lakes ecosystems caused by pumping.

Water Quality

Water quality analyses have been performed on several wells that penetrate the sand dune aquifer. Most the data we reviewed was provided by Rodney Weeks at DEQ. DEQ conducted two rounds of sampling in 1996; one in April and one in October. Three wells (No. 106, 108, and 110) are in the general vicinity of Delora Beach, the area tentatively identified as a possible wellfield site. Well No. 106 is closest to the beach and wells No. 108 and 110 are located further east near the old landfill (Figure 1). The groundwater quality was low in nitrates (0.02 to 1.4 mg/L) and total dissolved solids (160 mg/L), but high in iron (3.2 mg/L) and manganese (0.07 mg/L). Atrazine, an herbicide, was detected once at



Note: Figure from Environmental Geology & Ground Water Report titled "Carrying Capacity of the Clatsop Plains Sand-Dune Aquifer" dated August 1977.

Figure 4-2
Clatsop Plains Groundwater Flow
System Schematic

well No. 108 but it is unclear if this detection is anomalous. In addition, a sample at well No. 106 had an elevated lead concentration. It is possible that this detection is a result of decomposition of the galvanized steel well casing.

Nitrate is likely the constituent of most concern. This type of aquifer is susceptible to leaching of nitrate from septic systems and other human activities. The low concentrations found near Delora Beach suggest limited influence from these sources, although the landfill to the east is a potential source of contamination to the aquifer. The drinking water standard is 10 mg/L. While effective treatment exists for nitrate removal, it is generally recommended that raw water sources with high nitrate levels be avoided. We recommend further monitoring for nitrates coupled with land use controls to assess the potential for elevated nitrate levels.

High iron and manganese concentrations are common in dunal aquifer systems. Both constituents have secondary standards only, meaning they pose aesthetic and not health problems in drinking water. The secondary standard for iron is 0.3 mg/L; the level measured in the Delora Beach area was about 10 times higher. The secondary standard for manganese is 0.05 mg/L, slightly less than the 0.07 mg/L concentration measured.

Elevated iron concentrations in the dunal aquifer at Coos Bay were shown to be associated with the vegetation and organic matter present in the dune area. The Coos Bay study also indicated that iron concentrations varied greatly across the aquifer. We expect that these same conditions are true for the Clatsop Plains Aquifer. Iron levels appear to be highest north of Delora Beach, where there is a greater density of vegetation.

The available data suggest that treatment for iron and manganese will be required.

All three wells have low salinity; however, they are not deep enough to determine if the salinity increases with depth. Further evaluation for salinity indicators such as chlorides and total dissolved solids is warranted.

Land Use

The current land use in the Clatsop Plains sand dune area primarily consists of recreation and residential housing. Additional development is expected to encroach upon the sand dune area, although it may be restricted because of the limited capacity of the dune sands to process septic drain field leachate.

The Clatsop Plains Aquifer is vulnerable to contamination from storm water runoff, commercial activities, and septic drain field leachate. Because of this vulnerability, we recommend conducting a contaminant source inventory and developing an aquifer protection plan that would guide future development in the area while also protecting the drinking water supply, should a groundwater supply be developed.

Regional Planning Efforts

In 1995 the legislature established a fund for a Clatsop County Regional Problem Solving (RPS) pilot project. The purpose of the RPS project is to develop guidelines for making land use decisions in the county. Issues including water quantity, water quality, land use, environmental concerns (e.g., wetlands), and recreation are being studied so that alternatives for sustainable development can be developed. Fifteen agencies and groups (including the City of Warrenton) are members of the RPS team. The study will be completed by the end of June 1997. After the RPS study is completed, an implementation plan will be prepared that identifies specific zoning requirements and establishes cooperative agreements between agencies to facilitate land use decisions. The implementation plan is scheduled to be completed in 1998.

The RPS project provides an opportunity for the city to gain support for a wellfield site and other system improvements. For example, if the city chooses to develop a wellfield in the Clatsop Plains area, the city could petition for portions of this area to be designated for land uses compatible with protecting groundwater quality. This would be a significant first step in developing a wellhead protection program. The RPS project could also be a vehicle to educate the citizens of Warrenton about the need for developing and protecting a supplemental groundwater supply.

Steps Required to Develop a Water Supply System in the Dunal Aquifer

If the City of Warrenton chooses to pursue the dunal aquifer system as a supplemental water supply , additional field investigation of the specific areas for future wells is required. The scope of the additional work should include:

- Determine if the aquifer can produce up to 2 mgd of water meeting drinking water standards.
- Identify treatment needs for iron and manganese. The probable treatment technique for iron and manganese is oxidation, sedimentation and filtration.
- Evaluate site specific hydrogeologic parameters, including aquifer permeability, well spacing, and aquifer recharge potential.
- Apply for water rights.
- Determine a groundwater development approach that meets the needs of the city without causing adverse impacts to the dune ecology and dunal lake system.

A central component of the water quality task is to determine nitrate levels in the target wellfield area, and the potential for future nitrate contamination. Elevated nitrate levels (above approximately 10 mg/L) may present a fatal flaw to the Clatsop Plains supply. We recommend an initial water quality monitoring program focusing on nitrate, iron, manganese, total dissolved solids, and chloride. The full aquifer study will need to include analysis for all Safe Drinking Water Act contaminants.

Development of a water supply system will require completion of a technical study while simultaneously conducting a planning and coordination effort to educate the public about the project, to identify funding for the project, and to integrate with the regional planning effort presently underway. The technical and planning areas should be performed simultaneously so that the technical information can be incorporated into the regional planning and public interest process. This approach will save time in bringing the new system on line.

The technical evaluation of the dunal aquifer system would include completion of the following steps.

1. *Siting the Wellfield.* Issues that should be evaluated in locating a suitable wellfield location include:

    a) Cost to acquire the land

    b) Proximity to the present water conveyance system

    c) Compatible land uses surrounding selected wellfield site

    d) Proximity to contaminant sources (predecessor to a wellhead protection plan)

        i) septic fields

        ii) environmental concerns

    e) Proximity to naturally poor water quality (iron or saline)

    f) Impacts to surface water bodies (dunal lakes)

    A wellfield approach, consisting of a group of wells each with a capacity of 200 gpm, will likely result in fewer negative impacts on the dunal system than using 2 or 3 wells pumping at higher rates. Based upon a preliminary review of land use in the Clatsop Plains area, it appears that the Delora Beach area (identified in the city's previous water master plan) may be suitable for developing a wellfield. This location is identified on Figure 4-1. This land is owned by the county, is not highly developed, and is reasonably close to the Warrenton distribution system.

2. *Site Specific Groundwater Evaluation Field Program.* After a wellfield site is identified, a specific study should be conducted to evaluate the following conditions of the dunal system:

    a) Aquifer and well yield

    b) Water quality

    c) Water budget (estimate long term sustainable yield)

    d) Impacts to local dunal lakes and possible effects from pumping

    e) Saltwater encroachment

    The field program should include installation of a test production well and several monitoring wells, including multi-level wells near the beach edge to monitor the location of the saltwater/fresh water interface and any effects from pumping. Existing private wells and DEQ monitoring wells should be incorporated into this field program to minimize the field study costs where possible. In addition, a detailed surface water system study (including water level and flow measurements) should be conducted for evaluation and design of the wellfield. Aquifer water quality will be evaluated to assess the need for treatment.

3. *Permitting and Wellhead Construction.* If the field program and initial planning and coordination activities indicate that favorable conditions exist for developing a wellfield system, several additional steps will be required to bring the new water system on line. The major additional steps include:

   a) Obtaining a groundwater permit from the Oregon Water Resources Department (will require preparation of a water management plant)

   b) Preparation of an aquifer development plan

   c) Wellhead and piping design

   d) Wellhead and piping construction

   e) Water level monitoring during initial operation

   f) Long term water level monitoring

An alternative approach is to further investigate water quality before investing in the full evaluation of the aquifer as described in the preceding outline. More information is needed concerning the quality of groundwater throughout the Delora Beach area. Data from additional wells is needed to better characterize the spatial variability and the type of treatment required. A limited study could be conducted that involves the installation of three monitoring wells and collection of samples for analysis of total dissolved solids, nitrate, iron, manganese and chloride. We recommend locating the wells 2,000 feet apart within the Delora Beach Park owned by Clatsop County. After the wells are installed, water samples could be collected by city staff on a quarterly basis. The analyses are routine and can be performed at a number of laboratories. CH2M HILL could prepare a brief sampling and analysis plan for use by the city. Not only the what but the how of sampling is critical to obtain meaningful results.

Aquifer Storage and Recovery (ASR)

ASR is a modified groundwater approach to meeting Warrenton's supply needs, wherein treated water from the surface sources is injected into the aquifer during winter months, and withdrawn during summer months. Like a supplemental groundwater supply, ASR also provides an emergency supply that is independent of the surface supplies.

Although a relatively new technology, ASR is quickly being implemented throughout the nation and worldwide. In Oregon, Salem is installing an ASR system capable of producing up to 20 mgd. The Joint Water Commission in the Hillsboro area is also installing an ASR system.

In Warrenton's case, ASR may offer a significant cost advantage over a direct groundwater supply from Clatsop Plains by eliminating the need for iron and manganese treatment. It may be possible to control iron and manganese levels to below drinking water standards after several recharge cycles. Elimination of treatment likely changes the Clatsop Plains alternative from being cost-prohibitive to cost-effective. ASR may also reduce or eliminate some environmental concerns in the aquifer, particularly the concern about impacts to the dunal lakes.

Some additional investigation is needed to confirm ASR's applicability in the Clatsop Plains aquifer. Redox and pH water quality information is needed, as well as samples of the aquifer matrix for laboratory analysis. We recommend these tests as part of the limited aquifer investigation package described below.

Project Costs

The limited water quality and ASR investigation, described as an alternative preliminary step to the full aquifer investigation, is estimated to cost $25,000. This includes $4,000 to engineer the monitoring wells, prepare a sampling and analysis plan, and to assist with the first round of sampling; and $10,000 in construction costs for installation of the three monitoring wells. Quarterly sampling for one year is estimated to cost $1,000. A total of $10,000 is allocated for ASR investigations.

Table 4-1 lists budget level cost estimates for completing the aquifer investigation: initial planning, site specific groundwater investigations, permitting, and wellhead construction. These costs are provided for planning purposes only and must be revised after a course of action is decided upon by the city. They do not include an allowance for administration or contingency. The level of effort (and costs) will be determined in part by the requirements of the permitting agencies, and these requirements cannot be predicted completely at this time.

For cost estimating purposes, we have assumed that the city intends to develop a wellfield in one area rather than wells dispersed over a large area. This will streamline the planning, permitting, and wellhead protection efforts. It will also result in lower costs for treatment, easements, and pipelines. We assumed that 7 wells are needed to achieve the 2 mgd target yield. A lesser or greater number of wells may be required. Not included is the cost to construct a pipeline from the wellfield to the Warrenton distribution system, easement or land purchase, and treatment costs. These items will be further evaluated when more is known.

Treatment for iron and manganese, if required, may add $1.5 million to the construction cost for the project, for a 2 mgd supply. This estimate is based on a unit cost of $0.75 per gallon, but does not reflect specific design definition for this project. Treatment would also add significantly to the annual operation and maintenance costs for this supply. As described under the *Aquifer Storage and Recovery* section, a groundwater supply using ASR may eliminate the need for iron and manganese treatment.

TABLE 4-1
Budget Level Cost Estimate
Full Implementation of a Clatsop Plains Wellfield

Item	Engineering Cost	Construction or Lab Cost
Wellfield siting assistance	$5,000	
Subtotal	$5,000	
Groundwater Investigation		
Test well installation (suitable for production well)	$10,000	$60,000
Piezometers (6 wells; pump testing, level monitoring)	10,000	30,000
Water quality testing (full list of SDWA contaminants)	5,000	10,000
Surface water (lake impact, environmental) study	35,000	
Senior review and report preparation	10,000	
Subtotal	$ 70,000	$100,000
Permitting and Well Construction		
Water right permit and water management plan	$30,000	
Aquifer development plan	10,000	
Wellhead and wellfield piping design	30,000	
Construct 6 wells	40,000	$300,000
Design and construct well pump, house and wellhead piping for 7 wells (1 test well plus 6 new wells)	60,000	490,000
Subtotal	$170,000	$790,000
Iron and manganese treatment facility	$180,000	$1,500,000
Subtotal	$180,000	$1,500,000
Total	**$425,000**	**$2,390,000**

CHAPTER 5

Treatment

The water from the Lewis and Clark River and its tributaries is of generally high quality. The addition of chlorine for disinfection has been the primary form of treatment provided in the past. Lime is added to raise the pH of the water to reduce corrosion of pipe materials and home plumbing systems.

Until development of the federal Surface Water Treatment Rule in 1989, chlorination only was acceptable for meeting the microbial treatment requirements. This new rule requires filtration of nearly all surface supplies. Warrenton performed monitoring to determine if the system could remain unfiltered with enhanced disinfection, but raw water coliform levels were unacceptably high.

Therefore, the city investigated filtration alternatives in 1993 and 1994, culminating in the report titled, *Preliminary Filtration Evaluation Study* (April 1994, by CH2M HILL). The recommendation was to install slow sand filtration if proven successful by pilot testing. Under CH2M HILL's guidance, city staff conducted a year-long pilot study beginning in February 1995. The results of this study are summarized in, *Slow Sand Filtration Pilot Study Report*, May 1996. The conclusion was that slow sand filtration may be an acceptable filtration method, although the source water quality is not ideal for its application. Filter run lengths may be shorter than desired and slow sand filtration may not control disinfection by-products to the degree necessary to comply with proposed future regulations.

Subsequent to the pilot study report, CH2M HILL has provided the city with an updated cost estimate for construction of a 6.0 mgd slow sand filtration plant. This is a conceptual level estimate only. It is summarized in Table 5-1. The total project cost is $4,900,000. Factors such as final site layout, escalation in media costs, code requirements for the chlorination system, design of the operations building, permitting requirements and bidding climate will determine the actual cost. Furthermore, there is a possibility that pretreatment or other modifications to the treatment system will be needed to overcome short filter run lengths or excessive levels of disinfection by-products.

CH2M HILL also provided the city with comparison conceptual level cost estimates for two other filtration technologies, conventional rapid media filtration and membrane microfiltration. Tables 5-2 and 5-3 present these estimates. Table 5-4 lists relative advantages and disadvantages of the three technologies.

After city staff reviewed the cost estimates and discussed the relative advantages and disadvantages of the three, they tentatively decided to plan for installation of a slow sand filtration plant. This is the alternative presented in the capital improvements plan.

TABLE 5-1
Slow Sand Filter Plant Cost Estimate
Conceptual Level for 6.0 mgd

Item		Cost
Earthen basins plus liner		$1,000,000
Media (gravels, sand, underdrain)		$1,730,000
Control structures, yard piping, mechanical		$400,000
Operations building, chlorination, controls		$200,000
Modifications to settling basins		$270,000
	Subtotal	$3,600,000
Engineering and contingencies @ 35%		$1,300,000
	Total	**$4,900,000**

Notes:
1. Land cost not included.
2. Media costs based on January 1997 SSF bid for Astoria.
3. Cost for earthen basins based on site visit in January 1997.
4. Assumes three filters, each 125 feet by 200 feet bottom area.

TABLE 5-2
Conventional Filtration Plant Cost Estimate
Conceptual Level for 6.0 mgd

Item		Cost
Total Seaside plant construction cost		$3,750,000
Delete clearwell from Seaside cost		$295,000
	Subtotal	$3,455,000
Escalation for January 1997 (2%)		$69,000
	Subtotal for 4 mgd	$3,524,000
Unit cost per 1 mgd		$881,000
	Subtotal for 6 mgd	$5,286,000
Allowance for exp. to 8 mgd		$100,000
Modifications to settling basins		$270,000
	Construction total for 6 mgd	$5,390,000
Engineering, contingencies, and administrative costs @ 35%		$1,890,000
	Total	**$7,300,000**

Explanation: Cost based on recent plant construction for Seaside. Seaside installed a 4-mgd package plant that was completed in 1996. Total construction cost was $4.85M, of which $3.75M was for plant, including building, clearwell, settling basins, low lift pumping, chemical feeds, chlorination (with scrubber), and yard piping. Does not include land cost.

TABLE 5-3
Membrane Filtration Plant Cost Estimate
Conceptual Level for 6.0 mgd

Item		Cost
6-mgd microfiltration plant construction		$4,400,000
	Subtotal	$4,400,000
Allowance for expansion to 8 mgd		$100,000
Modifications to settling basins		$270,000
	Construction total for 6 mgd	$4,770,000
Engineering, contingencies, and administrative costs @ 35%		$1,670,000
	Total	**$6,400,000**

Notes:
Microfiltration plant cost includes membrane system, building, chlorination facilities, pumping, and washwater handling facilities. Land cost is not included. Based on AWWA Research Foundation study published May 1996.

TABLE 5-4
Warrenton Filtration Comparison
Relative Comparison of Three Filtration Technologies

Criteria	Slow Sand Filtration	Conventional Filtration	Microfiltration
Reliability (process control)	+	0	+
Operator time	+	-	0
Mechanical maintenance	+	-	-
Chemical handling	+	-	0
Washwater system	+	-	0
Process flexibility for future regulations	-	+	0
Process flexibility for changing water quality conditions	-	+	0
Disinfection by-products control	-	+	0
Land area	-	+	+
Note: + is favorable, 0 is neutral, - is negative			
Estimated capital cost	$4,900,000	$7,300,000	$6,400,000
Estimated annual O&M cost	$160,000	$150,000	$240,000
20-year annual cost (6%)	$590,000	$790,000	$800,000

Notes on O&M costs:
These must be considered rough estimates. They include labor, maintenance, filter sand replacement (every 8 years) for slow sand, membrane replacement for microfiltration, energy, and chemicals. However, additional evaluation is needed. The amounts are relative and depend to a large extent on the assumption for the amount of labor time.

CHAPTER 6

Storage

This chapter contains an evaluation of Warrenton's distribution storage. Storage is necessary for daily operations and system reliability. The amount of storage needed can be determined by considering its specific purposes.

> **Purposes of distribution storage**
> Equalization: storage to meet peak demands
> Fire: storage required for fire fighting
> Emergency: storage that provides a reserve for system failures

These three categories of storage needs are met by finished water storage reservoirs (tanks) located within the distribution system, near the customers. Storage tanks are not divided into separate sections for the various components, but a review of storage needs using these divisions is helpful for determining how much storage is needed.

Another sizing factor is water quality. Even when treated water meets all regulations and is aesthetically pleasing, storage of this water for an extended time can result in a deterioration of its quality. Long detention periods can impart an unpleasant taste and odor, or allow bacteriological growth. Therefore, sizing and design of storage reservoirs must also give consideration to water quality.

Chapter Summary

Warrenton's existing water system includes two storage reservoirs. One is the uncovered, 1.6 mg Clatsop Plains steel tank. The second is the 0.2 mg ground level East Harbor tank.

When compared to the size of Warrenton's water system, the present amount of storage is inadequate to provide reliable system operation. The current deficit is about 3 million gallons, and this deficit will increase as the service population grows. We recommend construction of a 3.0 mg North Coast reservoir to address this deficit. A second tank, also sized at 3.0 mg if demands grow as projected, can be added in a few years. Property and pipelines to accommodate both tanks should be set aside during the initial construction. Other improvements are also recommended, as summarized in the following list.

> **Chapter Recommendations**
> Plan for the installation of two 3.0 mg concrete North Coast reservoirs
> Set overflow of North Coast reservoirs to give gravity service to Warrenton
> Construct first North Coast tank as soon as possible to address current storage deficit
> Cover existing Clatsop Plains tank
> Abandon use of East Harbor tank
> Construct the second North Coast reservoir in about 10 years, as growth dictates

The project cost estimate for constructing one 3.0 mg reservoir is $2,160,000. The budgetary allowance for covering the Clatsop Plains tank is $200,000.

Existing Storage Facilities

Warrenton's present system includes two storage reservoirs, as summarized in Table 6-1.

The largest of the two is the Clatsop Plains reservoir, a 1.6 million-gallon (mg) ground-level, uncovered, steel tank. It is located east of Highway 101 across from Camp Rilea. The overflow elevation of this reservoir is 175 feet. It is allowed to continuously overflow to ensure turnover. Otherwise, because of its elevation and location in the system, water stagnates in this tank. The 1979 master plan estimated the overflow rate to average 150 gpm, or 0.22 mgd. The tank is relatively low in elevation compared to the normal hydraulic grade line, causing it to fill and usually remain full. It only empties when pressure in the system drops below the altitude valve set point, and this occurs only during unusually high demands. This reservoir is said to float on the system because not all water must pass through it. Its fills and empties as pressures in the transmission line rise and fall. An altitude valve on the connection line controls the overflow rate to keep it to a minimum.

The second storage reservoir is located in Warrenton on East Harbor Drive. It holds 0.225 mg. It is a ground-level steel tank used mainly for fire flows. Because it is ground level, water must be pumped from this tank to enter the surrounding pressurized system. Pumps are activated by a pressure sensor in the system. When pressures drop in the water system (as caused by high fire flows or an emergency), the pumps start automatically. The current pressure setting is approximately 50 psi. Water also tends to stagnate in this tank because pressures only infrequently drop below 50 psi. A higher pressure setting alleviates the stagnation concern, but results in higher operation and maintenance costs since the pumps are used more often.

Service Levels

Distribution systems are divided into service levels with elevation ranges selected to keep pressures within a range of approximately 40 pounds per square inch (psi) to 100 psi. Oregon Health Division (OHD) rules require that a minimum of 20 psi be maintained at all times, and customers find pressures less than 40 psi to be unsatisfactory. On the high end, pressures above about 100 psi result in more leaky pipes, especially in home plumbing systems. Using the values of 40 and 100 psi, the elevation ranges that the primary reservoir can acceptably serve are also noted in Table 6-1.

TABLE 6-1
Existing Reservoirs
Overflow elevations, volumes and service levels

Name	Volume (mg)	Overflow Elevation	High Service Elevation	Low Service Elevation
Clatsop Plains	1.6	175 feet	83 feet	0 feet
East Harbor Drive[1]	0.225	Ground Level	---	---

[1] Storage from the East Harbor Drive reservoir is pumped, and the service elevation range depends on the pump sizing.

Storage Regulations

The OHD rules for public water systems include a section on construction standards for finished water storage (Oregon Administrative Rules, 333-61-050 (7)). The significant rules for Warrenton's storage evaluation are listed below. The rules do not include requirements for the volume of storage that must be provided, other than the general requirement that the volume shall be increased for systems with hydrants.

> **Storage Regulations:**
>
> Finished water storage facilities shall have watertight roofs
>
> Finished water storage capacity shall be increased to accommodate fire flows when fire hydrants are provided
>
> Where a single inlet/outlet pipe is installed and the reservoir floats on the system, provisions shall be made to insure an adequate exchange of water and to prevent degradation of the water quality and to assure the disinfection levels required in the rules.

Warrenton's existing storage reservoirs do not comply with these rules. The Clatsop Plains tank is uncovered. OHD's construction standards would not allow an uncovered tank to be constructed anymore, although uncovered tanks that were constructed prior to adoption of the rules, such as Warrenton's, may remain in service.

Both of Warrenton's existing tanks have single inlet/outlet pipes and are said to float on the system, although the East Harbor tank has a pumped return. Therefore, both need provisions to ensure an adequate exchange of water and prevent degradation of water quality. This can be accomplished in the East Harbor tank by raising the pressure setting so that the pumps operate more frequently; however, this results in higher operation and maintenance costs.

Storage capacity is evaluated in subsequent sections of this chapter. The amount of storage in the system is insufficient to meet fire, equalization, and emergency needs.

Since the Clatsop Plains tank fills and empties by gravity, the operators can only control the rate of turnover by allowing the tank to overflow. This is generally successful, but requires daily supervision by operations staff and results in wasted water. On occasion, the waste rate becomes a significant portion of total system use, when system conditions change rapidly. As treatment and pumping costs rise and conservation methods are put into practice, the overflow method for maintaining water quality in tanks will not be acceptable.

Storage Criteria

The three purposes for storage were introduced at the beginning of this chapter. They are discussed for Warrenton's system in the following paragraphs.

Equalization Storage

The treatment plant and transmission pipeline are designed to produce water at the maximum day demand rate. The difference between this rate and peak hour use is met by the equalization portion of finished water storage. This storage quantity is used during the daytime and replenished during the nighttime.

The amount of equalization storage needed varies from system to system, depending on factors such as the proportion of industrial to residential users, and the climate. Industrial customers generally use water at relatively even rates over a 24-hour period, whereas residential customers may use water at more than twice the average rate during the daytime hours. Climate influences water use because it influences outdoor watering patterns.

In some cases, water systems have been able to carefully track use throughout a 24 hour period to determine the amount of equalization storage needed, but these data were not available for Warrenton. In the absence of such data, it is recommended that typical criteria be applied. Equalization values in the Pacific Northwest range from 18 to 30 percent of the maximum day demand. We recommend 25 percent as a reasonable value for the Warrenton system.

Fire Storage

The OHD rules stipulate that finished water storage be increased if the system includes hydrants, as Warrenton's system does. The maximum Insurance Services Office (ISO) fire flow requirement for insurance rating purposes is 3,500 gpm with a recommended three hour duration. This represents a volume of 630,000 gallons.

Emergency Storage

Sizing finished water storage for emergencies is perhaps the most subjective criteria of the three storage components. It depends on how vulnerable the water system is to failure. In Warrenton's case, the factors to consider include the source and the transmission pipeline. Warrenton's source is fed by gravity and would not be significantly affected by power failures. The reliability of Warrenton's proposed filtration plant will, however, be a factor. The relatively long transmission pipeline, eight miles from the source to the city, could be considered a liability. However, it has a history of very few major leaks or breaks, and so appears to be a reliable component.

Warrenton has an emergency 12-inch connection with the City of Seaside. This emergency connection is fed through the Gearhart water system. The service pressure provided by this connection is inadequate to serve all of Warrenton. Although emergency service from Seaside is weak, the connection does supply some emergency water into the Warrenton water system when all commercial demands are turned-off.

A commonly accepted emergency storage volume is two times average day demand. This criteria is based on an allowance of two days to restore a loss of water to the system. It is assumed that during an emergency situation, public notification could reduce water consumption to average day demand levels or less. Since Warrenton has a gravity source and some emergency source capacity through Seaside, reducing emergency storage to 1.5 days would be an appropriate criteria.

It may be acceptable to further reduce emergency storage if Warrenton develops a second source, such as the Clatsop Plains groundwater source. The amount that emergency storage should be reduced would depend on the reliability and capacity of the well fields.

Storage Needs

The use of the equalization, fire and emergency storage criteria provide a means to evaluate the volume of finished storage that is needed in Warrenton. The three components of storage are, ideally, additive. However, because of water quality deterioration that may occur with long detention times and the financial impact of constructing new reservoirs, we recommend that total storage equal the sum of the equalization and emergency storage volumes. This approach assumes that an emergency event would not occur simultaneously with a peak day and a major fire.

The storage criteria are applied to the demand projections contained in Chapter 1 to arrive at recommended storage volumes per design year. These are shown in Table 6-2. As demands grow, the storage need increases because equalization and emergency volumes are based on demands.

Using this approach, the present storage need totals 4.8 mg. If demands grow as projected in Chapter 1, the storage need will grow to 6.7 mg in year 2016.

The city's two existing reservoirs hold 1.8 mg. However, water from the small East Harbor tank must be pumped, limiting this tank's usefulness and reliability, and making it expensive to operate. For a long term strategy, we recommend replacement of the volume held in the East Harbor tank.

Following this approach, the current system storage equals 1.6 mg. Compared to the current need of 4.8 mg, this results in a deficit of 3.2 mg. The deficit is projected to grow to 5.1 mg in year 2016. This is shown in the far right column of Table 6-2.

Warrenton's 1979 water master plan recommended a new 3.5 mg reservoir. This project has not been undertaken, resulting in the current deficit.

TABLE 6-2
Storage Needs Projection

Year	Demand Projections (mgd)		Storage Needs (mg)				Storage Deficit (mg)
	Average	Maximum	Equalization	Fire*	Emergency	Total	
1994	2.3	5.1	1.3	0.63	3.5	4.8	3.2
1995	2.4	5.2	1.3	0.63	3.5	4.8	3.2
1996	2.4	5.3	1.3	0.63	3.6	4.9	3.3
1997	2.4	5.4	1.3	0.63	3.7	5.0	3.4
1998	2.5	5.4	1.4	0.63	3.7	5.1	3.5
1999	2.5	5.5	1.4	0.63	3.8	5.1	3.5
2000	2.6	5.6	1.4	0.63	3.8	5.2	3.6
2001	2.6	5.6	1.4	0.63	3.9	5.3	3.7
2002	2.6	5.7	1.4	0.63	4.0	5.4	3.8
2003	2.7	5.8	1.4	0.63	4.0	5.5	3.9
2004	2.7	5.8	1.5	0.63	4.1	5.5	3.9
2005	2.8	5.9	1.5	0.63	4.2	5.6	4.0
2006	2.8	6.0	1.5	0.63	4.2	5.7	4.1
2007	2.9	6.1	1.5	0.63	4.3	5.8	4.2
2008	2.9	6.1	1.5	0.63	4.4	5.9	4.3
2009	3.0	6.2	1.5	0.63	4.4	6.0	4.4
2010	3.0	6.3	1.6	0.63	4.5	6.1	4.5
2011	3.1	6.4	1.6	0.63	4.6	6.2	4.6
2012	3.1	6.4	1.6	0.63	4.7	6.3	4.7
2013	3.2	6.5	1.6	0.63	4.8	6.4	4.8
2014	3.2	6.6	1.6	0.63	4.8	6.5	4.9
2015	3.3	6.7	1.7	0.63	4.9	6.6	5.0
2016	3.3	6.8	1.7	0.63	5.0	6.7	5.1

Notes:
1. The average and maximum demand projections have been reduced by the target conservation values.
2. Equalization storage equals 25% of the maximum day demand.
3. Fire flow storage equals 3,500 gpm for 3 hours.
4. Emergency storage equals 1-1/2 times the average day demand.
5. Total storage need is the sum of equalization and emergency storage.
6. Storage deficit equals total storage need minus 1.6 mg, the volume of the Clatsop Plains tank.

* Fire flow is not included in storage needs total. Fire flow will come from emergency volume.

Recommended Improvements and Costs

We recommend a phased approach of adding two 3.0 mg reservoirs to address the city's current storage deficit. Both should be installed on common property and at the same overflow elevation, with piping to allow isolation of either tank. This will facilitate maintenance and operation. The overflow elevation is evaluated in Chapter 7, Distribution System. The goal is to set it at a level that provides acceptable gravity supply to Warrenton and Hammond. If possible, the design should also result in improved circulation in the Clatsop Plains tank.

The first of the two tanks should be installed as soon as planning, funding, property acquisition and design can be completed, because the city is operating with a significant deficit. Adding a 3.0 mg tank will nearly erase the current deficit. The second tank could be added in about 10 years. The growth in demands should be evaluated at that time to determine if 3.0 mg is an appropriate size for this tank.

A location to the north and east of the Clatsop Plains reservoir, in the North Coast area, has been proposed by city staff. The two new tanks will be referred to as the North Coast tanks.

We recommend prestressed concrete as the most economical long-term material of construction for reservoirs that are 2 mg and larger. We also recommend that separate inlet and outlet pipes be included to improve circulation, as well as sampling features to allow careful monitoring of water quality.

The budget level project cost estimate for the two 3.0 mg concrete tanks is $2,250,000 each. This estimate is based on a construction estimate of $1,800,000 plus an allowance of 25 percent for engineering and contingencies. The total does not include connecting pipelines or property cost. It also assumes that foundation and sitework requirements will not be unusual.

Until a conceptual plan for covering the Clatsop Plains tank is developed, it is not possible to develop a reliable budgetary estimate for this project. A place holder figure of $200,000 will be included in the capital improvements plan.

CHAPTER 7

Distribution System

This chapter contains an analysis of Warrenton's water distribution system, including transmission, for existing and future demands. Recommended improvements are listed at the end of the chapter. A map showing locations of existing distribution facilities and recommended improvements is attached to this master plan report.

This chapter is divided into the following sections:

- Chapter summary
- Description of existing system
- Modeling approach
- Regulations
- Existing (1996) system evaluation
- Future (2016) system evaluation
- Recommendations

Chapter Summary

All of Warrenton's water is supplied from surface sources located northeast of the City of Seaside. Water is delivered to Warrenton by gravity through approximately 7 miles of transmission pipes, which run parallel to Highway 101. Gearhart, Surf Pines and Camp Rilea are fed from connections to the transmission mains. The distribution system consists of 4 through 18-inch diameter pipes.

The current transmission and distribution system has several deficiencies. Water stagnates in the main distribution reservoir, the Clatsop Plains tank. Pressures in the north area of the system, near Hammond and Fort Stevens, are low during high demand periods. The system is incapable of providing sufficient fire flows to many areas, particularly the north areas of the system.

A model of the system was developed to evaluate improvements and to determine the impact of demand growth. Based on the results, we recommend the following:

- Locate the proposed new North Coast Reservoirs at a higher elevation than the Clatsop Plains Tank. Install a booster pump station to fill the North Coast Reservoirs. Together, these improvements will raise pressures throughout the Warrenton and Hammond areas, and improve fire flows.

- Add pipelines to connect the North Coast Reservoirs to the system and to improve flow to the north end of the system, as shown on the attached maps.

- Carry out a leak detection survey and pipe repair program to reduce accounted for water.

Description of Existing System

All of Warrenton's drinking water is supplied from surface water intakes located in the coast range northeast of the City of Seaside. Water is delivered to Warrenton by gravity through 18, 20, and 24-inch-diameter ductile iron transmission pipes. Approximately 11,200 feet of 24-inch diameter pipe carries water from the source to east of Gearhart; 14,500 feet of 20-inch diameter pipe carries water from the 24-inch pipe to the south end of Surf Pines; and 38,000 feet of 18-inch diameter pipe from the 20-inch pipe to the center of Warrenton. The transmission mains were installed in 1974. With the exception of a leak in the early 1990's in the 20-inch-diameter portion, no other major leaks have been observed on the transmission pipeline. The transmission mains are expected to be in good condition with many years of useful service remaining.

The intake impoundments range in elevation from 340 feet to 375 feet. The main stem of the Lewis and Clark River intake impoundment is at elevation 347.3 feet. The 17 mg storage impoundment is at an approximate elevation of 310 feet. Since these elevations result in static pressures near Gearhart of 120-150 psi, a pressure reducing valve has been installed on the 24-inch transmission line. It is currently set to limit downstream pressures to 95 psi maximum.

Because of Warrenton's flat topography, the existing system consists of only one service level. Most of the service area lies between elevation 20 and 50 feet, based on USGS contour maps. The highest land elevation currently served is approximately 65 feet near Fort Stevens Park. The highest elevation within the future service area is about 75 feet south of Warrenton near the North Coast Industrial Park areas.

Most water systems strive to provide all customers with a minimum of 40 psi water pressure under peak hour conditions and a maximum of 90 psi under static conditions. To meet these pressure conditions, the Warrenton system requires a *static* elevation head of between 167 feet and 228 feet. Higher heads may be required to maintain at least 40 psi under all flow conditions.

The existing distribution system is served by two storage tanks. One is located east of Camp Rilea and Highway 101 in the coast range foothills, approximately 2,500 feet off of the 18-inch transmission main. It is called the Clatsop Plains tank. It is a ground-level, 1.6 million gallon concrete tank with an overflow elevation of 175 feet. The Clatsop Plains tank is connected to the transmission line upstream of the major demands within Warrenton. The hydraulic grade line at the Clatsop Plains tank is above 175 feet most of the time. This means that the Clatsop Plains tank remains normally full until high demand conditions in Warrenton drop the hydraulic grade line below 175 feet near the tank. Because this high demand condition occurs infrequently, water tends to stagnate in the Clatsop Plains tank. To prevent stagnant water, Warrenton operators adjust a two-way altitude valve at the tank to allow it to overflow to maintain good water quality.

The second tank is a 225,000 gallon ground-level steel tank located on East Harbor Drive. The East Harbor tank is located at ground level within the distribution system and, therefore, all water used from this tank must be pumped. The East Harbor tank is mainly used for fire fighting storage. The East Harbor tank is equipped with two 60-horsepower 1,000 gpm fire pumps which are automatically turned on when the pressure in the system drops below 50 psi.

There are several drawbacks to the low elevation East Harbor tank. The tank is inefficient in terms of energy use because water flows by gravity into the tank but needs to be re-pumped out of the tank. The available pressure head from the gravity source is lost. Secondly, it requires more operation and maintenance by Warrenton staff than does a gravity tank. Thirdly, the turnover in the tank is low resulting in deterioration of the water quality.

Modeling Approach

The CYBERNET analysis software was used to simulate the hydraulics of the Warrenton water system. This is the software product owned by Warrenton. A computer model input file was provided by Warrenton staff for the existing distribution system. The input file contains pipes, junctions, pump, and reservoir data. Junctions are the interconnecting points of the pipeline network. The existing Warrenton system model contains approximately 55 pipes and 45 junctions. The distribution piping system in the model consists of 6- to 20-inch-diameter pipelines. Pipelines 4-inches and smaller have negligible affect on the transmission capacity of the overall distribution system.

Demand data were supplied by Warrenton staff. The distribution of demands throughout the system were reviewed and modified slightly to more closely match actual 1994 water use data. Demands for the six largest water users were distributed to the junction closest to the large demands. The remaining demands in the system were distributed evenly to the remaining junctions in the model.

The accuracy of the system model was verified by comparing simulated results with observed system pressures in Warrenton. Pressures at the Warrenton City Hall have been monitored and recorded for the past two years. They have been observed to fall below 60 psi during maximum day demands (MDD). The model results for 1996 MDD show pressures in Warrenton varying approximately between 40 to 55 psi, with a value of 44 psi at City Hall. The model results under 1996 average day demand (ADD) conditions show pressures in Warrenton varying between 70 to 75 psi, with a value of 72 psi at City Hall. The model appears to give a reasonable correlation with actual system results.

A computer input file for year 2016 was developed for Warrenton's future distribution system using projected demands and proposed improvements. Future growth was proportionally applied to the areas expecting growth. Specific areas of expected growth were obtained from Warrenton planning staff. A high percentage of growth was applied to the North Coast Industrial Park and the area south of the airport. The projected large seafood processing demands listed in Table 2-2, were distributed to the junctions closest to the large demands.

The computer model output provides an indication of how the system responds to various demands and operational conditions. The output lists the pressures and hydraulic grade lines at the junctions, velocity and friction losses through the pipes, and the operating conditions of all the facilities (e.g., pump station flows and reservoir flows).

Several conditions were modeled to ensure that the system performs acceptably. The conditions listed below were modeled for both the existing system (with current, 1996, demands) and for the future system (2016 demands with needed system improvements):

- Average day demands
- Maximum day demands
- Peak hour demands
- Reservoir refill during nighttime low demands
- MDD with fire flows

The supply source for 2016 demand was varied to reflect two possible scenarios. One was that the existing surface supply would be expanded and would be capable of meeting all demands. The second was to assume that a Clatsop Plains wellfield supply capacity of 2 mgd was developed in the Delora Beach area.

Regulations

The Oregon Health Division (OHD) has regulatory authority over public water systems in Oregon. In general, OHD's rules govern the quality of water and not the manner in which it is distributed. However, the rules do contain basic construction standards and some of these apply to distribution systems.

Significant rules for the distribution analysis are summarized below and are taken from OAR 333-61-050:

- Distribution piping shall be designed and installed so that the pressure measured at the property line of any user shall not be reduced below 20 psi.
- Wherever possible, dead ends shall be minimized by looping. Where dead ends are installed, blow-offs of adequate size shall be provided for flushing.
- Wherever possible, distribution pipelines shall be located on public property. Where pipelines are required to pass through private property, easements shall be obtained from the property owner and shall be recorded with the county clerk.
- Wherever possible, booster pumps shall take suction from reservoirs to avoid the potential for negative pressures on the suction line, which could result when the pump suction is directly connected to a distribution main. Pumps that take suction from distribution mains shall be provided with a low pressure cutoff switch on the suction side set at no less than 20 psi.

Existing (1996) System Evaluation

This section presents a summary and evaluation of the existing Warrenton distribution system under 1996 demand conditions. The 1996 Warrenton MDD is approximately 5.3 mgd (3,680 gpm). The demand projections are presented in Chapter 2, Water Requirements. A copy of the computer model input and output data for the 1996 MDD scenario is included in Appendix D.

The existing system is served by gravity from the surface source, and by gravity from the Clatsop Plains tank during high demand periods. The 18-inch pipeline scheduled to be constructed in 1997 on Main Street from SE 9th Avenue to NE 5th Street was included in the model.

Findings

1. The Clatsop Plains tank empties at about 1,080 gpm during 1996 MDD and refills under ADD conditions. Because of headloss from the Clatsop Plains tank to the major demands in Warrenton, the reservoir is unable to empty a significant amount of water unless pressures in Warrenton drop to 40 to 55 psi, such as under MDD conditions.

2. The weak areas of the distribution system are near Hammond and Fort Stevens. Pressures during 1996 MDD conditions in these areas vary between 25 to 35 psi. These areas are currently fed by an 8-inch and 10-inch grid. Under peak hour demand conditions, pressures in the Hammond and Fort Stevens areas are extremely low. The computer model shows negative pressures in this area. However, it is likely that the even distribution of unaccounted-for water throughout the system is suppressing the pressures shown in the model. We recommend monitoring of pressures in this area during high summer demand (peak hour) conditions to develop a record of actual system performance. Until a leak survey is conducted and all customers are metered, the exact locations of the unaccounted-for water will be difficult to model.

3. The system cannot provide needed fire flows to several areas. Fire flow was analyzed during 1996 MDD with the East Harbor reservoir supplying 1,000 gpm fire flow into the distribution system. The results are shown in Table 7-1.

Future (2016) System Evaluation

This section presents an evaluation of the Warrenton distribution system under year 2016 demand conditions. Projected demands are presented in Table 2-5. A copy of the computer model input and output data for the 2016 MDD scenario is included in Appendix E.

The 2016 Warrenton system was assumed to be served by a new North Coast tank located near the North Coast Industrial Park. The 18-inch pipeline grid to serve the North Coast Industrial Park and the 18-inch proposed loop to Main Avenue and East Harbor Street were included in the model.

TABLE 7-1
Existing System Fire Flow Analysis

Location; Junction	ISO Required Fire Flow[1] (gpm)	Model Results: Fire Flow (gpm)	Model Results: Residual Pressure (gpm)	Notes
S. Main and Cemetery Road (Comm./Res.); J60	3,500	1,630	40	Minimum 20 psi at Hammond, J130
SW Cedar and SW 9th (Comm./Res.); J80	3,500	1,360	38	Minimum 20 psi at Hammond, J130
Airport Road and C.G. Road (Comm.); J300	3,500	530	20	
S. Main and SW 2nd (Comm.); J90	2,250	1,320	37	Minimum 20 psi at Hammond, J130
Skipanon Drive and Harbor Place (Comm.); J110	1,500	1,250	36	Minimum 20 psi at Hammond, J130
Warrenton Drive and NW 13th (Comm.); J120	3,500	500	26	Minimum 20 psi at Hammond, J130
Heceta Place and Pacific Drive (Comm.); J130	3,500	310	20	
Pacific Drive and Lake Drive (Comm.); J140	1,750	310	20	
Russell Drive and Russell Place (Comm./Res.); J160	2,000	270	20	
Hwy. 101 and Harbor Street (Comm.); J110	2,250	1,250	36	Minimum 20 psi at Hammond, J130

[1]A maximum fire flow of 3,500 gpm was analyzed in the model. Fire flow requirements that are larger than 3,500 gpm are not used by the ISO in determining the public protection classification of a municipality.

Findings (assuming an overflow elevation of 175 feet for the North Coast tank)

1. The Clatsop Plains tank overflow elevation is 175 feet. If the new North Coast tanks also have a 175-foot overflow elevation, they will not refill under 2016 ADD conditions. (The North Coast tank will refill at about 250 gpm under 1996 ADD conditions.) The North Coast tank would empty at about 2,300 gpm under 2016 MDD conditions. The existing Clatsop Plains tank can be refilled under 2016 ADD conditions, however, under 2016 MDD conditions, the Clatsop Plains tank would not empty as the hydraulic gradeline along the transmission main, near the Clatsop Plains tank, is about 179 feet.

2. To refill the North Coast tank by gravity from the existing surface source, approximately 6.8 miles of 24-inch parallel transmission main is needed from the source to the intersection of Highway 101 and Ridge Road. With the 24-inch parallel transmission main in the model, the North Coast tank can be refilled at about 1,400 gpm under 2016 ADD condition.

3. An alternative to constructing the 6.8 miles of transmission main is to construct a booster pump station to refill the North Coast tank. With the North Coast tank located close to Warrenton's major demands, the water in the North Coast tank would provide pressure to Warrenton's system as it empties to serve demands. With a booster station

refilling the North Coast tank, the overflow elevation of the North Coast tank can be increased from 177 feet elevation without significant impact to tank refill. This gives Warrenton some leeway in locating the new North Coast tank, and impacts system performance to some extent, as described in the following section.

Findings (North Coast tank elevation of 225 feet)

1. Under 2016 peak hour demands conditions and with the North Coast tank set at 225 feet overflow, the pressures in the Warrenton system are generally between 40 to 70 psi. The weak areas of the system are near Hammond and Fort Stevens. The peak hour pressures in these areas are approximately 15 to 20 psi. To increase pressures to 40 psi and above during peak hour demands would require replacing the existing 8- and 10-inch mains with equivalent 12-inch mains from Main Avenue and NE 5th Street to Pacific Drive and Heceta Street and replacing the existing 6-inch main with an equivalent 12-inch main on School Road from Main Avenue to Ridge Road.

2. A booster pump station taking suction from the 18-inch transmission main near Ridge Road and supplying 3,500 gpm (5.1 mgd) into the Warrenton system would require approximately 130 feet of pumping head under 2016 ADD conditions. The discharge pressure at the booster pump station would be approximately 90 psi, about an 18 psi increase compared to static pressures downstream of the pump station. The North Coast tank would refill at approximately 1,800 gpm. Under 2016 ADD conditions and the booster pump station supplying 3,500 gpm, the Clatsop Plains tank would be emptying at approximately 1,500 gpm. This would help the Clatsop Plains tank to draw down to prevent stagnant water.

   The 5.1 mgd pumping rate will meet projected demands within Warrenton. However, as maximum day demands within Warrenton approach this rate, the pressure reducing valve on the transmission pipe may need to be set higher to obtain more transmission capacity. Higher flows will require a second booster pump station, located approximately half-way between the pressure reducing valve and the pump station near Ridge Road.

3. If a 2 mgd groundwater supply was developed in the Delora Beach area, the new source would likely connect into the distribution system near Ridge Road and Highway 101, downstream of the booster pump station. If a 2 mgd new source was developed and functional, the booster pump station capacity could be reduced by 2 mgd. The discharge head required by the groundwater source would be similar to the booster pump station discharge head.

4. With the addition of the new North Coast tank, fire flow is generally adequate in most areas with the exception of the extremities of the system that are served by 8-inch mains. Fire flows were analyzed under 2016 MDD condition. Table 7-2 presents the fire flow results. The areas that have low fire flow capabilities are Hammond, Fort Stevens, and the airport. With no improvements to the distribution system other than the proposed 18-inch grid in the North Coast Industrial Park to Main Avenue and East Harbor Drive, the available fire flows in the Hammond and Fort Stevens area vary between 300 to 750 gpm. The available fire flows near the airport are about 600 to 800 gpm. Improvements are noted in Table 7-2 which will provide these areas with the ISO fire flow requirements.

TABLE 7-2
Future System Fire Flow Analysis

Location; Junction	ISO Required Fire Flows[1] (gpm)	Model Results: Fire Flow (gpm)	Model Results: Residual Pressure (psi)	Improvements Needed to Meet ISO Fire Flow Requirements
S. Main and Cemetery Road (Comm./Res.); J60	3,500	3,500	50	
SW Cedar and SW 9th (Comm./Res.); J80	3,500	3,500	44	
Airport Road and C.G. Road (Comm.); J300	3,500	3,500	20	16-inch main needed from North Coast Industrial Park 18-inch grid to the airport to meet 3,500 gpm fire flow
S. Main and SW 2nd (Comm.); J90	2,250	3,500	42	
Skipanon Drive and Harbor Place (Comm.); J110	1,500	3,070	39	
Warrenton Drive and NW 13th (Comm.); J120	3,500	3,500	36	Equivalent 18-inch main needed from Main Avenue and NE 5th Street to Pacific Drive and Chinook Street to meet 3,500 gpm fire flow (J150 at 20 psi)
Heceta Place and Pacific Drive (Comm.); J130	3,500	3,500	26	Additional equivalent 18-inch main needed from Pacific Drive and Chinook Street to Pacific Drive and Heceta Place to meet 3,500 gpm fire flow (J150 at 16 psi)
Pacific Drive and Lake Drive (Comm.); J140	1,750	1,750	20	Additional equivalent 16-inch main needed from Pacific Drive and Heceta Place to Pacific Drive and Lake Drive
School Road; J140	1,750	1,750	20	Equivalent 12-inch main needed from Main Avenue to Ridge Road
Russell Drive and Russell Place (Comm./Res.); J160	2,000	2,000	20	Additional equivalent 12-inch main needed from Pacific Drive and Lake Drive to Russell Drive and Russell Place to meet 2,000 gpm fire flow
Hwy. 101 and Harbor Street (Comm.); J110	2,250	3,070	39	

[1] A maximum fire flow of 3,500 was analyzed in the model. Fire flow requirements that are larger than 3,500 gpm are evaluated individually and not used by the ISO in determining the public protection classification of a municipality.

Pipeline improvements are also needed in the Surf Pines area to increase fire flows and complete a looped distribution network. These consist of sections of 8-inch diameter pipes at both the south and north ends of Surf Pines, as shown on the attached maps.

Recommended Improvements

1. We recommend an overflow elevation of approximately 215 to 225 feet for the proposed North Coast tanks. This will raise static pressures in the Warrenton by 16 to 21 psi. The higher elevation, 225 feet, is preferable to obtain higher peak hour and fire flows and pressures in the Hammond and Fort Stevens area of the system. However, it will also

result in higher pressures in downtown Warrenton where it will exacerbate the problem of leaking pipes and higher pumping costs. The lower pressure, 215 feet, may lessen the impacts in the downtown area, but also reduces pressures and flows in the north area. We suggest that anywhere in the 215 to 225 feet elevation range is acceptable, and the deciding factor can be the topography of available reservoir sites. Unaccounted for water rates and leakage should be closely monitored when the higher system head is implemented.

2. We recommend the installation of a booster pump station to pump to the 225 feet overflow level and fill the proposed North Coast tanks. The alternative of installing a large, parallel transmission main is more costly and gives marginal results, and is therefore not recommended. For redundancy and emergency capability, the booster pump station should be sized to serve the Warrenton MDD in the event that the North Coast tank is out of service. The design should include a bypass around the pump station so that the system can operate from the existing Clatsop Plains tank if necessary. By installing a bypass, the city will not forfeit the reliability of a gravity system. The projected 2016 MDD inside Warrenton City limits is 5.1 mgd (3,500 gpm). A pumping capacity of 3,500 gpm allows the system to serve Warrenton's demand even with the North Coast tank off-line. If a groundwater source is developed at Delora Beach (in the Clatsop Plains Aquifer), the capacity of the booster pump station can be reduced by the amount of flow pumped directly from the wells into the Warrenton area. The head on the well pumps should be sized to pump to the 225 feet overflow level and fill the proposed North Coast tanks.

3. A possible location for the booster pump station would be on the 18-inch transmission main near Ridge Road. This would allow demands south of the booster pump station to be served by the existing Clatsop Plains tank. A booster pump station at this location would discharge into the existing 18-inch and 8-inch mains that run north into the Warrenton distribution system. This booster pump station and the first North Coast tank must constructed simultaneously since they will function as one system.

   The addition of a pump station will add operation and maintenance costs to Warrenton's system. Assuming an average pumping rate of 2.4 mgd (1800 gpm), the annual electricity cost at $0.06 per kilowatt-hour equals $34,000.

4. The value of the East Harbor fire flow tank is greatly reduced once the North Coast tank is installed. Remove this tank from service if maintenance costs become excessive or water quality problems result from low turnover.

5. Peak hour pressures in the Hammond and Fort Stevens area, particularly as demands grow, will remain low even after installation of the proposed North Coast Reservoir. We recommend installation of new mains from Main Avenue and NE 5th Street to Pacific Drive and Heceta Street to alleviate this problem. An equivalent 12-inch-diameter main (that is, the carrying capacity of the new main plus existing mains should be equivalent to a 12-inch-diameter pipe) will increase pressures in this area to above 40 psi. However, to enable fire flow conditions to be met, an equivalent 18-inch-diameter main is needed.

6. Other pipeline improvements are needed to obtain minimum fire flow requirements throughout the northern portion of the system. These include an equivalent 16-inch-diameter main from Pacific Drive and Heceta Place to Pacific Drive and Lake Drive, and

a 12-inch equivalent main from this point to Russell Drive and Russell Place. A 12-inch-diameter equivalent main on School Road from Main Avenue to Ridge Road is also recommended.

7. Pipeline improvements are needed in the Surf Pines area to increase fire flows and complete a looped distribution network. These consist of sections of 8-inch diameter pipes at both the south and north ends of Surf Pines, as shown on the attached maps.
8. To allow for industrial development near the airport, an 18-inch-diameter main is needed from the North Coast Industrial Park grid to the airport.
9. We recommend a leak survey to locate leaks within the distribution system. Coupled with the installation of meters on all customer accounts, this will provide the city with valuable information on unaccounted for water and actual system demands, and will help to target pipelines for replacement and repair.

Maps

The two attached maps (one for the north end and one for the south) display the recommended improvements described above. Use the maps as a guide, and not an inflexible plan for construction. The location and size of the proposed pipelines is subject to city review at the time the improvements are implemented. The locations are approximate because of the limited level of detail contained in a planning document. The sizes were determined by modeling the system using projected future demands. Larger or smaller pipelines than those shown may be needed or acceptable depending on actual water demand growth within the system.

CHAPTER 8

Improvements Plan

This chapter summarizes the improvements discussed in the preceding chapters, and presents a capital improvements plan for the water system. A list of non-capital recommendations is also included.

Capital Improvements Plan

Table 8-1 outlines a capital improvements plan for the city. The improvements are organized in the table according to recommended implementation dates. The improvements are also identified on the attached maps.

With the exception of the two supply projects (No.'s 3 and 8), the projects scheduled for 1997 through 2001 all address existing needs.

The largest single project is the installation of the slow sand filtration plant. The city has tentatively selected slow sand filtration as the treatment technology to bring the system into compliance with the Surface Water Treatment Rule of the Safe Drinking Water Act. The other large projects that address existing needs include installing a new 3 million gallon distribution storage tank, a new booster pump station, and new distribution pipelines throughout the system.

Additional pipeline projects and a second reservoir will be needed as growth occurs. Their timing may need adjustment if growth occurs more rapidly or slowly than projected. Future projects are identified in Table 8-1 by shading.

Non-Capital Recommended Improvements

Table 8-2 lists other recommendations coming out of the study that do not involve capital expenditures for the construction of facilities. These include establishing a program for accurate accounting of water use (which will be possible once meters are installed), installing stream gauging stations in the watershed, implementing conservation measures, participating in the Clatsop County planning group, and developing a water management plan.

Cost Estimating Background

Table 8-1 lists both construction and total project costs. Total project costs include a contingency to cover unknowns in construction plus engineering. The total project cost should be used for budgeting purposes. An allowance of 35 percent for contingencies and engineering was included for the filtration plant, pump station, and covering the Clatsop Plains tank projects. An allowance of 25 percent was included for the new storage tanks and pipelines. An allowance of 50 percent was included for the raw water impoundment.

Costs are representative for June 1997, at an approximate *Engineering News Record* Construction Cost Index of 6370.

The estimated costs for installing pipelines are based on the unit prices shown in Table 8-3.

TABLE 8-1

Water System Capital Improvements Plan

No.	Year	Project Description	Need for Project	----Estimated Cost---- Construction	Total	Comments
CIP-1	1997-1999	Install meters for all flat rate customers (total of 1,285)	1) Reduce waste (improve demand side conservation), 2)Improve tracking of water consumption and unaccounted for water		$156,000	Total project cost includes only purchase cost of meters, as provided by city. Does not include labor for installation.
CIP-2	1997-1998	Construct 6.0 mgd slow sand filtration plant for currently used Lewis and Clark River sources	Compliance with the Surface Water Treatment Rule of the Safe Drinking Water Act	$3,600,000	$4,900,000	Estimate based on plant cost update submitted to city in January 1997; earthen basin design
CIP-3	1997-1998	Investigate groundwater in Clatsop Plains: limited water quality monitoring and preliminary evaluation of aquifer storage and recovery potential	To determine if further evaluation of the Clatsop Plains aquifer is warranted		$15,000	For limited water quality monitoring. Additional investigations needed to confirm source.
CIP-4	1998	Install North Coast 3.0 million gallon reservoir plus 2,500 feet of 18 inch connecting pipe (Pipe No. 550) (obtain property and design for eventual addition of second tank)	1) Meet storage needs for daily peaks, fire protection and emergency, 2) Improve pressures and fire flows throughout Warrenton	$1,980,000	$2,480,000	Assumes prestressed concrete construction for reservoir; cost does not include land or unusual site work
CIP-5	1998	Install booster pump station for North Coast reservoir, 5.1 mgd firm capacity package station	1) Supply feed to North Coast reservoir, 2) Improve pressures and fire flows throughout Warrenton	$250,000	$340,000	Assumes packaged type of pump station in concrete block building
CIP-6	1999	Install cover on existing Clatsop Plains tank	To protect finished water quality from airborne contaminants and vandalism	$150,000	$200,000	Estimated without benefit of conceptual design
CIP-7	1999	Install new pipes in Surf Pines area, 16,000 feet of 8-inch	To improve fire flows and develop looped system	$700,000	$880,000	See Table 3 for pipeline unit prices
CIP-8	2000	Install new 17 mg raw water impoundment in watershed	To address projected supply deficiency	$400,000	$600,000	City staff provided cost estimate based on update of cost to install existing impoundment (1985); includes only minimal allowance for permitting, and environmental mitigation; does not include land cost

TABLE 8-1

Water System Capital Improvements Plan

No.	Year	Project Description	Need for Project	----Estimated Cost---- Construction	Total	Comments
CIP-9	2000	Install pipelines to north of city: 10,700 feet of 18 inch, 3,500 feet of 16 inch, and 2,400 feet of 12 inch (Pipes No. 120, 130, 140, and 160)	Improve fire service and peak hour pressures to Hammond and other areas north of Warrenton	$1,390,000	$1,740,000	See Table 3 for pipeline unit prices
CIP-10	2000	Install pipelines in vicinity of North Coast reservoir: 5,700 feet of 18 inch (Pipe No. 570 and 590)	Increase usefulness of new reservoir by improving supply capability into central Warrenton	$410,000	$510,000	See Table 3 for pipeline unit prices
CIP-11	2001	Pipe leak survey plus repairs	Reduce high unaccounted for water (in conjunction with metering)		$50,000	Allowance. Extent of survey and repair depends on determination of unaccounted for water rate once all customers are metered
CIP-12	2002	Install pipelines in vicinity of North Coast reservoir: 8,500 feet of 18 inch (Pipes No. 600, 620, and 640)	Improve pressures and supply to central Warrenton	$610,000	$760,000	See Table 3 for pipeline unit prices
CIP-13	2004	Install pipelines east to west on School Road, from Main Avenue to Ridge Road: 5,000 feet of 12 inch (Pipes No. 240 and 250)	Improve fire service and peak hour pressures to Hammond and other areas north of Warrenton	$300,000	$380,000	See Table 3 for pipeline unit prices
CIP-14	2005	Install pipeline to service development near airport: 4,500 feet of 16 inch (Pipe No. 580)	Needed only when development near airport occurs (cost paid by developers?)	$290,000	$360,000	See Table 3 for pipeline unit prices
CIP-15	2007	Install second North Coast 3.0 mg reservoir	Meet storage needs for daily peaks, fire, and emergency	$1,800,000	$2,250,000	Assumes prestressed concrete design for reservoir
				TOTAL	**$15,600,000**	

Note: Growth-related needs are shaded.

TABLE 8-2
Non-Capital Recommendations

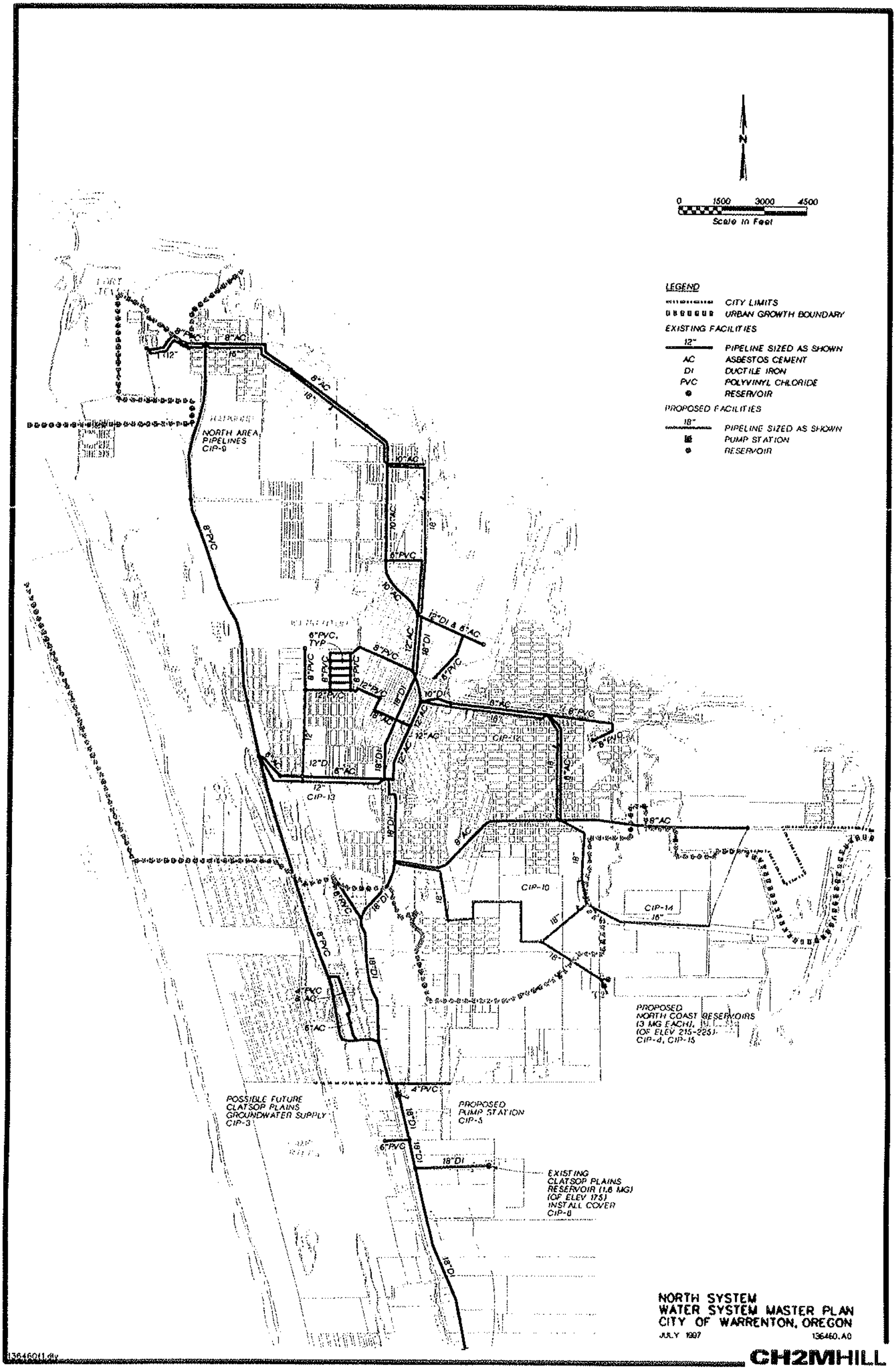
	Item Description	Need
1	Track and record monthly *consumption* figures by the following categories: inside Warrenton, outside Warrenton, and large industrial	To use in calculating unaccounted for water rate; to use in determining per capita use, particularly as conservation measures implemented; to use in establishing equitable rate structures
2	Estimate unmetered public uses of water (hydrant flushing, fire fighting, overflows, etc.) on a monthly basis	To use in calculating unaccounted for water rate
3	Record *production* on a daily basis, and compile records annually	To use in determining total system demands and unaccounted for water rates; to use in indirectly monitoring watershed yields
4	Install stream gauging on the city's four surface supplies	To develop a long-term record of available yields
5	Review water rights to determine seniority of city's surface rights. Initiate discussions with ODFW to assess validity of assuming city can capture all stream flow during drought	To identify limitations that may be placed on current supply
6	Implement conservation measures consisting of appropriate rate structures, audits of large customers, public information and education programs, and other measures	Goal is to reduce maximum day demands and average demands during the late summer and early fall period; conservation awareness is an important compliment to installing meters as the city plans to begin in 1997
7	Develop curtailment plan	Prepare plan beforehand to address a community water shortage
8	Actively participate in the Clatsop County Regional Problem Solving pilot project, particularly as related to the potential for a future Clatsop Plains groundwater supply	To have a say in land use planning that may enhance the potential for a Clatsop Plains groundwater supply
9	Consider filing for water rights for the Clatsop Plains groundwater supply	To establish the earliest possible priority date for this supply
10	Abandon use of East Harbor tank (after new North Coast tank is in service) if maintenance or water problems are significant	To reduce system O&M costs and improve water quality to customers
11	Develop a Water Management Plan along the guidelines of the Water Resources Department, to outline conservation and curtailment plans and goals	This plan may be required to obtain water rights extensions or new water rights

TABLE 8-3

Pipe Construction Cost Estimates

	Cost per Linear Foot	
Diameter (inches)	Under Asphalt Pavement	Open Ground
8	$44	$36
10	$55	$45
12	$60	$48
16	$80	$64
18	$90	$72

Assumes ductile iron pipe, with typical amounts of fittings, valves, and hydrants.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0 1500 3000 4500
Scale in Feet
LEGEND
CITY LIMITS
URBAN GROWTH BOUNDARY
EXISTING FACILITIES
12"
PIPELINE SIZED AS SHOWN
AC ASBESTOS CEMENT
DI DUCTILE IRON
PVC POLYVINYL CHLORIDE
RESERVOIR
PROPOSED FACILITIES
18"
PIPELINE SIZED AS SHOWN
PUMP STATION
RESERVOIR
NORTH AREA PIPELINES CIP-9
CIP-12
CIP-13
CIP-10
CIP-14
PROPOSED NORTH COAST RESERVOIRS (3 MG EACH) (OF ELEV 215-225) CIP-4, CIP-15
POSSIBLE FUTURE CLATSOP PLAINS GROUNDWATER SUPPLY CIP-3
PROPOSED PUMP STATION CIP-5
EXISTING CLATSOP PLAINS RESERVOIR (1.6 MG) (OF ELEV 175) INSTALL COVER CIP-8
NORTH SYSTEM
WATER SYSTEM MASTER PLAN
CITY OF WARRENTON, OREGON
JULY 1997
136460.A0
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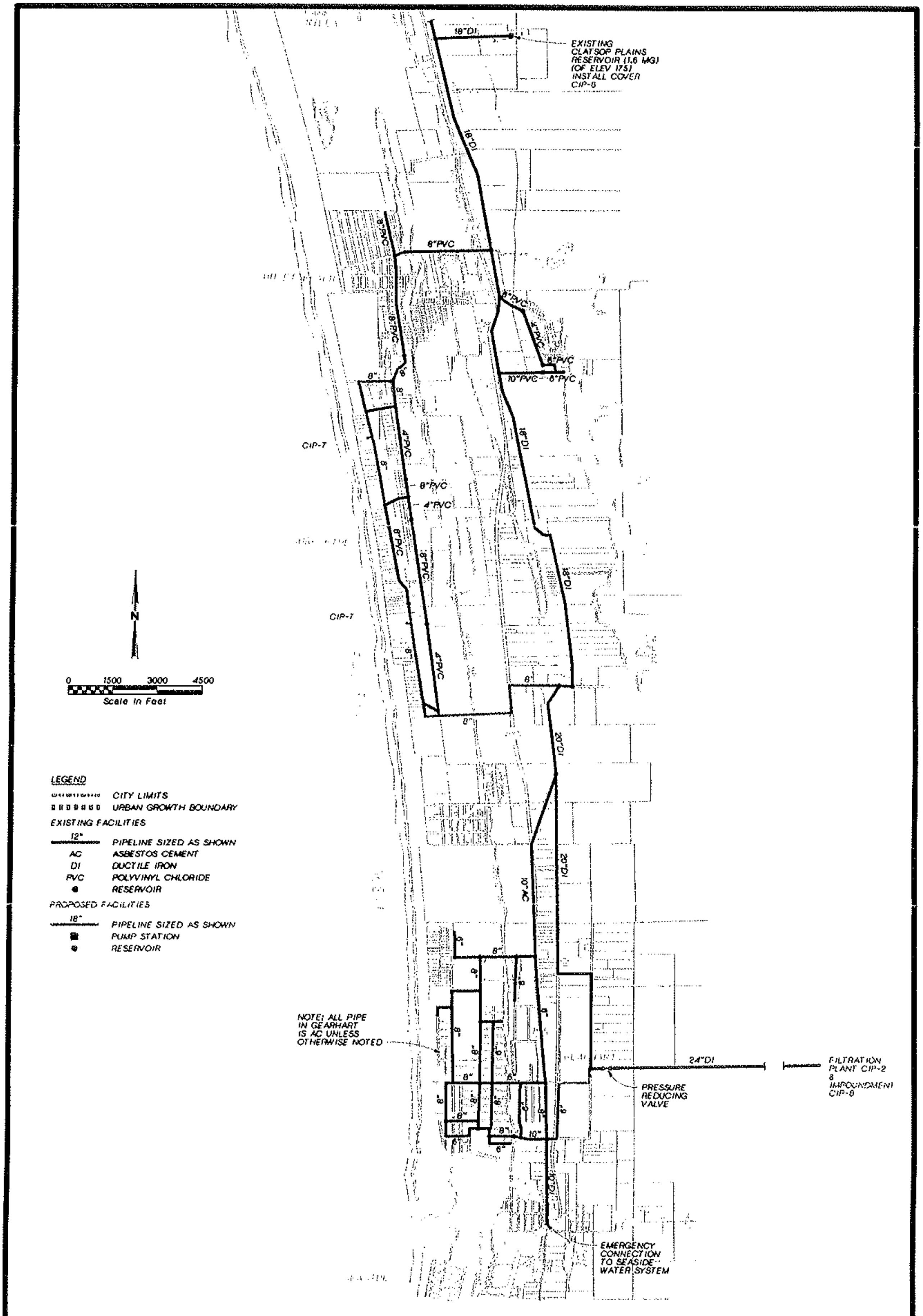
136460f1.dv



SOUTH SYSTEM
WATER SYSTEM MASTER PLAN
CITY OF WARRENTON, OREGON
JULY 1997 136460.A0

13646012.dgv

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APPENDIX A

1994 Water Use Records

VARRENTON

Y WATER REPORT

FILE = WAT-REX1

MO.	YR.	BOOK	NO SERVICE #	FLAT RATE #	FLAT RATE GAL.	NO. OF ACC. #	CONSUMPT. UNITS GAL.	ESTIMATED FLAT RATE CONSUMPTIO TOTAL BASE CHARGE $	TOTAL CONSUMPT CHARGE $		10,000 TOTAL CONSUMPT GAL	GAL/MO. TOTAL CONSUMP %	TOTAL PRODUCTION GAL.
1	94	10	8	0	0	127	4,330,100	$1,203.18	$2,870.58		4,330,100	12.53%	
1	94	20	3	0	0	144	767,000	$850.20	$1,126.50		767,000	2.22%	
1	94	25	2	0	0	198	1,953,500	$1,373.82	$1,743.29		1,953,500	5.65%	
1	94	30	3	4	0	177	1,499,800	$1,438.80	$1,787.40		1,539,800	4.46%	
1	94	35	3	25	0	102	391,000	$660.58	$562.50		641,000	1.86%	
1	94	40	33	159	0	254	606,600	$2,310.20	$561.70		2,196,600	6.36%	
1	94	45	30	186	0	285	7,295,400	$2,515.35	$3,375.70		9,155,400	26.50%	
1	94	50	34	354	0	411	547,000	$3,879.60	$508.70		4,087,000	11.83%	
1	94	55	17	96	0	122	2,996,200	$1,156.70	$1,397.13		3,956,200	11.45%	
1	94	60	27	170	0	228	377,600	$2,110.70	$298.10		2,077,600	6.01%	
1	94	65	17	103	0	124	25,100	$1,008.50	$21.10		1,055,100	3.05%	
1	94	70	3	30	0	41	479,300	$620.00	$305.20		779,300	2.26%	
1	94	80	6	0	0	34	2,014,900	$694.00	$983.74		2,014,900	5.83%	
						TOTAL =	23,283,500		TOTAL CONSUM =		34,553,500	100.00%	56,070,000
2	94	10	8	0	0	127	3,758,600	$1,208.40	$2,758.37		3,758,600	9.36%	
2	94	20	3	0	0	145	702,700	$851.22	$1,036.05		702,700	1.75%	
2	94	25	2	0	0	193	1,958,300	$1,372.20	$1,331.65		1,958,300	4.87%	
2	94	30	2	4	0	176	5,462,300	$1,452.84	$3,683.55		5,502,300	13.70%	
2	94	35	3	25	0	101	348,600	$663.58	$492.90		598,600	1.49%	
2	94	40	33	161	0	256	1,259,100	$2,320.70	$872.26		2,869,100	7.14%	
2	94	45	29	186	0	284	10,270,600	$2,513.45	$4,612.70		12,130,600	30.19%	
2	94	50	34	353	0	410	533,300	$3,857.50	$494.50		4,063,300	10.11%	
2	94	55	17	96	0	122	3,034,000	$1,164.70	$1,396.64		3,994,000	9.94%	
2	94	60	26	172	0	227	348,300	$2,228.30	$272.00		2,068,300	5.15%	
2	94	65	16	105	0	125	29,900	$1,023.10	$21.90		1,079,900	2.69%	
2	94	70	2	31	0	41	287,800	$630.00	$229.04		597,800	1.49%	
2	94	80	7	0	0	35	851,600	$694.00	$230.54		851,600	2.12%	
						TOTAL =	28,845,100		TOTAL CONSUM =		40,175,100	100.00%	49,520,000
3	94	10	8	0	0	128	4,891,500	$1,210.44	$3,072.50		4,891,500	13.75%	
3	94	20	3	0	0	148	2,688,300	$855.54	$1,011.30		2,688,300	7.55%	
3	94	25	2	0	0	198	642,800	$1,375.92	$772.20		642,800	1.81%	
3	94	30	2	4	0	176	2,101,500	$1,448.58	$2,103.81		2,141,500	6.02%	
3	94	35	3	25	0	101	362,500	$666.40	$507.75		612,500	1.72%	
3	94	40	34	162	0	258	602,200	$2,340.40	$516.67		2,222,200	6.24%	
3	94	45	29	191	0	291	8,544,900	$2,523.57	$3,956.72		10,454,900	29.38%	
3	94	50	38	354	0	413	526,500	$3,892.50	$483.15		4,066,500	11.43%	
3	94	55	17	96	0	122	1,995,300	$1,158.00	$946.02		2,955,300	8.30%	
3	94	60	27	172	0	228	364,600	$2,140.30	$292.30		2,084,600	5.86%	
3	94	65	17	108	0	129	36,700	$1,053.50	$32.70		1,116,700	3.14%	
3	94	70	2	32	0	42	570,600	$639.00	$345.18		890,600	2.50%	
3	94	80	7	0	0	35	819,200	$694.00	$211.26		819,200	2.30%	
						TOTAL =	24,146,600		TOTAL CONSUM =		35,586,600	100.00%	55,780,000
4	94	10	8	0	0	128	5,375,200	1,271	$3,525.91		5,375,200	16.50%	
4	94	20	3	0	0	146	712,400	878	$1,110.53		712,400	2.19%	
4	94	25	2	0	0	193	726,500	1,432	$975.46		726,500	2.23%	
4	94	30	2	4	0	177	1,412,550	$1,520.84	$1,820.58		1,452,550	4.46%	
4	94	35	4	25	0	103	369,300	$698.53	$561.99		619,300	1.90%	
4	94	40	34	165	0	261	589,200	$2,425.10	$545.05		2,239,200	6.87%	
4	94	45	30	186	0	286	7,850,700	$2,654.17	$3,832.81		9,710,700	29.81%	
4	94	50	35	357	14,000	414	551,000	$4,071.17	$522.31		4,107,000	12.61%	
4	94	55	18	95	0	122	1,606,500	$1,207.10	$830.64		2,556,500	7.85%	
4	94	60	27	169	0	225	446,900	$2,227.70	$398.21		2,136,900	6.56%	
4	94	65	17	110	0	131	41,300	$1,092.18	$39.61		1,141,300	3.50%	
4	94	70	2	31	0	41	572,500	$661.20	$339.63		882,500	2.71%	
4	94	80	7	0	0	36	914,100	$727.39	$307.43		914,100	2.81%	
						TOTAL =	21,168,150		TOTAL CONSUM =		32,574,150	100.00%	55,380,000
5	94	10	8	0	0	127	6,838,300	$1,270.35	$4,474.59		6,836,300	19.17%	
5	94	20	3	0	0	144	875,900	$892.65	$1,373.84		875,900	2.46%	
5	94	25	2	0	0	194	936,900	$1,435.35	$1,341.85		936,900	2.63%	
5	94	30	2	4	0	175	2,473,250	$1,526.07	$2,608.99		2,513,250	7.05%	
5	94	35	3	27	0	104	471,200	$737.40	$724.05		741,200	2.08%	
5	94	40	35	159	0	256	648,700	$2,419.54	$616.71		2,238,700	6.28%	
5	94	45	28	187	0	285	9,556,500	$2,705.74	$4,665.95		11,426,500	32.04%	
5	94	50	39	355	0	416	590,700	$4,075.76	$567.48		4,140,700	11.61%	
5	94	60	27	170	0	226	908,800	$2,238.20	$655.95		2,608,800	7.31%	
5	94	65	19	108	0	131	28,200	$1,102.99	$25.69		1,108,200	3.11%	
5	94	70	2	31	0	41	846,700	$661.20	$534.72		1,156,700	3.24%	
5	94	80	7	0	0	35	1,083,900	$725.88	$406.35		1,083,900	3.04%	
						TOTAL =	25,257,050		TOTAL CONSUM =		35,667,050	100.00%	71,460,000

APPENDIX B

Population Projections

VARRENTON
.Y WATER REPORT

FILE =WAT-REX1

ESTIMATED FLAT RATE CONSUMPTIO 16,000 GAL/MO.

MO.	YR.	BOOK	NO SERVICE #	FLAT RATE #	FLAT RATE GAL	NO. OF ACC. #	CONSUMPT. UNITS GAL.	TOTAL BASE CHARGE $	TOTAL CONSUMPT CHARGE $		TOTAL CONSUMPT GAL	TOTAL CONSUMP %	TOTAL PRODUCTION GAL.
10	94	10	7	0	0	131	12,188,400	$1,153.19	$7,217.90		12,188,400	21.83%	
10	94	20	3	0	0	147	1,223,700	$901.16	$1,835.35		1,223,700	2.19%	
10	94	25	2	0	0	199	1,410,100	$1,445.18	$2,098.25		1,410,100	2.53%	
10	94	30	2	4	0	180	2,987,600	$1,539.49	$3,066.60		3,027,600	5.42%	
10	94	35	3	28	0	104	550,200	$762.60	$868.51		830,200	1.49%	
10	94	40	35	161	0	260	1,314,700	$2,409.85	$914.08		2,924,700	5.24%	
10	94	45	28	185	0	283	7,053,000	$2,698.04	$3,597.65		8,903,000	15.95%	
10	94	50	39	361	0	422	938,600	$4,152.95	$728.05		4,546,600	8.14%	
10	94	55	17	98	0	122	1,853,000	$1,228.10	$960.77		2,813,000	5.04%	
10	94	60	26	171	0	227	10,933,000	$2,260.68	$4,888.89		12,643,000	22.65%	
10	94	65	16	117	0	137	23,200	$1,216.70	$24.59		1,193,200	2.14%	
10	94	70	4	31	0	43	2,391,300	$681.20	$1,269.56		2,701,300	4.84%	
10	94	80	7	0	0	35	1,424,300	$728.10	$685.37		1,424,300	2.55%	
						TOTAL =	44,289,100		TOTAL CONSUM =		55,829,100	100.00%	70,740,000
11	94	10	7	0	0	129	9,466,500	$1,159.05	$5,718.72		9,466,500	14.26%	
11	94	20	3	0	0	145	1,025,000	$903.30	$1,565.47		1,025,000	1.54%	
11	94	25	2	0	0	198	1,162,200	$1,457.40	$1,713.63		1,162,200	1.75%	
11	94	30	2	5	0	179	3,116,100	$1,520.03	$3,153.14		3,168,100	4.77%	
11	94	35	3	28	0	106	8,900,500	$757.37	$6,026.61		9,180,500	13.83%	
11	94	40	36	156	0	258	802,400	$2,409.10	$712.87		2,362,400	3.56%	
11	94	45	29	186	0	284	7,205,000	$2,711.30	$3,569.12		9,065,000	13.66%	
11	94	50	38	360	0	420	1,678,700	$4,173.41	$1,022.74		5,278,700	7.95%	
11	94	55	18	96	0	124	1,734,700	$1,246.53	$904.07		2,694,700	4.06%	
11	94	60	26	171	0	228	7,744,600	$2,292.80	$3,459.08		9,454,600	14.25%	
11	94	65	19	114	0	137	9,207,600	$1,185.20	$32.70		10,347,600	15.59%	
11	94	70	3	34	0	45	1,565,500	$688.55	$899.49		1,905,500	2.87%	
11	94	80	7	0	0	37	1,258,400	$734.27	$456.79		1,258,400	1.90%	
						TOTAL =	54,867,200		TOTAL CONSUM =		66,367,200	100.00%	58,470,000
12	94	10	7	0	0	130	5,766,500	$1,156.09	$3,661.60		5,786,500	14.87%	
12	94	20	3	0	0	148	721,500	$899.96	$1,123.86		721,500	1.85%	
12	94	25	2	0	0	201	1,730,600	$1,458.47	$1,702.82		1,730,600	4.45%	
12	94	30	2	5	0	178	1,522,400	$1,532.00	$2,018.48		1,572,400	4.04%	
12	94	35	3	28	0	104	425,400	$762.60	$663.80		705,400	1.81%	
12	94	40	35	158	0	259	555,500	$2,429.63	$578.39		2,135,500	5.49%	
12	94	45	28	189	0	288	6,138,400	$2,718.65	$3,126.66		8,028,400	20.63%	
12	94	50	37	363	0	422	661,200	$4,207.88	$605.32		4,291,200	11.03%	
12	94	55	20	97	0	127	1,893,500	$1,248.20	$1,003.48		2,863,500	7.36%	
12	94	60	27	171	0	230	4,808,600	$2,278.90	$2,240.73		6,518,600	16.75%	
12	94	65	19	113	1,400	136	34,200	$1,195.70	$34.78		1,162,800	2.99%	
12	94	70	3	32	0	43	2,274,400	$671.70	$973.97		2,594,400	6.67%	
12	94	80	8	0	0	38	798,600	$731.17	$350.83		798,600	2.05%	
						TOTAL =	27,350,800		TOTAL CONSUM =		38,909,400	100.00%	71,390,000
									TOTAL YEAR CONSUM. =		569,025,980		843,660,000

CITY OF WARRENTON
Warrenton, Oregon 97146-0250
P.O. Box 250 • 503/861-2233
FAX: 503/861-2351

October 15, 1996

Paul Berg
CH2M Hill
2300 NW Walnut Blvd
Corvallis, OR 97330

Dear Mr. Berg:

Attached is information regarding population projections that you requested from the City of Warrenton.

The City adopted the 2.95% compounded or 4% uncompounded growth rate for the 20 year planning horizon.

I hope this information is adequate for your needs. If you find that it is not, please do not hesitate to contact me at (503) 861-0920, and I will make every effort to provide you with additional information.

Very truly yours,

CITY OF WARRENTON

Janet Wright

Janet Wright
Planner

janet\berg.ltr

10/17

CLATSOP COUNTY POPULATION PROJECTION

APRIL, 1995

	1990 Census	1994 (PSU)	2000	2005	2010	2015
ASTORIA	10,069	10,050	11,036	11,598	12,190	12,872
CANNON BEACH	1,221	1,330	1,403	1,486	1,569	1,649
GEARHART	1,027	1,125	1,194	1,255	1,319	1,386
SEASIDE	5,359	5,655	6,368	7,031	7,763	8,571
WARRENTON (+ HAMMOND)	3,270	3,820	4,738	5,503	6,268	7,033
TOTAL CITIES	20,946	21,980	24,739	26,873	29,109	31,511
UNINCORP. CLATSOP CO.	12,355	11,920	12,653	13,299	13,977	14,690
TOTAL COUNTY	33,301	33,900	37,392	40,172	43,086	46,201

1994 → 2015 : 1.0 %

→ 2015 : 2.75 %

combined 2.57 % compounded

Post-it™ brand fax transmittal memo 7671 | # of pages ▸ 2

To Janet W	From MIKE M.
Co.	Co.
Dept.	Phone # 436 1061

NOTES ON POPULATION PROJECTIONS

1. Astoria's population for 1995 is estimated at 10,500 with the addition of 180 US Coast Guard housing units. 1995 - 2015 population projection is at 1% per year.

2. Cannon Beach population is from Ordinance 94-14, "Cannon Beach Background Report Population, Housing and Land Use Sections." 2010 - 2015 is estimated at approximately 1% per year.

3. Gearhart population is estimated at 1% growth per year.

4. Seaside population is estimated at 2% growth per year.

5. Warrenton population is taken from March 2, 1995 memorandum from Janet Wright, Planner, to Technical Advisory Committee, entitled "4% population growth justification". Projection is 4% per year not compounded, or 2.9% compounded. All other projections are compounded.

6. Unincorporated Clatsop County population is projected at 1% growth per year.

CITY OF WARRENTON
Warrenton, Oregon 97146-0250
P.O. Box 250 • 503/861-2233
FAX: 503/861-2351

April 25, 1995

Oregon Department of Land Conservation & Development
Attn: Dale Jordan, Field Representative
1175 Court Street, NE
Salem, OR 97310

RE: City of Warrenton Population Projections

Dear Dale:

In response to the input provided by Anna Russo, and yourself I have been working with Clatsop County and other Municipalities in the North Coast area regarding population projections for the City of Warrenton.

Mike Morgan, who is under contract with the County to work on the population projections provided me with the County Population projections. I have enclosed them with this letter. I provided copies of the population figures, and contacted the City's of Seaside, Cannon Beach, and Astoria. They have indicated that they do not see a problem with the Warrenton's project growth of 2.95% compounded or 4% uncompounded figure. JD White & Company will be using these population projections for the Wetland Conservation Plan.

Please let me know if you need any further information regarding the population projections for Warrenton. I can be contacted at (503) 861-2233.

Very truly yours,

CITY OF WARRENTON

Janet Wright, Planner

jw\jgw
Janet\popltr.dlc
cc: Anna Russo
Arthur Sherman

CITY OF WARRENTON
Warrenton, Oregon 97146-0250
P.O. Box 250 • 503/861-2233
FAX: 503/861-2351

July 10, 1995

Clatsop County
Community Planning & Development
Attn: Curt Schneider, Director
P.O. Box 179
Astoria, Or 97138

RE: Population Growth Justification

Dear Mr. Schneider:

I have reviewed the projected population figures for Clatsop County complied by Mike Morgan. As you know from discussion that have taken place over the last several months Warrenton has been working on its reviewing and updating population projections and buildable lands inventories to the year 2015 for the Wetland Conservation Planning process. Our population projection was calculated to be a 4% uncompounded growth rate or a 2.95% compounded rate. I reviewed these projections with yourself and Mike prior to using them as a basis in our buildable lands data.

After a review of the population projections that you sent, I see that the projection for Warrenton is being revised down to a 2% per year compounded rate. I am asking that you re-evaluate Warrenton's population projection and revise it up to the 2.95% compounded rate that we originally agreed upon. I am providing you with the following background and justification for our population projections.

Mary Dorman, of Dorman and Associates, provided several options regarding population growth in Warrenton. Ms. Dorman also conducted an analysis using the Oregon Department of Transportation (ODOT) projections and Portland State University (PSU) projections. Ultimatly the City felt that none of the projections provided by ODOT or PSU provided a realistic picture of the growth that is, and has been, taking place in Warrenton.

As a result of her review, Ms. Dorman provided the City with population projections which consisted of a low, medium and high growth rate. The projections start from a 1994 base population of 3,820, and are extended 21 years to 2015. The low projection results in a population of 5,416, (2% growth rate), or a net addition of 76 people/year; the moderate projection is 6,235, (3% growth rate) or a net addition of 115 people/year; the high growth projection provided a total of 7,033, (4% growth rate uncompounded, 2.95% compounded), with a net addition of 153 people/year. These figures were reviewed by the Planning Commission and the Warrenton City Commission and the 2.95% compounded rate was adopted.

Warrenton has been growing at a faster rate then anywhere in Clatsop County over the past two years, and it is expected that this growth will continue. The primary reason for this expectation is that Warrenton is the regional economic center of Clatsop County. An abundance of industrial land and tourism opportunities place Warrenton in a unique position to experience greater than average industrial and commercial growth in the projected time-frame.

The City believes that the 2.95% compounded growth rate provides a more realistic picture of the future, for the following reasons:

- Warrenton is seen as a regional industrial and commercial center in Clatsop County.

- There is high demand for commercial land that is not present in other North Coast Communities. Development as a result of the high demand will increase the population and require housing and services close to employment sites.

- There is also a large inventory of ready-to-build industrial sites that would draw additional population to Warrenton.

- Historically Warrenton has not focused on tourism as its main economic base. The City has aimed at diversifying its economic base, to draw people who need to be in close proximity to their jobs.

- Warrenton is building a new Tourist Information Center, which will bring visitors to Warrenton. We anticipate that our population will increase because of new employment opportunities in tourist related industries.

- Warrenton's focus on natural resource based industries provides unique economic development opportunities, which is not being aggressively pursued by other North Coast Communities.

- Warrenton foresees the ability to capture a significant portion of timber processing that is soon to be available from the regenerated Tillamook Burn.

- Astoria is beginning construction of the Sea Food Lab which will conduct research into the use of seafood. We anticipate a spin-off in facilities and therefore an increase in jobs and population in Warrenton because of the Sea Food Lab.

- Warrenton is experiencing significant population growth. In 1993 and 1994 multi-family construction consisted of 81 new apartment units; 90 new single family residences; and a 32 unit manufactured dwelling park. In the first three months of 1995 a 21 lot and a 19 lot subdivision; and a 9 unit townhouse project have been approved. A proposed 77 unit apartment complex has just been granted a street vacation request.

- Fort Stevens State Park and Fort Clatsop National Memorial provide a significant increase in population in Warrenton that is seasonal. Fort Stevens State Park is the largest state parking in Oregon; with approximately 1 million visitors each year. Fort Clatsop is a National Memorial to the Lewis and Clark Expedition which draws approximately 100,000 visitors each year. Fort Clatsop has plans to expand the memorial, which will increase Warrenton's population, create jobs and housing needs.

- Warrenton's tax rate is lower because the assessed valuation of the City has increased faster than the 6% increase allowed by State Law. Because the assessment per $1,000 valuation is lower, it provides an attractive climate for development.

- Recently Warrenton began targeting Eco-tourism through the completion of the Waterfront Revitalization Plan. Warrenton is now working on expanding a trails and bike path system along the Columbia River which will connect Warrenton to Fort Stevens and Fort Clatsop. Expansion of services to this area will create jobs and housing demands.

- The City has had numerous discussions with developers regarding the proposed Factory Outlet Mall near Fred Meyer and Costco. This will increase the population in Warrenton and create a demand for housing.

- There have been grant applications filed with the State which will bring water and sewer to the Alumax site. A recent zone change off of Dolphin Avenue could provide additional impetus for development in this area.

As you can see Warrenton has put a lot of thought into the population projections that were previously proposed and reviewed by your office. While we agree that there can be down-swings in the economic future and that one or two years of growth does not mean that we will have consistent growth over the 20 year period, we also believe that we can meet the 2.95% compounded population projection. We would appreciate your consideration in revising the population projections to meet our goals.

If you have any questions please do not hesitate to contact me at 861-0920, I would be happy to discuss them with you.

Sincerely,

Janet Wright

Janet Wright, Planner

wetlands\popjus.cty

DRAFT

RESIDENTIAL LAND NEED PROJECTIONS
(4% Population Growth)

Population estimate 1994	3,820 People
Population projection 2015	7,033 People
Projected growth	3,213 People
Estimated household size	2.64 People
Dwelling units to accommodate growth	1,217 units
Factor for vacancy, demolition & second homes (5%)	61 units
Total dwelling units needed to year 2015	1,278 units
Housing mix: single family/multi-family	1,022 SF
80% SF/20% MF	256 MF
Housing mix: single family/mobile home	716 SF
70% SF/30% MH	307 MH
Residential land requirements:	
single family @ 4 du/AC	179 Acres
mobile home @ 4du/AC	77 Acres
multi-family @ 16 du/AC	16 Acres
Total land need for housing	272 Acres
Assumed land requirements for roads, utility corridors, etc.	
single family @ 20%	36 Acres
mobile home @ 15%	12 Acres
multi-family @ 10%	2 Acres
Total land need for residential infrastructure	49 Acres
Total land need for housing & infrastructure	320 Acres
single family & mobile home	303 Acres
multi-family	18 Acres

Source: Mary Dorman & Associates & The JD White Co., Inc.
20-Mar-95
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APPENDIX C

1996 ISO Fire Flow Survey

CRS®
ISO Commercial Risk Services, Inc. • 3000 Executive Parkway • Suite 510 • P.O. Box 5126
San Ramon, CA 94583-2300 • (510) 830-8778 • FAX: (510) 830-4691

March 15, 1996

Bob Fackler, Chairman, B.O.D.
Warrenton R.F.P.D.
P. O. Box 250
Warrenton, OR 97146

Dear Mr. Fackler:

We wish to thank you and many others involved for the cooperation given to our representative during our recent survey. We have completed our evaluation of the fire insurance classification for the District that resulted from the combination of Warrenton and Hammond with Warrenton R.F.P.D. The protection class for the combined district is now Class 5.

Formerly Warrenton was Class 6, while Hammond and the R.F.P.D. were Class 5. The new classification may have a favorable effect in the property insurance premium calculations for many insured properties within Warrenton R.F.P.D. depending upon which area they are located within. The new classification will be effective May 1, 1996.

The purpose of our visit was to gather information needed to determine a fire insurance classification which may be used in the calculations of property insurance premiums. This survey was not conducted for property loss prevention or life safety purposes and no life safety or property loss prevention recommendations will be made.

The new classification willaffect typical mercantile properties to a degree depending upon the type of building construction, occupancy and other property insurance premium calculations. Property owners within the district whould consult with their insurance companies to determine the effect of the change.

This applies only for insurance companies using ISO property insurance premium calculations. However, numerous insurance companies use other than ISO property insurance premium calculations so that the effect of the change in class may be different for their policy holders.

The district classification applies to properties with a needed fire flow of 3500 gpm or less. The private and public protection at properties with larger needed fire flows are individually evaluated, and may vary from the district classification.

We are attaching a copy of our Grading Sheet and the results of the hydrant flow tests witnessed during our survey. Extra copies of this letter and attachments are also enclosed so that you may distribute them to other interested parties, if you desire to do so.

If you have any questions concerning the new classification, or the resulting change in property insurance premium calculations, please let us know.

Very truly yours,

Harry J. Alcantara
Manager, Public Protection

cc: Duane Mullins, Fire Chief
enclosures
jkr

CLASSIFICATION DETAILS

Municipality:	WARRENTON FD	State:	OR
Population:	3285	Date Surveyed:	MAR. 1995
Total Credit:	58.15	Class:	5

SUMMARY OF CREDIT

Feature	Credit Assigned	Maximum Credit
Receiving and Handling Fire Alarms..............	6.7	10.00
Fire Department..................................	27.84	50.00
Water Supply.....................................	24.95	40.00
*Divergence......................................	1.34	
Total Credit-	58.15	100.00

The Public Protection Classification is based on the total percentage credit as follows:

Class	Percentage Credited
1	90.00 or more
2	80.00 to 89.99
3	70.00 to 79.99
4	60.00 to 69.99
5	50.00 to 59.99
6	40.00 to 49.99
7	30.00 to 39.99
8	20.00 to 29.99
9	10.00 to 19.99
10	0 to 9.99

*Divergence is a reduction in credit to reflect a difference in the relative credits for Fire Department and Water Supply.

The above classification has been developed for use in property insurance premium calculations.

CLASSIFICATION DETAILS (continued)

WATER SUPPLY

his section of the Fire Suppression Rating Schedule reviews the water supply system that is available for fire suppression in the municipality.

	CREDIT	
	ACTUAL	MAXIMUM
1. (Item 616) Credit for the Water System This item reviews the supply works, the main capacity and the hydrant distribution.	21.28	35.00
2. (Item 621) Credit for Hydrants This item reviews the type of hydrants and the method of installation.	1.76	2.00
3. (Item 631) Credit for Inspection and Condition of Hydrants This item reviews the frequency of inspections of hydrants and their conditions.	1.91	3.00
4. (Item 640) Total Credit for Water Supply	24.95	40.00
Relative Classification for Water Supply	4	

CLASSIFICATION DETAILS (continued)

FIRE DEPARTMENT

This section of the Fire Suppression Rating Schedule reviews the engine, ladder and service companies, equipment carried, response to fires, training and available fire fighters.

	CREDIT ACTUAL	CREDIT MAXIMUM
1. (Item 513) Credit for Engine Companies This item reviews the number of engine companies and the hose and equipment carried.	9.91	10.00
2. (Item 523) Credit for Reserve Pumpers This item reviews the number of reserve pumpers and the equipment carried on each.	0.92	1.00
3. (Item 532) Credit for Pump Capacity This item reviews the total available pump capacity.	5.00	5.00
4. (Item 549) Credit for Ladder Service This item reviews the number of ladder and service companies and the equipment carried.	1.34	5.00
5. (Item 553) Credit for Reserve Ladder Service This item reviews the number of reserve ladder and service trucks, and the equipment carried.	0.12	1.00
6. (Item 561) Credit for Distribution This item reviews the percent of the built-upon area of the city which has a first-due engine company within 1 1/2 miles and a ladder service company within 2 1/2 miles.	2.80	4.00
7. (Item 571) Credit for Company Personnel This item reviews the average number of equivalent fire fighters and company officers on duty with existing companies.	3.52	15.00+
8. (Item 581) Credit for Training This item reviews the training facilities and their use.	4.23	9.00
9. (Item 590) Total Credit for Fire Department	27.84	50.00+

Relative Classification for Fire Department 5

+ This indicates that credit for manning is open-ended, with no maximum credit for this item.

ISO COMMERCIA SK SERVICES, INC.

HYDRANT FLOW DATA SUMMARY

City WARRENTON State OR Zip 97146 Witnessed by J.S.M. for ISO Date 3/21/95

TEST NO.	TYPE DIST.*	TEST LOCATION	SERVICE	FLOW-GPM INDIVIDUAL HYDRANTS			FLOW-GPM TOTAL	PRESSURE PSI STATIC	PRESSURE PSI RESID.	FLOW AT 20 PSI NEEDED **	FLOW AT 20 PSI AVAIL.	REMARKS
1	COMM/RES	S. Main & Cemetery Rd	MAIN		630	630	1260	86	67	3500	2500	
2	COMM/RES	SW Cedar & SW 9th	MAIN		440	470	910	88	62	7000 1000	1500	
3	COMM	Airport Rd & C.G. Rd	MAIN		340	380	720	78	20	6000 5000 3000	700	
4	COMM	S. Main & SW 2	MAIN		530	500	1030	90	75	2250	2400	ok
5	COMM	Skipanon Dr & Harbor Pl.	MAIN				750	92	58	1500	1100	2/23/82
6	COMM	Warrenton Dr. & NW 13	MAIN		650	630	1280	90	78	4500 1500	3300	
7	COMM	Heceta Pl. & Pacific Dr	MAIN		380	340	720	88	55	5000 3500	1100	
8	COMM	Pacific Dr. & Lake Dr	MAIN		340	340	680	90	51	1750	950	
9	COMM/RES	Russell Dr & Russell Pl.	MAIN		380	360	740	74	24	2000	750	
10	COMM	Hwy 101 & Harbor St.	MAIN		770	770	1540	85	71	2250	3500	ok

THE ABOVE LISTED NEEDED FIRE FLOWS ARE FOR PROPERTY INSURANCE PREMIUM CALCULATIONS ONLY AND ARE *NOT* INTENDED TO PREDICT THE MAXIMUM AMOUNT OF WATER REQUIRED FOR A LARGE SCALE FIRE CONDITION. THE AVAILABLE FLOWS ONLY INDICATE THE CONDITIONS THAT EXISTED AT THE TIME AND AT THE LOCATION WHERE TESTS WERE WITNESSED.

* Comm = Commercial; Res = Residential

** Needed is the rate of flow for a specific duration for a full credit condition. Needed Fire Flows greater than 3,500 gpm are not considered in determining the classification of the city when using the Fire Suppression Rating Schedule.

07/91 page 36

APPENDIX D

1996 Computer Model

1996 MDD
OUTPUT FILE

```
+----------------------------------------------------+
|                MAXIMUM DIMENSIONS                  |
|                                                    |
|   Number of pipes ..................... 1000       |
|   Number of pumps .....................  250       |
|   Number junction nodes................ 1000       |
|   Flow meters .........................  250       |
|   Boundary nodes ......................  100       |
|   Variable storage tanks ..............  250       |
|   Pressure switches ...................  250       |
|   Regulating Valves....................  250       |
|   Items for limited output ............ 1000       |
|   limit for non-consecutive numbering ..10260      |
+----------------------------------------------------+
```

Cybernet version 2.18. SN: 1132183902-1000

Extended Description:

U N I T S   S P E C I F I E D

```
FLOWRATE ............ = gallons/minute
HEAD (HGL) .......... = feet
PRESSURE ............ = psig
```

O U T P U T   O P T I O N   D A T A

OUTPUT SELECTION: ALL RESULTS ARE INCLUDED IN THE TABULATED OUTPUT

S Y S T E M   C O N F I G U R A T I O N

```
NUMBER OF PIPES ....................(p) =   49
NUMBER OF JUNCTION NODES ...........(j) =   41
NUMBER OF PRIMARY LOOPS ............(l) =    7
NUMBER OF BOUNDARY NODES ...........(f) =    2
NUMBER OF SUPPLY ZONES .............(z) =    1
```

```
*************************************
S I M U L A T I O N   R E S U L T S
*************************************
```

The results are obtained after 4 trials with an accuracy = 0.00364

S I M U L A T I O N   D E S C R I P T I O N

CyberNet Version 2.18. Copyright 1991,92 Haestad Methods Inc.
Run Description: 1996 MDD system w/18" on Main St completed
Drawing: WAR1996

P I P E L I N E   R E S U L T S

ATUS CODE: XX -CLOSED PIPE   BN -BOUNDARY NODE   PU -PUMP LINE
CV -CHECK VALVE   RV -REGULATING VALVE   TK -STORAGE TANK

PIPE NUMBER	NODE NOS. #1	#2	FLOWRATE (gpm)	HEAD LOSS (ft)	PUMP HEAD (ft)	MINOR LOSS (ft)	LINE VELO. (ft/s)	HL/ 1000 (ft/ft)
10-BN	0	360	2556.36	9.70	0.00	0.00	2.61	1.63
20	10	20	2181.49	0.57	0.00	0.00	2.23	1.22
30	20	30	2129.22	26.80	0.00	0.00	2.68	1.94
40	30	190	2024.68	2.83	0.00	0.00	2.55	1.77
50	40	50	2860.66	12.73	0.00	0.00	3.61	3.36
60	50	60	2492.75	11.42	0.00	0.00	3.14	2.60
70	60	70	2388.21	5.74	0.00	0.00	3.01	2.41
80	70	80	2100.05	5.48	0.00	0.00	2.65	1.90
90	80	90	511.37	1.96	0.00	0.00	1.45	1.00
100	90	100	393.11	0.57	0.00	0.00	1.12	0.61
110	100	110	332.30	1.39	0.00	0.00	0.94	0.45
120	110	120	542.36	19.92	0.00	0.00	2.22	2.70
130	120	130	452.36	18.78	0.00	0.00	2.89	5.73
140	130	140	-128.84	1.94	0.00	0.00	0.82	0.56
150	140	150	-233.38	9.31	0.00	0.00	1.49	1.68
160	140	160	52.27	0.25	0.00	0.00	0.33	0.11
170	150	170	-285.65	22.93	0.00	0.00	1.82	2.44
180	170	180	-263.37	7.62	0.00	0.00	1.68	2.10
190	180	50	-315.64	23.45	0.00	0.00	2.01	2.94
200	190	370	1920.14	11.96	0.00	0.00	2.42	1.61
210	190	200	52.27	0.35	0.00	0.00	0.33	0.11
220	30	210	52.27	0.39	0.00	0.00	0.59	0.43
230	60	220	52.27	0.75	0.00	0.00	0.59	0.43
240	80	230	126.82	6.95	0.00	0.00	1.44	2.21
250	230	170	74.55	1.49	0.00	0.00	0.85	0.82
260	100	240	234.54	0.62	0.00	0.00	0.96	0.57
270	240	400	182.27	3.78	0.00	0.00	2.07	4.32
280	250	260	-79.08	3.57	0.00	0.00	0.90	0.92
290	250	270	104.54	0.88	0.00	0.00	0.67	0.38
300	270	280	52.27	0.11	0.00	0.00	0.33	0.11
310	70	260	235.89	11.06	0.00	0.00	1.51	1.72
320	260	290	104.54	1.47	0.00	0.00	0.67	0.38
330	290	300	52.27	0.29	0.00	0.00	0.33	0.11
340	90	380	65.99	0.01	0.00	0.00	0.19	0.02
350	310	320	104.54	0.08	0.00	0.00	0.30	0.05
360	320	330	52.27	0.15	0.00	0.00	0.33	0.11
370	110	340	101.87	3.69	0.00	0.00	1.16	1.47
380	40	350	37.40	0.20	0.00	0.00	0.42	0.23
390	10	360	-2233.76	10.51	0.00	0.00	2.28	1.27
400	40	370	-2950.33	3.65	0.00	0.00	3.72	3.56
410-BNCV	0	370	1082.46	10.24	0.00	0.00	3.07	4.00
430	80	380	1409.60	1.97	0.00	0.00	1.78	0.91
440	110	340	624.03	3.69	0.00	0.00	1.77	1.44
450	380	420	1266.50	0.54	0.00	0.00	1.60	0.74
,60	310	380	-156.81	0.25	0.00	0.00	0.44	0.11
480	250	400	-77.73	2.22	0.00	0.00	0.88	0.89
500	400	410	52.27	0.00	0.00	0.00	0.07	0.00
520	110	420	-988.23	1.41	0.00	0.00	1.25	0.47
530	100	420	-226.00	0.02	0.00	0.00	0.28	0.03

JUNCTION NODE RESULTS

JUNCTION NUMBER	JUNCTION TITLE	EXTERNAL DEMAND (gpm)	HYDRAULIC GRADE (ft)	JUNCTION ELEVATION (ft)	PRESSURE HEAD (ft)	JUNCTION PRESSURE (psi)
10-1		52.27	208.93	25.00	183.93	79.70
20-1	20 TO 18	52.27	208.36	25.00	183.36	79.45
30-1		52.27	181.56	25.00	156.56	67.84
40-1		52.27	163.11	25.00	138.11	59.85
50-1		52.27	150.38	25.00	125.38	54.33
60-1	HIGH SCHOOL	52.27	138.96	25.00	113.96	49.38
70-1	S Main & Alt	52.27	133.22	25.00	108.22	46.90
80-1	S Main & 9th	52.27	127.74	25.00	102.74	44.52
90-1	CITY HALL	52.27	125.78	25.00	100.78	43.67
100-1		52.27	125.21	25.00	100.21	43.42
110-1	War Dr @ 5th	52.27	123.82	25.00	98.82	42.82
120-1	NYGARD, BIOP	90.00	103.89	25.00	78.89	34.19
130-1	PT. ADAMS SE	581.20	85.12	25.00	60.12	26.05
140-1		52.27	87.06	25.00	62.06	26.89
150-1	FT. STEVENS	52.27	96.37	25.00	71.37	30.93
160-1	FT. STEVENS	52.27	86.81	25.00	61.81	26.78
170-1		52.27	119.30	25.00	94.30	40.86
180-1		52.27	126.93	25.00	101.93	44.17
190-1		52.27	178.72	25.00	153.72	66.61
200-1	SUNSET BEACH	52.27	178.38	25.00	153.38	66.46
210-1	CULLABY LAKE	52.27	181.17	25.00	156.17	67.67
220-1		52.27	138.21	25.00	113.21	49.06
230-1		52.27	120.79	25.00	95.79	41.51
240-1		52.27	124.59	25.00	99.59	43.15
250-1	HARBOR/MARLI	52.27	118.59	25.00	93.59	40.55
260-1	MARLIN/101	52.27	122.16	25.00	97.16	42.10
270-1	SHILO INN	52.27	117.71	25.00	92.71	40.17
280-1	YOUNGS BAY P	52.27	117.60	25.00	92.60	40.13
290-1		52.27	120.69	25.00	95.69	41.47
300-1	AIRPORT	52.27	120.41	25.00	95.41	41.34
310-1	SW2/CEDAR	52.27	125.52	25.00	100.52	43.56
320-1		52.27	125.43	25.00	100.43	43.52
330-1	ROD GRAMSON	52.27	125.28	25.00	100.28	43.46
340-1	PACIFIC COAS	725.90	120.13	25.00	95.13	41.23
350-1	CAMP RILEA	37.40	162.92	25.00	137.92	59.76
360-1	GEARHART MET	322.60	219.44	25.00	194.44	84.26
370-1		52.27	166.76	25.00	141.76	61.43
380-1		52.27	125.77	25.00	100.77	43.67
400-1		52.27	120.80	25.00	95.80	41.52
410-1		52.27	120.80	25.00	95.80	41.51
420-1		52.27	125.23	25.00	100.23	43.44

SUMMARY OF INFLOWS AND OUTFLOWS

INFLOWS INTO THE SYSTEM FROM BOUNDARY NODES
(-) OUTFLOWS FROM THE SYSTEM INTO BOUNDARY NODES

PIPE NUMBER	FLOWRATE (gpm)

```
          --------------------
            10          2556.36
           410          1082.46

 ET SYSTEM INFLOW  =   3638.82
NET SYSTEM OUTFLOW =      0.00
NET SYSTEM DEMAND  =   3638.82


**** CYBERNET SIMULATION COMPLETED ****


DATE:  1/03/1997
TIME: 11:45:54
```

1996 MDD
INPUT FILE

```
**********************************************
 S U M M A R Y   O F   O R I G I N A L   D A T A
**********************************************
```

CyberNet Version 2.18. Copyright 1991,92 Haestad Methods Inc.
Run Description: 1996 MDD system w/18" on Main St completed
Drawing: WAR1996

P I P E L I N E   D A T A

STATUS CODE: XX -CLOSED PIPE   BN -BOUNDARY NODE   PU -PUMP LINE
CV -CHECK VALVE   RV -REGULATING VALVE

PIPE NUMBER	NODE NOS. #1	#2	LENGTH (ft)	DIAMETER (in)	ROUGHNESS COEFF.	MINOR LOSS COEFF.	BND-HG (ft)
10-BN	0	360	5937.0	20.0	110.00	0.00	229.1
20	10	20	467.0	20.0	110.00	0.00	
30	20	30	13783.0	18.0	110.00	0.00	
40	30	190	1599.0	18.0	110.00	0.00	
50	40	50	3790.0	18.0	110.00	0.00	
60	50	60	4385.0	18.0	110.00	0.00	
70	60	70	2386.0	18.0	110.00	0.00	
80	70	80	2892.0	18.0	110.00	0.00	
90	80	90	1960.0	12.0	110.00	0.00	
100	90	100	937.0	12.0	110.00	0.00	
110	100	110	3093.0	12.0	110.00	0.00	
120	110	120	7368.0	10.0	110.00	0.00	
130	120	130	3278.0	8.0	110.00	0.00	
140	130	140	3468.0	8.0	110.00	0.00	
150	140	150	5538.0	8.0	110.00	0.00	
160	140	160	2370.0	8.0	110.00	0.00	
170	150	170	9380.0	8.0	110.00	0.00	
180	170	180	3624.0	8.0	110.00	0.00	
190	180	50	7973.0	8.0	110.00	0.00	
200	190	370	7449.0	18.0	110.00	0.00	
210	190	200	3281.0	8.0	110.00	0.00	
220	30	210	906.0	6.0	110.00	0.00	
230	60	220	1753.0	6.0	110.00	0.00	
240	80	230	3149.0	6.0	110.00	0.00	
250	230	170	1803.0	6.0	110.00	0.00	
260	100	240	1088.0	10.0	110.00	0.00	
270	240	400	876.0	6.0	110.00	0.00	
280	250	260	3884.0	6.0	110.00	0.00	
290	250	270	2313.0	8.0	110.00	0.00	
300	270	280	1025.0	8.0	110.00	0.00	
310	70	260	6449.0	8.0	110.00	0.00	
320	260	290	3858.0	8.0	110.00	0.00	
330	290	300	2728.0	8.0	110.00	0.00	
340	90	380	640.0	12.0	110.00	0.00	
350	310	320	1585.0	12.0	110.00	0.00	
360	320	330	1448.0	8.0	110.00	0.00	
370	110	340	2507.0	6.0	110.00	0.00	
380	40	350	851.0	6.0	110.00	0.00	
390	10	360	8266.0	20.0	110.00	0.00	
400	40	370	1027.0	18.0	110.00	0.00	
410-BNCV	0	370	2558.0	12.0	110.00	0.00	177.00

430	80	380	2175.0	18.0	110.00	0.00
440	110	340	2555.0	12.0	110.00	0.00
450	380	420	721.0	18.0	110.00	0.00
460	310	380	2258.0	12.0	110.00	0.00
480	250	400	2487.0	6.0	110.00	0.00
500	400	410	466.0	18.0	110.00	0.00
520	110	420	3002.0	18.0	110.00	0.00
530	100	420	806.0	18.0	110.00	0.00

J U N C T I O N   N O D E   D A T A

JUNCTION NUMBER	JUNCTION TITLE	EXTERNAL DEMAND (gpm)	JUNCTION ELEVATION (ft)	CONNECTING PIPES				
10-1		52.27	25.00	20	390			
20-1	20 TO 18	52.27	25.00	20	30			
30-1		52.27	25.00	30	40	220		
40-1		52.27	25.00	50	380	400		
50-1		52.27	25.00	50	60	190		
60-1	HIGH SCHOOL	52.27	25.00	60	70	230		
70-1	S Main & Alt	52.27	25.00	70	80	310		
80-1	S Main & 9th	52.27	25.00	80	90	240	430	
90-1	CITY HALL	52.27	25.00	90	100	340		
100-1		52.27	25.00	100	110	260	530	
110-1	War Dr @ 5th	52.27	25.00	110	120	370	440	520
120-1	NYGARD, BIOP	90.00	25.00	120	130			
130-1	PT. ADAMS SE	581.20	25.00	130	140			
140-1		52.27	25.00	140	150	160		
150-1	FT. STEVENS	52.27	25.00	150	170			
160-1	FT. STEVENS	52.27	25.00	160				
170-1		52.27	25.00	170	180	250		
180-1		52.27	25.00	180	190			
190-1		52.27	25.00	40	200	210		
200-1	SUNSET BEACH	52.27	25.00	210				
210-1	CULLABY LAKE	52.27	25.00	220				
220-1		52.27	25.00	230				
230-1		52.27	25.00	240	250			
240-1		52.27	25.00	260	270			
250-1	HARBOR/MARLI	52.27	25.00	280	290	480		
260-1	MARLIN/101	52.27	25.00	280	310	320		
270-1	SHILO INN	52.27	25.00	290	300			
280-1	YOUNGS BAY P	52.27	25.00	300				
290-1		52.27	25.00	320	330			
300-1	AIRPORT	52.27	25.00	330				
310-1	SW2/CEDAR	52.27	25.00	350	460			
320-1		52.27	25.00	350	360			
330-1	ROD GRAMSON	52.27	25.00	360				
340-1	PACIFIC COAS	725.90	25.00	370	440			
350-1	CAMP RILEA	37.40	25.00	380				
360-1	GEARHART MET	322.60	25.00	10	390			
370-1		52.27	25.00	200	400	410		
380-1		52.27	25.00	340	430	450	460	
400-1		52.27	25.00	270	480	500		
410-1		52.27	25.00	500				
420-1		52.27	25.00	450	520	530		

APPENDIX E

2016 Computer Model

2016 MDD

OUTPUT FILE

```
+-----------------------------------------------+
|              MAXIMUM DIMENSIONS               |
|                                               |
|  Number of pipes ..................... 1000   |
|  Number of pumps .....................  250   |
|  Number junction nodes................ 1000   |
|  Flow meters .........................  250   |
|  Boundary nodes ......................  100   |
|  Variable storage tanks ..............  250   |
|  Pressure switches ...................  250   |
|  Regulating Valves....................  250   |
|  Items for limited output ............ 1000   |
|  limit for non-consecutive numbering ..10260  |
+-----------------------------------------------+
```

Cybernet version 2.18. SN: 1132183902-1000

Extended Description:

U N I T S   S P E C I F I E D

```
FLOWRATE ............ = gallons/minute
HEAD (HGL) .......... = feet
PRESSURE ............ = psig
```

O U T P U T   O P T I O N   D A T A

OUTPUT SELECTION: ALL RESULTS ARE INCLUDED IN THE TABULATED OUTPUT

S Y S T E M   C O N F I G U R A T I O N

```
NUMBER OF PIPES ....................(p) =   57
NUMBER OF JUNCTION NODES ...........(j) =   44
NUMBER OF PRIMARY LOOPS ............(l) =   11
NUMBER OF BOUNDARY NODES ...........(f) =    3
NUMBER OF SUPPLY ZONES .............(z) =    1
```

```
***********************************
  S I M U L A T I O N   R E S U L T S
***********************************
```

The results are obtained after 7 trials with an accuracy = 0.00119

S I M U L A T I O N   D E S C R I P T I O N

CyberNet Version 2.18. Copyright 1991,92 Haestad Methods Inc.
Run Description: 2016 MDD system w/18" industrial grid
Drawing: WAR2016

P I P E L I N E   R E S U L T S

ATUS CODE: XX -CLOSED PIPE BN -BOUNDARY NODE PU -PUMP LINE
CV -CHECK VALVE RV -REGULATING VALVE TK -STORAGE TANK

PIPE NUMBER	NODE NOS. #1	#2	FLOWRATE (gpm)	HEAD LOSS (ft)	PUMP HEAD (ft)	MINOR LOSS (ft)	LINE VELO. (ft/s)	HL/ 1000 (ft/ft)
10-BN	0	360	1967.85	5.97	0.00	0.00	2.01	1.01
20	10	20	1464.85	0.27	0.00	0.00	1.50	0.58
30	20	30	1412.58	12.53	0.00	0.00	1.78	0.91
40	30	190	1308.04	1.26	0.00	0.00	1.65	0.79
50	40	50	1057.06	2.01	0.00	0.00	1.33	0.53
60	50	60	721.71	1.15	0.00	0.00	0.91	0.26
70	60	70	617.17	0.47	0.00	0.00	0.78	0.20
80	70	80	1472.88	2.84	0.00	0.00	1.86	0.98
90	80	90	320.87	0.83	0.00	0.00	0.91	0.42
100	90	100	185.53	0.14	0.00	0.00	0.53	0.15
110	100	110	391.79	1.88	0.00	0.00	1.11	0.61
120	110	120	643.37	27.34	0.00	0.00	2.63	3.71
130	120	130	542.57	26.30	0.00	0.00	3.46	8.02
140	130	140	-108.53	1.41	0.00	0.00	0.69	0.41
150	140	150	-235.40	9.46	0.00	0.00	1.50	1.71
160	140	160	52.27	0.25	0.00	0.00	0.33	0.11
170	150	170	-298.80	24.93	0.00	0.00	1.91	2.66
180	170	180	-230.82	5.97	0.00	0.00	1.47	1.65
190	180	50	-283.09	19.17	0.00	0.00	1.81	2.40
200	190	370	1203.50	5.04	0.00	0.00	1.52	0.68
210	190	200	52.27	0.35	0.00	0.00	0.33	0.11
220	30	210	52.27	0.39	0.00	0.00	0.59	0.43
230	60	220	52.27	0.75	0.00	0.00	0.59	0.43
240	80	230	206.12	17.08	0.00	0.00	2.34	5.42
250	230	170	120.25	3.60	0.00	0.00	1.36	2.00
260	100	240	-10.11	0.00	0.00	0.00	0.04	0.00
270	240	400	-62.38	0.52	0.00	0.00	0.71	0.59
280	250	260	-54.28	1.78	0.00	0.00	0.62	0.46
290	250	270	104.54	0.88	0.00	0.00	0.67	0.38
300	270	280	52.27	0.11	0.00	0.00	0.33	0.11
310	70	260	58.32	0.83	0.00	0.00	0.37	0.13
320	260	290	104.54	1.47	0.00	0.00	0.67	0.38
330	290	300	52.27	0.29	0.00	0.00	0.33	0.11
340	90	380	83.07	0.02	0.00	0.00	0.24	0.03
350	310	320	122.84	0.11	0.00	0.00	0.35	0.07
360	320	330	70.57	0.27	0.00	0.00	0.45	0.18
370	110	340	114.13	4.55	0.00	0.00	1.29	1.81
380	40	350	41.90	0.24	0.00	0.00	0.48	0.28
390	10	360	-1606.45	5.71	0.00	0.00	1.64	0.69
400	40	370	-1151.23	0.64	0.00	0.00	1.45	0.62
410-XXBN	0	370						
430	80	380	893.62	0.85	0.00	0.00	1.13	0.39
440	110	340	699.07	4.55	0.00	0.00	1.98	1.78
450	380	420	749.31	0.20	0.00	0.00	0.94	0.28
:60	310	380	-175.11	0.31	0.00	0.00	0.50	0.14
480	250	400	42.69	0.73	0.00	0.00	0.48	0.29
500	400	410	-71.96	0.00	0.00	0.00	0.09	0.00
520	110	420	-1129.05	1.80	0.00	0.00	1.42	0.60
530	100	420	432.01	0.08	0.00	0.00	0.54	0.10

540	70	430	-966.31	3.43	0.00	0.00	1.22	0.45
550-BN	430	0	-2726.55	7.49	0.00	0.00	3.44	3.07
570	430	440	1541.04	2.19	0.00	0.00	1.94	1.07
580	440	450	219.20	0.13	0.00	0.00	0.28	0.03
590	440	260	1102.64	2.08	0.00	0.00	1.39	0.57
600	260	250	949.87	1.73	0.00	0.00	1.20	0.44
620	250	410	804.66	0.73	0.00	0.00	1.01	0.32
640	410	100	680.42	0.52	0.00	0.00	0.86	0.24

J U N C T I O N   N O D E   R E S U L T S

JUNCTION NUMBER	JUNCTION TITLE	EXTERNAL DEMAND (gpm)	HYDRAULIC GRADE (ft)	JUNCTION ELEVATION (ft)	PRESSURE HEAD (ft)	JUNCTION PRESSURE (psi)
10-1	Add 200 SF S	141.60	217.45	25.00	192.45	83.40
20-1	20 TO 18	52.27	217.18	25.00	192.18	83.28
30-1		52.27	204.65	25.00	179.65	77.85
40-1		52.27	197.71	25.00	172.71	74.84
50-1		52.27	195.70	25.00	170.70	73.97
60-1	HIGH SCHOOL	52.27	194.55	25.00	169.55	73.47
70-1	S Main & Alt	52.27	194.08	25.00	169.08	73.27
80-1	S Main & 9th	52.27	191.24	25.00	166.24	72.04
90-1	CITY HALL	52.27	190.41	25.00	165.41	71.68
100-1		52.27	190.27	25.00	165.27	71.62
110-1	War Dr @ 5th	64.27	188.38	25.00	163.38	70.80
120-1	NYGARD, BIOP	100.80	161.05	25.00	136.05	58.95
130-1	PT. ADAMS SE	651.10	134.75	25.00	109.75	47.56
140-1	Add 50 SF	74.60	136.17	25.00	111.17	48.17
150-1	FT. STEVENS	63.40	145.63	25.00	120.63	52.27
160-1	FT. STEVENS	52.27	135.92	25.00	110.92	48.06
170-1		52.27	170.56	25.00	145.56	63.07
180-1		52.27	176.53	25.00	151.53	65.66
190-1		52.27	203.39	25.00	178.39	77.30
200-1	SUNSET BEACH	52.27	203.04	25.00	178.04	77.15
210-1	CULLABY LAKE	52.27	204.26	25.00	179.26	77.68
220-1		52.27	193.80	25.00	168.80	73.15
230-1	Add 75 SF	85.87	174.16	25.00	149.16	64.64
240-1		52.27	190.27	25.00	165.27	71.62
250-1	HARBOR/MARLI	52.27	191.47	25.00	166.47	72.14
260-1	MARLIN/101	52.27	193.25	25.00	168.25	72.91
270-1	SHILO INN	52.27	190.59	25.00	165.59	71.76
280-1	YOUNGS BAY P	52.27	190.48	25.00	165.48	71.71
290-1		52.27	191.78	25.00	166.78	72.27
300-1	AIRPORT	52.27	191.49	25.00	166.49	72.15
310-1	SW2/CEDAR	52.27	190.08	25.00	165.08	71.54
320-1		52.27	189.97	25.00	164.97	71.49
330-1	ROD GRAMSON,	70.57	189.70	25.00	164.70	71.37
340-1	PACIFIC COAS	813.20	183.84	25.00	158.84	68.83
350-1	CAMP RILEA	41.90	197.47	25.00	172.47	74.74
360-1	GEARHART MET	361.40	223.16	25.00	198.16	85.87
370-1		52.27	198.35	25.00	173.35	75.12
380-1		52.27	190.39	25.00	165.39	71.67
400-1		52.27	190.74	25.00	165.74	71.82
410-1		52.27	190.74	25.00	165.74	71.82
420-1		52.27	190.19	25.00	165.19	71.58
430-1	North Coast	219.20	197.51	60.00	137.51	59.59
440-1	North Coast	219.20	195.32	60.00	135.32	58.64

450-1 North Coast	219.20	195.19	60.00	135.19	58.58

UMMARY OF INFLOWS AND OUTFLOWS

(+) INFLOWS INTO THE SYSTEM FROM BOUNDARY NODES
(-) OUTFLOWS FROM THE SYSTEM INTO BOUNDARY NODES

PIPE NUMBER	FLOWRATE (gpm)
10	1967.85
550	2726.55

NET SYSTEM INFLOW = 4694.41
NET SYSTEM OUTFLOW = 0.00
NET SYSTEM DEMAND = 4694.41

**** CYBERNET SIMULATION COMPLETED ****

DATE: 1/03/1997
TIME: 12:04:38

****

S U M M A R Y   O F   O R I G I N A L   D A T A

****

2016 MDD INPUT FILE

CyberNet Version 2.18. Copyright 1991,92 Haestad Methods Inc.
Run Description: 2016 MDD system w/18" industrial grid
Drawing: WAR2016

P I P E L I N E   D A T A

STATUS CODE: XX -CLOSED PIPE   BN -BOUNDARY NODE   PU -PUMP LINE
CV -CHECK VALVE   RV -REGULATING VALVE

PIPE NUMBER	NODE NOS. #1	#2	LENGTH (ft)	DIAMETER (in)	ROUGHNESS COEFF.	MINOR LOSS COEFF.	BND-HGI (ft)
10-BN	0	360	5937.0	20.0	110.00	0.00	229.14
20	10	20	467.0	20.0	110.00	0.00	
30	20	30	13783.0	18.0	110.00	0.00	
40	30	190	1599.0	18.0	110.00	0.00	
50	40	50	3790.0	18.0	110.00	0.00	
60	50	60	4385.0	18.0	110.00	0.00	
70	60	70	2386.0	18.0	110.00	0.00	
80	70	80	2892.0	18.0	110.00	0.00	
90	80	90	1960.0	12.0	110.00	0.00	
100	90	100	937.0	12.0	110.00	0.00	
110	100	110	3093.0	12.0	110.00	0.00	
120	110	120	7368.0	10.0	110.00	0.00	
130	120	130	3278.0	8.0	110.00	0.00	
140	130	140	3468.0	8.0	110.00	0.00	
150	140	150	5538.0	8.0	110.00	0.00	
160	140	160	2370.0	8.0	110.00	0.00	
170	150	170	9380.0	8.0	110.00	0.00	
180	170	180	3624.0	8.0	110.00	0.00	
190	180	50	7973.0	8.0	110.00	0.00	
200	190	370	7449.0	18.0	110.00	0.00	
210	190	200	3281.0	8.0	110.00	0.00	
220	30	210	906.0	6.0	110.00	0.00	
230	60	220	1753.0	6.0	110.00	0.00	
240	80	230	3149.0	6.0	110.00	0.00	
250	230	170	1803.0	6.0	110.00	0.00	
260	100	240	1088.0	10.0	110.00	0.00	
270	240	400	876.0	6.0	110.00	0.00	
280	250	260	3884.0	6.0	110.00	0.00	
290	250	270	2313.0	8.0	110.00	0.00	
300	270	280	1025.0	8.0	110.00	0.00	
310	70	260	6449.0	8.0	110.00	0.00	
320	260	290	3858.0	8.0	110.00	0.00	
330	290	300	2728.0	8.0	110.00	0.00	
340	90	380	640.0	12.0	110.00	0.00	
350	310	320	1585.0	12.0	110.00	0.00	
360	320	330	1448.0	8.0	110.00	0.00	
370	110	340	2507.0	6.0	110.00	0.00	
380	40	350	851.0	6.0	110.00	0.00	
390	10	360	8266.0	20.0	110.00	0.00	
400	40	370	1027.0	18.0	110.00	0.00	
410-BNCV	0	370	2558.0	12.0	110.00	0.00	177.00

430	80	380	2175.0	18.0	110.00	0.00	
440	110	340	2555.0	12.0	110.00	0.00	
450	380	420	721.0	18.0	110.00	0.00	
460	310	380	2258.0	12.0	110.00	0.00	
480	250	400	2487.0	6.0	110.00	0.00	
500	400	410	466.0	18.0	110.00	0.00	
520	110	420	3002.0	18.0	110.00	0.00	
530	100	420	806.0	18.0	110.00	0.00	
540	70	430	7626.0	18.0	110.00	0.00	
550-BN	430	0	2436.0	18.0	110.00	0.00	205.0
570	430	440	2049.0	18.0	110.00	0.00	
580	440	450	4473.0	18.0	110.00	0.00	
590	440	260	3610.0	18.0	110.00	0.00	
600	260	250	3960.0	18.0	110.00	0.00	
620	250	410	2272.0	18.0	110.00	0.00	
640	410	100	2224.0	18.0	110.00	0.00	

J U N C T I O N   N O D E   D A T A

JUNCTION NUMBER	JUNCTION TITLE	EXTERNAL DEMAND (gpm)	JUNCTION ELEVATION (ft)	CONNECTING PIPES				
10-1	Add 200 SF S	141.60	25.00	20	390			
20-1	20 TO 18	52.27	25.00	20	30			
30-1		52.27	25.00	30	40	220		
40-1		52.27	25.00	50	380	400		
50-1		52.27	25.00	50	60	190		
60-1	HIGH SCHOOL	52.27	25.00	60	70	230		
70-1	S Main & Alt	52.27	25.00	70	80	310	540	
80-1	S Main & 9th	52.27	25.00	80	90	240	430	
90-1	CITY HALL	52.27	25.00	90	100	340		
100-1		52.27	25.00	100	110	260	530	64(
110-1	War Dr @ 5th	64.27	25.00	110	120	370	440	52(
120-1	NYGARD, BIOP	100.80	25.00	120	130			
130-1	PT. ADAMS SE	651.10	25.00	130	140			
140-1	Add 50 SF	74.60	25.00	140	150	160		
150-1	FT. STEVENS	63.40	25.00	150	170			
160-1	FT. STEVENS	52.27	25.00	160				
170-1		52.27	25.00	170	180	250		
180-1		52.27	25.00	180	190			
190-1		52.27	25.00	40	200	210		
200-1	SUNSET BEACH	52.27	25.00	210				
210-1	CULLABY LAKE	52.27	25.00	220				
220-1		52.27	25.00	230				
230-1	Add 75 SF	85.87	25.00	240	250			
240-1		52.27	25.00	260	270			
250-1	HARBOR/MARLI	52.27	25.00	280	290	480	600	620
260-1	MARLIN/101	52.27	25.00	280	310	320	590	600
270-1	SHILO INN	52.27	25.00	290	300			
280-1	YOUNGS BAY P	52.27	25.00	300				
290-1		52.27	25.00	320	330			
300-1	AIRPORT	52.27	25.00	330				
310-1	SW2/CEDAR	52.27	25.00	350	460			
320-1		52.27	25.00	350	360			
330-1	ROD GRAMSON,	70.57	25.00	360				
340-1	PACIFIC COAS	813.20	25.00	370	440			
350-1	CAMP RILEA	41.90	25.00	380				

360-1	GEARHART MET	361.40	25.00	10	390		
370-1		52.27	25.00	200	400	410	
380-1		52.27	25.00	340	430	450	460
400-1		52.27	25.00	270	480	500	
410-1		52.27	25.00	500	620	640	
420-1		52.27	25.00	450	520	530	
430-1	North Coast	219.20	60.00	540	550	570	
440-1	North Coast	219.20	60.00	570	580	590	
450-1	North Coast	219.20	60.00	580			

CH2M HILL

Celebrating 50 Years

November 12, 1996

132630.WT.AS

Mr. Chuck Todd, P.E.
City Engineer
P.O. Box 250
Warrenton, OR 97146-0250

Dear Chuck:

Subject: Proposal for Pilot Testing Membrane Filtration

An emerging drinking water treatment technology that may have application for Warrenton is microfiltration. As we discussed when I met with you last week, CH2M HILL currently has access to a microfiltration portable pilot unit that could be used without cost. Results from pilot testing this technology would be useful as the city continues to consider options for compliance with the Surface Water Treatment Rule.

The following sections provide a description of microfiltration, discuss its advantages and disadvantages, and present a proposed plan for carrying out a pilot test.

Description of Microfiltration

Microfiltration is a physical straining process used for removing turbidity and microorganisms from drinking water. Water is pushed into the center of a hollow fiber, polypropylene membrane. The membranes have a nominal opening size of 0.2 microns, 10 to 30 times smaller than the size of the target organisms. The removed particles and organisms remain on the outside and are flushed away using compressed air and backwash water.

To provide a comparison, slow sand filtration is a biological/physical process and rapid rate filtration is a chemical/physical process. Microfiltration is simply a physical process. Similar to slow sand filtration, microfiltration requires no coagulation chemicals. It requires only limited operator process control and has been found to be very reliable in producing safe water. Also like slow sand filtration, its use is limited to source waters with relatively low levels of particulates. Warrenton's source waters appear to meet this requirement.

Because water must be pushed through the membrane, microfiltration does have pressure requirements. The required operating head is 30 to 50 psi, with a maximum pressure loss of 15 psi.

Corvallis Office
2300 NW Walnut Blvd., Corvallis, OR 97330-3538
P.O. Box 428, Corvallis, OR 97339-0428
541 752-4271
Fax No. 541 752-0276

Advantages of Microfiltration

The advantages of microfiltration include a small plant footprint (reducing the need for land purchase in Warrenton's case), reliability of treatment with little operator process control, availability of modular equipment that facilitates automatic control, and the avoidance of needing to use coagulant chemicals.

Disadvantages of Microfiltration

The disadvantages of microfiltration include a relatively high capital cost and the pressure limitations (the elevation of the plant must be carefully selected to provide the correct hydraulics, and low head pumping may be required). It also has less history than slow sand or rapid rate filtration, although the Oregon Health Division recognizes its effectiveness.

Comparison to Slow Sand Filtration

Microfiltration's reliability and avoidance of coagulant chemicals are advantages that are similar to slow sand filtration. In contrast to slow sand filtration, it offers a much smaller plant footprint and the ease of automatic control.

Slow sand filtration will require labor-intensive cleaning of the filters every 4 to 8 weeks (estimated from the pilot testing), and resanding about every 5 to 10 years. A microfiltration plant will require replacement of the membranes every 3 to 5 years, but this is a relatively easy task. It will also require periodic cleaning of the membranes using a washing chemical. The frequency of the cleaning is similar to the estimated frequency cleaning of the slow sand filters, but again the process is easier and faster for microfiltration.

A microfiltration plant would require more mechanical maintenance than a slow sand filtration plant.

As we briefly discussed last week, sand media costs for slow sand filtration have risen dramatically in recent months. The net result may be that capital costs for slow sand filtration and microfiltration are similar.

Proposed Pilot Testing Plan

One of the premier microfiltration equipment suppliers, Memtec, Inc., has offered CH2M HILL the use of their portable pilot testing unit over the next several months. The unit is currently located in the Portland area. It is easily transportable by pickup truck.

We would like to work with Warrenton in setting up and operating this unit over a two-month period to provide data for your system. We could provide onsite help in setting up the unit and guidance to you in operating it. On Warrenton's end, we would ask that your staff visit the facility once or twice per day on weekdays to make observations and record data. Only 10 to 15 minutes per visit would be needed. Data to record include raw and filtered turbidity, flow rate, and headloss through the unit. At the end of the study, we would summarize the data and provide the city with an assessment of the potential for using microfiltration. Pilot results would also be a basis for estimating costs for a full scale microfiltration plant.

I estimate that engineering services to assist the city in the pilot testing would be approximately $2,500. However, I would like to develop a firm scope of work with you before committing to a cost.

I believe this pilot testing would benefit the city in several ways. It will

- acquaint you and your staff with microfiltration so that you can make an informed decision about its use
- fulfill the Oregon Health Division's requirement for pilot testing in case the city wishes to use microfiltration
- provide a basis for developing design criteria so that costs for microfiltration can be estimated with more accuracy
- give the city a further comparison to using slow sand filtration. This will help to confirm in your mind whether or not slow sand filtration is the preferred treatment method. Even if the conclusion is not to use microfiltration, the pilot testing will be further evidence to the public that the city has investigated all options.

I covered a lot of ground in this letter, so please contact me if I can clarify the information or provide additional information. By presenting this opportunity I do not mean to bring into question the findings of the slow sand filtration pilot study report. Rather, I want you to be aware of another treatment approach that could provide benefits to the city.

I will look forward to hearing from you.

Sincerely,

CH2M HILL

Paul A Berg

Paul Berg, P.E.
Project Manager

CVO/Microlet.doc