CORVALLIS
ENHANCING COMMUNITY LIVABILITY

Corvallis Water Distribution System Facility Plan

July 1998

BROWN AND CALDWELL

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CITY OF CORVALLIS

WATER DISTRIBUTION SYSTEM FACILITY PLAN

July 1998

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EXECUTIVE SUMMARY

The Corvallis water distribution system needs to be expanded to improve operational characteristics and to accommodate planned growth in the community. This facility plan evaluates the existing water distribution system, identifies the planned growth for the community, projects water demand which will accompany the future growth, and recommends improvements to the water distribution system. Also included as part of the development of the facility plan is a computer model of the water distribution system to evaluate system operation and the impact of proposed improvements.

The 1996 Comprehensive Plan for the City of Corvallis is the basis for the growth projections in the water system facility plan. Service is planned for the urban growth boundary of Corvallis based upon the zoning that is currently designated for the growth areas. No provisions for water service are included in the facility plan for areas outside the urban growth area.

POPULATION AND WATER DEMAND PROJECTIONS

To develop the water facility plan, land use and population are the primary criteria for estimating future water demands. The projected population for the City of Corvallis are as follows:

Planning Period	Population
1997	50,000
Within 10 to 20 years	60,000
Within 20 to 40 years	80,000
At build out of urban growth boundary	120,000

The annual average water demand for Corvallis for the years 1992 through June 1997 was 7.5 million gallons per day (mgd) while the peak monthly water demand was 11.9 mgd, occurring in July 1996. Based on current water use records, existing development, and the existing population in Corvallis, the following unit water consumption values have been developed for the City of Corvallis:

Land use	Annual Average water consumption
Residential	76 gallons per capita per day
Commercial	1,000 gallons per acre per day
Industrial	3,750 gallons per acre per day
Institutional	1,550 gallons per acre per day

Experience shows that water demand in Corvallis varies seasonally based upon temperature and irrigation needs. Based on recent water production records and on residential water use data, the peaking factors used to estimate water use variations are shown in Table 1.

Table 1. Peaking Factors[a]

Description	Factor
Maximum month demand	1.5
Maximum daily demand	
Residential only	4.0
Average for city	2.0
Peak hourly demand	
Residential only	11.75
Average for city	4.6

[a] The annual average demand multiplied by the peaking factor yields the respective peak demand.

Given these factors and the unit consumption values presented above, the water demand for the community is summarized in Table 2.

Table 2. Water Demand Summary

Population inside urban growth boundary	Average daily water demands, mgd	Maximum daily water demand, mgd
50,000	7.5	15
60,000	10.0	20
80,000	13.5	27
120,000	20.0	40

SYSTEM EVALUATION

A computer model (CYBERNET) was used to evaluate the pipeline network, pumping stations, and reservoirs that make up the Corvallis water distribution system. This is a water distribution model that analyzes steady state flow and pressure conditions in the pipeline network. The program uses AutoCAD for graphical presentation of input and output data which is the basis for the map included in Appendix B.

As a first step, the existing water distribution system was modeled. A field test was conducted during which the existing system was stressed by releasing water at one fire hydrant. The flow and pressure was measured at the hydrant and at key points in the system. The same condition was then modeled using CYBERNET and the data compared to verify that the model represents the field conditions. Excellent correlation was achieved with the model.

The existing water distribution system was evaluated for three conditions:

- Peak hour demand
- Maximum day demand plus fire flow
- Low demand, high water treatment plant production

The existing system is generally adequate to provide a high level of customer service. There are a few areas where improvements are necessary to increase the available fire flow and pressure. The most significant improvement is necessary to alleviate an operational problem that has existed for some time. During periods of low demand, the production capacity at the H.D. Taylor Water Plant exceeds the capacity of the distribution system to deliver water to storage at the North Hills first level reservoir. When filling of the North Hills reservoirs is attempted, pressures in the south portion of the system exceed the maximum desirable pressure for safe operation. A north/south transmission pipeline constructed between the H.D. Taylor Water Treatment Plant (WTP) and the North Hills first level reservoirs will relieve these high pressures.

For future development, the piping network analysis was extended into the unimproved areas within the urban growth boundary and the model was expanded to include these improvements. As future development occurs, new transmission pipelines, pumping station improvements, and reservoirs will be necessary. The recommended improvements for the system are summarized in the following section.

RECOMMENDED IMPROVEMENTS

Improvements have been recommended for four specific phases. The first phase is for improvements that are necessary to upgrade the existing system to achieve the desired level of service as defined in Chapter 5. The most important part of this phase is a segment of the new north/south transmission pipeline between the north side of the downtown business district and the North Hills first level reservoirs. This will eliminate the high system pressure when filling of the reservoir is necessary during low demand periods. Also included in the first phase of work are several pipelines to provide looping of the system where hydrant pressures are not adequate.

Phase II improvements are planned to provide service in those areas that are expected to be developed and do not have adequate water service. The first and second service levels in the southwest part of the city, the second and third service levels in the Timberhill area, and the first service level between North Hills and Crescent Valley High School are the areas that are expected to grow in the next 10 to 20 years. The improvements necessary include the 1 MG Baldy second level reservoir and expansion of a number of the existing pumping stations and distribution pipelines. The completion of a portion of the north/south transmission pipeline between the H.D. Taylor WTP and the North Hills first level reservoirs is also recommended for Phase II. It is recommended that the remainder of the north/south transmission pipeline be completed in Phase III.

The Phase III and Phase IV improvements are recommended for those areas that will grow in the next 20 to 40 years and then for build out of the entire urban growth boundary. It is generally assumed that the growth will occur sequentially from the core area of the city out towards the outer boundary of the service area.

The estimated capital cost for water distribution system improvements is summarized in Table 3. All of these costs include provisions for engineering, overhead, and contingencies. Costs are presented at 1998 cost levels and do not include provisions for inflation. More details related to the proposed improvements are included in Chapter 7 of the report and in the appendices.

PUBLIC PROCESS AND ADOPTION

This facilities plan was reviewed and adopted with the following public review process:

- City council briefing on the plan objectives and the review process — July 20, 1998
- Public meeting to present the planning criteria — August 4, 1998
- Public meeting to present the results of the plan including the recommended system improvements — September 8, 1998

- Planning Commission Hearing on the plan and adoption of the Comprehensive Plan amendment — November 4, 1998
- City Council hearing on the plan and approval of the ordinance amending the Comprehensive Plan to include the facilities plan — December 7, 1998

Table 3. Estimated Costs of Improvements

	Capital cost[a,b] ($1,000)
Phase I - Existing system	
Pipelines	3,865
Total Phase I	3,865
Phase II - 50,000 to 60,000 people	
Reservoirs	1,138
Pumping stations	1,706
Pipelines	20,181
Total Phase II	23,025
Phase III - 60,000 to 80,000 people	
Reservoirs	4,577
Pipelines	12,409
Total Phase III	16,986
Phase IV - 80,000 to 120,000 people	
Reservoirs	6,522
Pumping stations	1,144
Pipelines	9,707
Total Phase IV	17,373
Total of all phases	61,249

[a] Based on Engineering News-Record Construction Cost Index for Seattle of 6,747 as reported July 6, 1998.

[b] Capital costs include construction cost plus 35 percent for engineering, overhead, and contingency.

CHAPTER 1

INTRODUCTION

The Corvallis water distribution system needs to be upgraded to improve operational characteristics and expanded to accommodate planned growth in the community. The purpose of this facility plan is to assess the adequacy of the existing Corvallis water distribution system and to identify future improvements. The study defines the levels of service to be provided, discusses required system improvements (including pumping, transmission, and storage), and provides estimated project costs.

A water facility plan was last completed for the Corvallis service area in 1981. Since that time, many of the proposed improvements have been constructed and conditions have changed. Most notably, the City of Philomath has developed its own water system. In 1993, the city completed a facility plan for the H.D. Taylor Water Treatment Plant that addresses supply and treatment issues and in 1997, an expansion of the plant was completed.

The study boundary includes all of the area within the Corvallis urban growth boundary. Full service will be provided to areas below elevation 560 feet above mean sea level inside the urban growth boundary to meet the established standards for pressure, fire flows, system storage, and reliability.

The initial step in this study was to assemble the water system maps and related information and prepare a computer model of the distribution system. Data from the existing computer model for the Corvallis water system was used as the base for this effort. The new computer model includes newly constructed pipelines, reflects current operational data for the supply, and includes the improvements made at existing pumping stations. The CYBERNET computer program used to model the Corvallis water distribution system is a PC-based system. Model results have been verified by flow and pressure testing of the distribution system.

Water use patterns in Corvallis for the last five years were reviewed and summarized. These provide the basis for estimating future water demand. Standards for providing water service are included in Chapter 5. These standards set maximum and minimum water pressures for water service, anticipated fire flows, and storage requirements.

Two input files have been developed for the computer model. The first represents the existing distribution network while the second includes the proposed future system improvements. The existing system model has been used to address operation of the existing system and to identify improvements needed to achieve the service standards established for the water distribution system.

Proposed improvements have been modeled to verify that the design standards are met at ultimate development. The model was used to stress the system to meet the full build out development scenario, with projected future flows based on analysis of population growth trends. The model was also used to simulate system operation during a major fire emergency.

A cost estimate of proposed improvements is included. All costs are referenced to the Engineering News-Record Construction Cost Index (ENRCCI) for Seattle. Unit construction costs have been developed for pumping stations, storage reservoirs, and various diameter pipelines. Using these unit costs and the ENRCCI, the city can update project costs to include inflation for future periods and assess the impact of possible changes.

CHAPTER 2

STUDY AREA CHARACTERISTICS

Water supply and distribution systems are highly dependent upon local conditions, including those of the environment, population, economic growth, and land use. The specific impacts of the study area factors on water system planning are summarized in this chapter.

PHYSICAL ENVIRONMENT

The physical environment is composed of factors that include service area, climate, topography, geology, water quality, air quality and terrestrial and aquatic biology. Of these, the main factors that influence water system planning are the service area, climate, topography, and geology. The other factors can influence specific project recommendations, but generally do not play a significant role in system planning. The discussion below describes the effect of these factors on water system planning.

Service Area

Planning for the Corvallis water distribution system is based on providing service to all of the areas within the Urban Growth Boundary (UGB) below elevation 560 feet above mean sea level (fmsl). Figure 2-1 delineates the UGB (1997) and the city limits (1997). These boundaries are the basis for estimating future water demands. Water demand is a function of area and land use. In residential areas, demand determination is based on housing density while in commercial, industrial, and institutional areas, demand is based on area utilized.

Climate

Climate is associated with longer-term trends related to temperature, precipitation and wind that impact the availability and demand for water. The importance of climatic data to water system planning is that water consumption tends to be highly seasonal. The Corvallis area is dry during the summer growing period with relatively high evaporation rates. This results in substantial water demands for irrigation of lawns, gardens, parks, and school yards. Water demand figures are discussed in Chapter 4. Since the water distribution system is planned to provide peak flows, large seasonal and diurnal fluctuations in water demand will increase the size and cost of the system.

Topography

Topography has a major impact on water system planning due to its influence on water pressures, pipeline layout, population dispersion, reservoir locations, and pumping station requirements. In Corvallis the terrain is sufficiently varied so that the water systems must be laid out with multiple pressure zones to provide a practical level of pressure to each customer.

The area within the Corvallis urban growth boundary consists of relatively flat land in the south and east areas of the community and adjacent to the Willamette River, with much steeper and higher ground located in the north and west areas. The elevations within the community vary from 215 fmsl near the river, to above 600 fmsl in the north and west. This difference in elevation requires that the area be divided into water service pressure levels. Currently, there are three service levels as shown below. In order to provide service above 560 fmsl, a fourth service level would be necessary. Currently, the comprehensive plan for the City of Corvallis has no land zoned for development above the third service level.

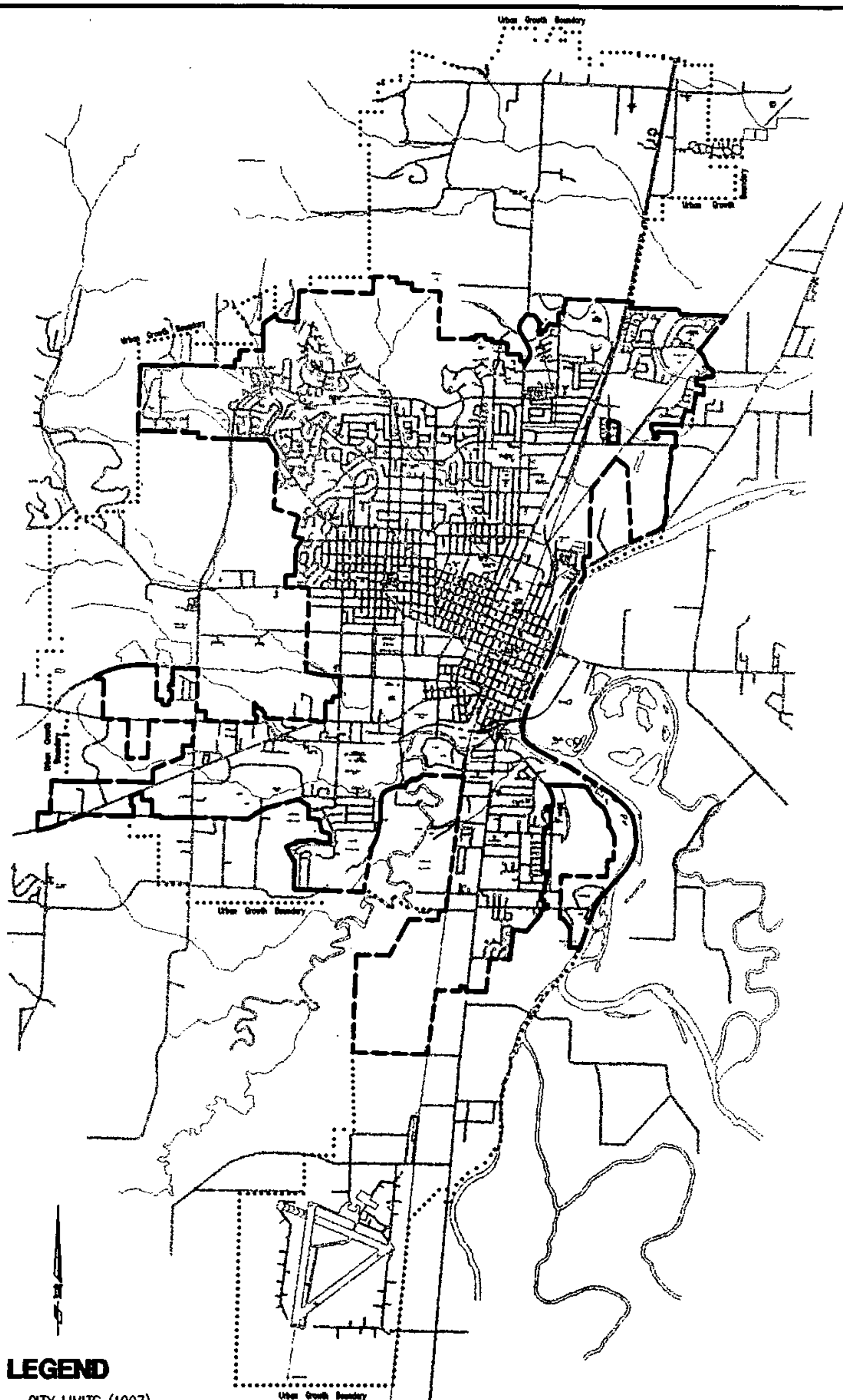
Elevation (fmsl)	Service level
410 to 560	Third
290 to 410	Second
210 to 290	First

Geology

The geology of the study area affects the cost of facilities due to the impact of subsurface structures on pipeline, reservoir, and pumping station construction. Particularly important are the proximity of bedrock to the surface and the location of groundwater relative to excavation and structures. The Corvallis area consists of unconsolidated materials composed of alluvial deposits in the lower elevations near the Willamette River. The higher elevation areas near the north and west are composed of thin layers of soils over unconsolidated and consolidated materials. Pipelines and other underground structures may encounter consolidated rock at higher elevations. Where rock is encountered the cost of pipeline construction will increase. Construction of underground facilities and pipelines in the alluvial deposits found at lower elevations may encounter groundwater during the wetter months.

HUMAN AND ECONOMIC ENVIRONMENT

The human and economic environment is defined as the complex interaction of the population of the planning area and its economy, land use patterns, and related activities. Projected population growth and land use patterns are used to project future system requirements.



LEGEND

— — — — CITY LIMITS (1997)

············ URBAN GROWTH BOUNDARY (1997)

FIGURE 2-1

CORVALLIS STUDY AREA

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Population

The projected population of the City of Corvallis is presented in Table 2-1. This information was provided by the Corvallis Planning Department. The 1997 population in the service area is estimated at 50,000. The estimated total population at build out in the urban growth boundary is 120,000 people.

Table 2-1. Corvallis Population Projections

	1997	Within 10 to 20 years	Within 20 to 40 years	Build out
Urban growth boundary	50,000	60,000	80,000	120,000

Land Use Planning

The City of Corvallis adopted its Comprehensive Plan in 1980 and revised the plan in 1996. The 1996 plan has been used in the preparation of this study. Land use classifications are the basis for estimating future water demand and the distribution of this demand. The Comprehensive Plan is currently being reviewed.

Residential Land Use. Residential land use is composed of the following five types of land use designation: low-density, medium-density, medium high-density, high-density, and intensive development sector. The intensive development sector land is planned for residential development in excess of six units per acre and neighborhood or community commercial development. Table 2-2 shows the range of residential units per acre for each of the residential land use designations. The typical values for dwelling units per acre were used for this study. To estimate population density, 2.3 people per living unit were assumed based on recommendations from the Corvallis Planning Department.

Commercial Land Use. Commercial land is made up of the central business district, shopping areas, linear, and professional offices. Linear commercial land is planned for auto-oriented and related commercial uses located along a major arterial street.

Industrial Land Use. Industrial land use is made up of light industrial, intensive industrial, limited industrial, and research technology centers. This area includes Hewlett-Packard which is the largest industrial account within the city.

Public/Institutional Land Use. Institutional land is that used for churches, schools, hospitals, parks, and airports. Oregon State University is the single largest institutional water customer in Corvallis.

Table 2-2. Residential Housing Densities

Land use designations	Density range units/acre	Typical density units/acre	Typical density[a] persons/acre
Low-density residential	2-6	4	9.2
Medium-density residential	6-12	9	20.7
Medium high-density residential	12-20	16	36.8
High-density residential	20+	20	46.0
Intensive development sector	6-20 residential + commercial	75% of area @ 12 and 25% commercial	20.7

[a] Value used for estimating water demand.

Projected Population Density

The City of Corvallis 1996 Comprehensive Plan Map was used to determine the amount of land within each one of the land use categories. A geographical information systems computer program was used to calculate the acreage of each land use area. Present and future populations for these sub-areas were derived by multiplying the area of residential land by the density of population per unit area. The land areas were also used to derive present and future water use by multiplying population by per capita water demand for residential use and by multiplying land use area by water demand per acre for commercial, industrial, and public/institutional users. The water demands on a per capita and per acre basis are developed in Chapter 4.

Using the existing developed residential area, average residential unit densities, and 2.3 persons for each unit, a population of 55,600 people was calculated. A similar evaluation of ultimate development population within the urban growth boundary indicates the urban growth boundary area will support 128,400 people. Higher population capacity for both existing development and ultimate buildout reflects the factor that 100 percent buildout is not achieved.

CHAPTER 4

WATER USE

The preparation of an improvement program for water supply and distribution systems requires the evaluation of past and present water use data and the projection of that data into the future. When combined with population and land use information presented in Chapter 2, water demand requirements are developed for the study area. Water demand information is generally prepared for present conditions and for several stages of future development, as addressed in comprehensive planning documents. Projections of water demand form the basis for planning and implementing water supply and distribution system improvements.

EXISTING WATER USE

The primary water use factors of importance for the Corvallis water system facility plan are annual average water use, average monthly water use, maximum daily demand, and peak hourly demand. Annual average water demand is a measure of the amount of water that must be supplied to the system from the available sources of supply. Monthly average demands show how water use is influenced by seasonal factors related to climate, irrigation, industrial production and residential use patterns. Maximum day demands are used to design pumping facilities serving areas with storage reservoirs. Maximum day demands plus fire demands are used to size storage reservoirs. Peak hour demands, plus fire demands, are used to size pipelines and to design pumping facilities serving areas without storage reservoirs. This section contains Corvallis water system data for the five years between 1992 and 1997. These data are presented to provide a complete picture of water use patterns in Corvallis.

Average Daily and Peak Daily Water Use

The average daily water use for each month for the period from July 1992 to June 1997 is shown in Tables 4-1 through 4-5. Also shown in these tables is the peak daily demand for each month and the annual average water use.

Table 4-1. Corvallis Water Use for 1992/1993

Month	Average daily demand, mg	Peak daily demand, mg
July	10.31	12.85
August	10.45	13.39
September	7.78	9.39
October	6.65	8.34
November	6.10	7.03
December	5.89	6.75
January	6.40	7.29
February	6.19	6.63
March	5.84	6.58
April	5.93	7.39
May	6.29	7.92
June	6.90	9.20
Average	7.06	----

Table 4-2. Corvallis Water Use for 1993/1994

Month	Average daily demand, mg	Peak daily demand, mg
July	8.53	10.30
August	10.26	14.87
September	9.76	11.90
October	7.29	9.62
November	6.38	9.13
December	5.89	7.58
January	5.92	7.24
February	5.89	6.72
March	5.82	6.89
April	6.13	6.87
May	7.11	8.99
June	7.82	10.65
Average	7.23	----

Table 4-3. Corvallis Water Use for 1994/1995

Month	Average daily demand, mg	Peak daily demand, mg
July	11.62	14.94
August	10.92	12.78
September	8.65	10.57
October	6.66	7.73
November	6.04	7.10
December	5.66	6.85
January	6.11	8.98
February	6.35	7.26
March	6.09	6.71
April	6.24	6.94
May	7.61	12.02
June	9.14	13.21
Average	7.59	----

Table 4-4. Corvallis Water Use for 1995/1996

Month	Average daily demand, mg	Peak daily demand, mg
July	11.37	14.65
August	10.91	14.01
September	8.81	10.47
October	6.87	7.58
November	6.49	7.41
December	6.12	6.91
January	6.48	7.42
February	6.71	7.63
March	6.30	6.89
April	6.41	6.99
May	6.62	7.76
June	9.10	10.96
Average	7.68	----

Table 4-5. Corvallis Water Use for 1996/1997

Month	Average daily demand, mg	Peak daily demand, mg
July	11.92	14.42
August	11.41	13.10
September	8.46	11.42
October	7.05	8.10
November	6.76	8.16
December	6.28	7.22
January	6.62	7.35
February	6.52	7.22
March	6.34	6.90
April	6.60	7.01
May	8.10	10.12
June	8.54	9.87
Average	7.90	-----

The average annual demand over the 5-year period was 7.49 million gallons per day (mgd). The peak monthly demand occurred in July 1996 and averaged 11.92 mgd. The lowest monthly demand occurred in December 1994 and averaged 5.66 mgd. The peak daily demand of 14.94 mgd occurred in July 1994.

Table 4-6 shows the average daily water demand for 1992 to 1997 divided by the population for those years. The average daily water demand varies from 153 gallons per capita per day (gpcd) to 160 gpcd, with an average of 157 gpcd. Note that these demands include all uses, including residential, commercial, industrial, and public/institutional.

Unit Consumption Values

An evaluation of the water use records for individual classes of users was prepared. These data are useful in planning future water demands based upon current consumption patterns and land use plans for future growth. Table 4-7 shows the water use of selected customers for the year 1992/1993. As shown in Table 4-6, the average demand in 1992/1993 was 153 gpcd, a value 2.6 percent lower than the average demand between 1992 and 1997 of 157 gpcd. Therefore, the demand values from Table 4-7 should be increased by 2.6 percent to more closely approximate average demand.

Residential Water Use. The average annual residential water use in 1992/1993 was 3.42 mgd. Since the population was approximately 46,000 people, residential demand averaged 74 gpcd. A value of 76 gpcd is calculated by increasing the value by 2.6 percent. For planning purposes, the average residential water demand of 76 gpcd is used for residential development.

Table 4-6. Corvallis Water Use for 1992 to 1997, gpcd

Year	Population	Average demand, mgd	Average demand[a], gpcd
1992-1993	46,260	7.06	153
1993-1994	46,195	7.23	157
1994-1995	47,485	7.59	160
1995-1996	49,275	7.68	156
1996-1997	50,000	7.90	158

[a] Demands include all uses, including residential, commercial, industrial, and public/institutional.

Table 4-7. Corvallis Water Use of Customer Classes for 1992/1993

Month	Demand (mgd)			
	All residential	All commercial/industrial	Hewlett-Packard	OSU
December	2.86	2.04	0.55	0.71
January	2.52	1.89	0.60	0.55
February	2.97	2.04	0.61	0.74
March	2.70	1.77	0.52	0.63
April	2.77	1.83	0.60	0.59
May	2.62	1.94	0.58	0.73
June	3.38	2.29	0.58	0.74
July	4.21	2.68	0.67	0.85
August	4.74	3.07	0.71	1.08
September	5.17	3.15	0.74	1.10
October	4.15	2.97	0.74	0.93
November	2.93	2.29	0.66	0.77
Annual average	3.42	2.33	0.63	0.79
Percent of total annual average demand	47.7	32.5	8.8	11.0

Commercial Water Use. Commercial water demand was estimated based on an average use rate of 1,000 gallons per acre per day (gpad). This planning value represents a typical value for commercial development.

Industrial Water Use. Using the total of 1992 to 1993 water used by all industrial and commercial users and assuming an average commercial water use of 1,000 gpad, the average industrial water use was 3,560 gpad. Increasing the total industrial and commercial use by 2.6 percent and assuming an average commercial use of 1,000 gpad, the average industrial water use is 3,653 gpad. Hewlett-Packard's water use was 3,778 gpad, spread out over the 167 acres the Hewlett-Packard property covers. For planning purposes, the average industrial demand of 3,750 gpad is used.

Public/Institutional Water Use. Institutional water use in Corvallis includes churches, schools, hospitals, parks, and the airport. Oregon State University, which covers approximately 532 acres, represents approximately 65 percent of the public and institutional land area within the current city limits. The water use for Oregon State University for 1992 to 1993 averaged 1,477 gpad. With an increase of 2.6 percent, the average demand is 1,515 gpad. For planning purposes, the public and institutional average demand of 1,550 gpad is used.

The water use values derived above were used to calculate existing water demand as well as future demand.

Nonrevenue Water Production

Water supply and distribution systems experience unaccounted water losses due to the combined effect of unmetered customers, leakage, inaccurate meters, system flushing, and miscellaneous hydrant uses. As a result, a portion of the water produced cannot be accounted for when the results of treatment plant production are compared to the summation of metered uses.

Nonrevenue water production for Corvallis has been determined by comparison of the total of all metered water consumption with the amount of water metered at the water treatment plants. A 5-year history of unaccounted water is shown in Table 4-8. A rate of 10 to 15 percent is considered good performance.

The city has a program to test and repair meters and all customers are metered. An audit of the system may or may not discover additional savings. As new pipes are added to the system and older pipes replaced, the loss of water through leakage may be reduced.

Table 4-8. Unaccounted for Water; 1992 to 1997

Year	Million gallons	Percent of total water production
1992/1993	157.38	6.1
1993/1994	315.42	12.0
1994/1995	248.41	9.0
1995/1996	287.09	10.2
1996/1997	177.74	6.2

Rates of Water Use

Effective planning and design of water supply, treatment and distribution facilities requires consideration of short-term water demand variations as well as average annual usage. Treatment plant design and operation is influenced by monthly and daily demands, and transmission and distribution mains, storage reservoirs and pumping stations are sized based on peak demands. Factors have been developed to convert average demands to peak demands based on water use records for the Corvallis system. These factors are discussed below.

Annual Water Demand. As shown in Tables 4-1 through 4-5, the average annual water demand varied between 7.06 and 7.90 mgd. Average annual demand for the Corvallis water system for the years 1992 to 1997 was 7.49 mgd.

Monthly Water Variations. Monthly water demand variations for the Corvallis water distribution systems are shown in Tables 4-1 through 4-5. The tables illustrate the seasonal nature of water demand in Corvallis. The monthly water use ranges from a low of 75 percent of the average annual demand, to a maximum of 153 percent of average annual demand. The maximum monthly water use averaged 149 percent of the average annual demand. This variation is mainly due to water use for irrigation during the summer months. For this study, maximum month water demand is determined by multiplying average day demand by a factor of 1.5.

Maximum Daily Demand. Maximum daily demand varies with the extremes of climate and the mix of customers using the water. Maximum daily demand is almost always on days of highest summer temperatures, when landscape irrigation and other uses peak. Table 4-9 shows that for the period between 1992 and 1997, the ratio of peak to average is approximately 2.0. This is a relatively low peak to average ratio, perhaps as a result of larger industrial demands that tend to be uniform around the year. A typical value is 2.5. For comparison, the peaking factor in Portland is approximately 2.4.

Table 4-9. Maximum Daily Demand Ratio for Corvallis; 1992 to 1997

Year	Annual average demand, mgd	Peak day demand, mgd	Ratio of maximum day to annual average demand
1992/1993	7.06	13.39	1.90
1993/1994	7.23	14.87	2.06
1994/1995	7.59	14.94	1.97
1995/1996	7.68	14.65	1.91
1996/1997	7.90	14.42	1.83
Average	7.49	14.45	1.92

For this study the maximum daily water demand was determined by multiplying average daily demand by a factor of 4.0 for areas which are predominantly residential (second and third service levels). For the combination of all users within the service area, including residential, commercial, industrial, and public/institutional users, the maximum daily water demand was determined by multiplying average daily demand by a factor of 2.0. The same value is used for projecting future water demands.

Peak Hourly Demand. Based on actual water meter readings in specific areas of the city, this study uses a peak hourly demand factor of 11.75 for residential users and by a factor of 4.6 for the combination of all users within the service area to estimate peak hourly demand.

The peaking factors used in this study are presented in Table 4-10.

Table 4-10. Peaking Factors[a]

Description	Factor
Maximum month demand	1.5
Maximum daily demand	
Residential only	4.0
Average for city	2.0
Peak hourly demand	
Residential only	11.75
Average for city	4.6

[a] The average demand multiplied by the peaking factor yields the respective peak demand.

FUTURE WATER USE

Planning of water supply and distribution systems requires projection of future water requirements, based on population forecasts, land use plans and unit water use values.

Present Water Use

Currently, the population within the urban growth boundary is approximately 50,000. The average daily water demand is 7.5 mgd and maximum daily demand is 15 mgd.

Water Demand in 10 to 20 Years

The anticipated 10 to 20-year growth is an increase of 10,000 people and a population of 60,000 within the city limits. Average daily water demands is projected to be 10 mgd with a corresponding maximum daily demand of 20 mgd.

Water Demand in 20 to 40 Years

The projected population in 20 to 40 years is 80,000 within the city limits. Average daily water demand is projected to be 13.5 mgd and the maximum daily demand 27 mgd.

Build Out Development

The projected population within the urban growth boundary at build out development is 120,000. Average daily water demands for build out development is 20 mgd and the maximum daily demand 40 mgd.

Water demands are summarized in Table 4-11.

Table 4-11. Water Demand Summary

Population inside urban growth boundary	Average daily water demands, mgd	Maximum daily water demand, mgd
50,000	7.5	15
60,000	10.0	20
80,000	13.5	27
120,000	20.0	40

CHAPTER 5

BASIS OF PLANNING

This chapter describes the standards and criteria that are used for planning of improvements to the water distribution systems. The function of this planning study is to develop reasonable approximations of the size, location, and cost of the required improvements in sufficient detail to permit evaluation of projects. Change in the location, sizing and cost of facilities should be expected as a result of the detailed analysis that is part of the predesign and design of specific improvements.

SERVICE STANDARDS

Service standards establish the minimum requirements for water quality, quantity, and pressure and determine the degree of fire protection and reliability the system should provide. In Oregon, the minimum standards are set forth in administrative rules and regulations established by the Oregon State Health Division (OSHD), which also has primary responsibility for administering the regulations established by the Environmental Protection Agency (EPA). In this study, all areas within the Corvallis urban growth boundary will receive full service to the established standards including fire protection.

Water Quantity

With regard to water quantity, all water system improvements recommended in this planning report conform to two objectives. First, the water supply system should be adequate to maintain water pressures between 40 pounds per square inch (psi) and 85 psi under normal operating conditions. Water pressures in limited areas of the distribution system can be allowed to drop to a minimum of 20 psi during emergency fire flow conditions. Secondly, the water system shall be capable of meeting customer demands without resorting to water rationing or other flow limitation measures. Water conservation measures required by the State of Oregon Uniform Plumbing Code (UPC), such as requirements for low-flow fixtures, are not considered to be undue customer limitations in this plan. Water conservation measures, as opposed to water rationing, should be encouraged by the city as part of the overall planning process.

System Reliability

In order to protect the public health, a water system must be reliable under all conditions. In most water systems, a substantial portion of the total investment is devoted to this purpose. Reliability can be increased by providing emergency storage; constructing duplicate source, treatment, pumping, transmission, and storage works; and by installing alternate power supplies. Reliability is enhanced by providing looped water distribution service and avoiding dead-end service pipelines wherever practical. Proper valving is also necessary to ensure system reliability.

Source of Supply and Treatment

Sources of supply must be developed in a manner that provides the necessary quantity of water and protection of the source and source improvements. The source of water for Corvallis is the H.D. Taylor and Rock Creek water treatment plants. Source development and treatment are discussed in the City of Corvallis H.D. Taylor Water Treatment Plant Facility Plan, dated June 1993, and in the City of Corvallis Water Plan, dated May 1981.

Distribution and Storage

The function of a water distribution system is to deliver water in adequate quantity and at acceptable pressure from the source of supply to the customers. Performance criteria for water distribution systems have been established by regulatory rules and accepted water industry standards. OSHD requires that a domestic water supply and distribution system be capable of meeting all peak daily demands and instantaneous demands during periods of maximum use without reducing the pressure below 20 psi at any service connection. This Facility Plan is based on not allowing water pressures to drop below 40 psi, except during a fire or other emergency when the minimum acceptable pressure will be 20 psi.

Distribution storage is necessary to reduce the peak loads placed on system production and transmission elements. Water storage capacity must include adequate fire reserve and is also required to meet emergency demands when the source of supply is interrupted. Storage requirements are defined later in this chapter.

Fire Protection. A water system must be capable of not only meeting domestic, commercial and industrial needs, but also of providing water in adequate quantities and at adequate pressures to meet fire-fighting requirements. In addition to the direct benefits of fire protection, a good water system has an indirect benefit of improving the fire insurance rating of the community. Capacity for fire suppression often is the controlling factor in sizing water system components and accounts for a large percentage of the total cost of the distribution system.

ISO Rating. Insurance Services Office, Inc. (ISO), has established criteria and a deficiency point system for evaluating the fire-fighting capacity of a city and for determining the community's fire insurance rating. A major portion of the rating is determined by the water supply and distribution system. The adequacy of the system is determined by comparing it to an ideal system. Forty percent of the ISO rating is determined by the condition and adequacy of the water system.

Fire Flow Requirements. The ISO has also developed a method of determining the required fire flow for structures that is based on the structures, size, materials of construction and exposure distance to adjacent buildings. A water system should be designed to deliver the required fire flow during a maximum day demand for the prescribed duration. Using ISO guidelines, the fire flow requirements within the Corvallis urban growth boundary have been established by the Corvallis Fire Department. These requirements are shown in Table 5-1. The table shows the total volume of water required for one typical fire. These values were used to size storage reservoirs. The reservoirs are sized for each service level by using the largest volume from one typical fire occurring on that service level. As an example, the largest fire flow volume which may be needed on the first service level is 2.3 MG. The reservoirs on the first service level are sized to accommodate this volume.

Fire Pressure Requirements. The fire flow standards set by the ISO require a minimum residual water pressure of 20 psi during a fire. Residual pressure, in this instance, is defined as the pressure in the main system near or within the zone where hydrant flows are occurring.

Pipeline Network. The distribution system should be designed with looped systems. The looped systems allow water to be delivered to a demand through more than one pipeline, increasing system reliability, improving water quality, and reducing headlosses. The ISO standards require that primary and secondary feeders extend throughout the system. These should be of sufficient size, considering their length and the characteristics of the area served, to deliver fire flow and consumption demands to all areas. The grid of distribution mains should consist of mains described in Table 5-2, which shows the minimum size recommended for the distribution system. If street layout or topography are not well suited to this arrangement, or dead ends and poor griding are unavoidable, the minimum main size should be determined by hydraulic evaluation.

Because this report is concerned with larger distribution pipelines, a detailed layout of minor distribution pipelines is beyond its scope. Minor distribution pipelines carry water to customers throughout the service area. Transmission and distribution pipelines should be routed through proposed new roadway right-of-ways and past planned residential developments and schools to provide the highest degree of fire protection.

Table 5-1. Fire Flow Requirements

Land-use classifications	Recommended fire flows[a]		
	Quantity, gpm	Duration, hours	Total volume for one typical fire, MG
Principal Business District	4,500	4	1.10
Minor Business Districts			
Partially or Unsprinklered Businesses	4,500	4	1.10
Fully Sprinklered Businesses	3,000	3	0.54
Schools and Institutions			
Elementary and Junior High Schools			
Typical Without Sprinkler Systems	4,000	4	1.00
Typical With Sprinkler System	3,000	3	0.54
High Schools			
Corvallis	5,500	5	1.60
Crescent Valley	5,500	5	1.60
Oregon State University			
Low Fire Hazard—1-2 stories, sprinklered, separated from nearby structures	3,000	3	0.54
Medium Fire Hazard—multi-story, sprinklered, some exposure to nearby structures	5,000	5	1.50
High Fire Hazard—multi-story, partly (or not at all) sprinklered, exposed to nearby structures	6,500	6	2.30
Hospital			
Good Samaritan	1,750	2	0.21
Industrial Areas and Tracts			
Partially or Unsprinklered Buildings	4,500	4	1.10
Fully Sprinklered Buildings	3,000	3	0.54
Hewlett-Packard	4,500	4	1.10
Residential			
Rural	1,000	2	0.12
Single-Family, Low Density	1,000	2	0.12
Single-Family, Medium Density	1,500	2	0.18
Single-Family, High Density	2,000	2	0.24
Multi-Family, High Density	3,000	3	0.54
Apartments and Dormitories	4,000	4	1.10

[a] Recommended fire flows were determined by following ISO guidelines and were reviewed by the Corvallis Fire Department in 1994.

Table 5-2. Minimum Size of Distribution Pipelines

Area[a]	Minimum diameter (inches)
Residential, low density	8
Residential, low density	8
Residential, medium density	10
Commercial	10
Industrial	12
Public use	10

[a] Guide for Determination of Required Fire Flow, Insurance Services Office, December 1974.

Valves. To isolate sections of main in the event of a break or for new construction, ISO standards require that the system be equipped with an adequate number of properly located valves. Table 5-3 presents the maximum valve spacing for long runs of pipelines that serve different functions. Connections of smaller mains in the distribution system to transmission pipelines should be valved so that the service disruption in any of the smaller mains does not require the major transmission line to be shut down. Service taps to transmission pipelines larger than 12-inches should be avoided. Within the distribution gridwork, valves should be placed on all but one leg at tees and crosses in an organized pattern that minimizes the length of pipeline shut down whenever repairs are needed.

Table 5-3. Maximum Valve Spacing Recommended by ISO

Pipeline function	Maximum spacing
Supply pipeline	1 mile
Transmission pipeline	1/4 mile
Residential distribution	800 feet
Commercial distribution	500 feet

Hydrants. All water used for public fire protection must be delivered through hydrants, except where interior sprinkling is provided. A sufficient number of hydrants must be in place to provide the required fire flows.

Table 5-4 provides the hydrant distribution for typical land use categories. Where fire flows in excess of 1,500 gallons per minute (gpm) are called for, the flows must be provided by more than one hydrant. Hydrant spacing is governed by the Uniform Fire Code.

Table 5-4. Standard Hydrant Distribution

Land use category	Maximum hydrant spacing, feet
Residential, low density	400
Residential, medium density	370
Residential, high density	325
Commercial, general	370
Industrial	300
Schools	300
Offices	350

The ISO issues a Class 9 rating if any part of a residence is located over 1,000 feet from a fire hydrant, or if any part of a business is located over 500 feet from a hydrant. The distances are measured from the hydrant to the farthest point on the outside of a structure along the fire route a fire hose must be laid. High-capacity hydrants with three outlets should be installed. To ensure proper operation and maintenance, all hydrants should be controlled by the city.

Storage Requirements. Storage within a distribution system allows the treated water source to operate at a relatively uniform rate of production, while constantly changing user demands are met. Storage also allows the system to supply fire flows, and to meet emergency demands when the water supply is interrupted. The primary functions of distribution storage are:

- Equalizing storage
- Fire-fighting reserve storage
- Emergency reserve storage

Equalizing storage permits the source, water treatment, pumping, and transmission facilities to operate at a capacity near average demand during a peak demand period, with flow in excess of supply made up from storage. This period is normally from one to five days. Equalizing storage is recommended to be about 20 percent of the total water use during the peak demand period. The equalizing storage used in this study is 20 percent of the maximum day demand.

Fire-fighting storage should be based upon the fire flow requirements shown in Table 5-1. It is assumed that fire flow requirements occur on the peak demand day. The fire-fighting demand is therefore additive to the equalizing storage.

Emergency storage provides system reliability in the event of failure of the source of supply, treatment plant, pumping stations or transmission pipelines. The emergency storage capacity is additive to the equalization storage and the fire-fighting storage. To calculate emergency storage requirements, assumptions on what measures will be taken during an emergency must be made. For this study the following assumptions were used.

- In the event that the Willamette River should become contaminated, it is estimated that it would take up to three days to allow the contamination to pass by the H.D. Taylor WTP or to modify the WTP process to treat the contaminated water.

- Immediately following the WTP shutdown, the public would be notified and water rationing measures would be adopted.

- If the shutdown were to occur during a period of peak day demand, it would take up to 12 hours for water rationing measures to be adopted, after which the demand would drop to one-half the annual average day demand for the remainder of the shutdown period.

- During the shutdown period, the Rock Creek WTP would be the sole supplier to the system. The production from the Rock Creek WTP would be a maximum of 5 million gallons per day. A booster pumping station will be required. This is more production than is accomplished currently for summer operation and increases the risk for introducing poor tasting water into the distribution system. To verify this level of production, it is recommended that the city conduct an additional study of the Rock Creek WTP production capability during summer conditions. Future emergency storage requirements may need to be adjusted to reflect the result of this evaluation.

- Response to an emergency depends on the ability of the city to reach its citizens with the necessary information. An extensive emergency curtailment plan is essential to effectively reduce water demand during an emergency.

Provision of too much storage can lead to water quality problems in the distribution system. If treated water remains too long in a storage tank, the disinfectant residual will dissipate, and bacterial regrowth can occur. To prevent this, additional chlorine can be added to reservoir contents, which has the unwanted impact of creating more disinfection byproducts.

The recommended storage capacities should provide adequate reserve without requiring further chlorination. The reservoirs must be managed to ensure that their contents are routinely turned-over in order to assure a fresh water supply. This can be accomplished by temporarily reducing water treatment plant output during periods of low demand, and drawing down each reservoir into the system. The reservoirs should be turned-over every two to three days. Normal reservoir cycling at peak demand periods will probably be sufficient to assure that this requirement is met.

Where sites are available at the proper elevation, it is desirable to build reservoirs at an elevation that will permit water to return to the system by gravity. The most common problem with upper zone service areas is providing adequate pressures in the immediate vicinity of the reservoirs. To be effective, the reservoir must allow water depth to fluctuate over its entire depth.

Pumping Stations. Pumping is required when adequate water pressure cannot be obtained by gravity and to move water to the higher pressure zones. Pumping stations have been evaluated and sized using the following guidelines. For pumping stations serving areas with storage reservoirs, the pumping stations should be capable of supplying the peak day demand with one pump out of service. For pumping stations serving areas without storage reservoirs, the pumping station should supply the peak hour demand with one service pump out of service and should have two additional pumps for serving fire flow needs. The two fire flow pumps will have a combined capacity equal to the fire flow requirements of the service area.

CHAPTER 6

WATER DISTRIBUTION SYSTEM EVALUATION

In the previous chapters of this report, the factors that influence water facilities planning for the Corvallis service area are discussed. This chapter presents the system evaluation that has been completed for upgrading the existing system and for meeting future water system demands.

EXISTING SYSTEM ANALYSIS

This section discusses the analysis of existing water system performance including the development of a CYBERNET computer model, verification of the model, existing system components, and the results of the analysis. Restrictions in the existing system storage, pumping, and distribution system are presented in this section.

Analysis

The first step in developing water system improvements is the evaluation of the existing system capacity to identify restrictions in distribution, storage, and pumping capacity. The existing system analysis includes the following steps:

- Develop computer model of the existing transmission system.
- Analyze the ability of the existing system to meet present-day and future demands.
- Identify water system deficiencies at current and projected levels of development.

The model was used to simulate average day demand and maximum water use conditions, as well as maximum day plus fire flow at selected sites within Corvallis. Assumptions used in this analysis include the following:

- Maximum allowable system pressure should not exceed 85 pounds per square inch (psi).
- Minimum allowable system pressure under average and maximum day demand conditions should not be less than 40 psi.
- Minimum system pressures at the meter within 2,000 feet of an imposed fire flow should not be less than 20 psi.

CYBERNET Computer Program

The computer program CYBERNET, developed by Haestad Methods, was used to evaluate the distribution system performance. The program was developed to analyze steady state flow and pressure conditions for pressure pipe networks. The program was written to accommodate any piping configuration and various hydraulic components such as pumps, valves, pressure regulating valves, and storage tanks. CYBERNET is also capable of carrying out an extended period simulation for considering the fill and draw cycle on storage tanks.

The CYBERNET model consists of a data file arrayed in a specific format describing the hydraulic, physical, and geographic characteristics of the distribution network. These parameters include pipe size and length, hydraulic factors, and ground elevations. The data file also describes the operating conditions including demand, reservoir levels, and pump operation. CYBERNET utilizes AutoCAD for a graphical presentation of input and output data.

Corvallis CYBERNET Model

The first step in developing the Corvallis CYBERNET model was to prepare a schematic of the pipe network. The schematic is a graphical representation of the key pipes in the system using numbered lines for pipes. Each junction node where pipes meet was also numbered. The key components in the existing system model included the following:

- All pipes 8 inches and larger in diameter.
- All existing pumping stations.
- The existing reservoirs.
- The existing Elmwood pressure reducing valve (PRV).

Model Verification

In order to verify that the computer model adequately describes the physical water system, field measurements of flow and pressure were compared with the model simulated flow and pressure data for a known set of supply, storage, and demand conditions. A field hydrant flow test was conducted in the afternoon of March 3, 1994.

The field test was performed on the first service level. Since the second and third service levels are separated from the first service level by pumping stations, it was not necessary to monitor the second and third service levels during the field test.

During the test, pressures were monitored at four locations within the distribution system. The monitoring locations were:

- 2213 Northeast Lancaster Drive
- 843 Northwest 12th Street
- 1310 Southwest Avery Park Drive
- H.D. Taylor Water Treatment Plant (WTP)

A flow of 2,500 gallons per minute (gpm) was discharged from a fire hydrant located at the north end of Lancaster Drive for a period of 30 minutes. During the test the rate of production from the Rock Creek and H.D. Taylor WTPs was monitored. The reservoir levels were also noted at the beginning and at the end of the test. Static and residual pressure readings were taken at each of the monitoring locations. A summary of the field calibration test data is presented in Table 6-1.

The model output data was also checked against output files from the model developed for the 1981 Water Plan. The field test information and the output data from the 1981 Water Plan Model was used to calibrate the CYBERNET model to reflect the actual physical condition within the water system. The calibrated model data is presented in Table 6-1. Field readings were provided by City of Corvallis staff.

Evaluation of Existing System Performance

An evaluation of the existing water distribution system was made under three conditions that stress the system capacity:

- Peak hour demand
- Maximum day demand plus fire flow
- Low demand, high water treatment plant production

The CYBERNET model was used to characterize the flow of water within the system, as well as to identify areas of high and low pressures. Problems associated with high system pressures include potential pipe failure, unacceptable household pressures and potential injury, and damage to hot water heaters, etc. Low pressures pose potential health problems and unacceptable fire pressures. A discussion of the model results for the three evaluated conditions follows.

Peak Hour Demand. The CYBERNET model showed that the existing network of distribution piping can adequately supply the peak hour demand on the system. To evaluate pressures in the second and third pressure zones, a peaking factor of 11.75 was used to simulate peak hour flow for these residential areas. Pressures in the upper pressure zone are adequate for this high demand period. To test peak hour pressures in the first pressure zone, a peaking factor of 4.6 was applied. Again, adequate pressures resulted in the model evaluation.

Table 6-1. Field Calibration Test and CYBERNET Model Data

	Water treatment plant flows (gpm)		Discharges from 2nd level pumping stations (gpm)			
	H.D. Taylor	Rock Creek	South Hills	36th and Grant	Hoover	North Hills
Field test	7,083	2,431	0	598	693	445
Model	7,102	2,392	0	601	692	446

	Flow from Hydrant at north end of Lancaster Drive (gpm)	North Hills reservoir, percent full		Baldy reservoir, percent full		Pressure at monitoring locations,[a] pounds per square inch			
		Beg	End	Beg	End	A	B	C	D
Field test	2,500	69	69.4	94.3	94.4	81/76[b]	72/70.5	80/80	73/71
Model	2,500	69	69.4	94.3	94.4	78.5/73	74/72	79.5/77.5	73.5/72

[a] Pressure monitoring locations are:
A - 2213 N.E. Lancaster Drive
B - 843 N.W. 12th Street
C - 1310 S.W. Avery Park Drive
D - H.D. Taylor WTP

[b] 81 - Pressure before test begins.
76 - Pressure measured during test.

Maximum Day Demand Plus Fire Flow. The existing system can deliver adequate fire flows to the majority of the fire hydrants with maximum day demand. Some hydrants located at the ends of dead end pipelines and which are supplied by smaller diameter pipelines have deficient fire flows. Deficiencies were discovered by ISO Commercial Risk Services, Inc., during their field testing which was performed on October 18, 1990. The computer model was used to verify these deficiencies. The model was also used to test fire flow at other locations in the system but no other deficiencies were found. ISO discovered deficiencies were verified on the first service level near Western View Intermediate School and near the airport.

A summary of the hydrants tested for ISO with potential fire flow deficiencies is shown in Table 6-2, also shown are computer model junction numbers and comments on how the deficiencies can be corrected.

Table 6-2. ISO Field Test Results (test performed October 18, 1990)

Location	Model junction number	Flow at 20 psi (gpm)		Comments
		Required	Available	
South 3rd Street and Tunison Avenue	J374	5,500	5,100	Required flow can be met by other hydrants in area.
Southwest 35th Street and Philomath Boulevard	J400	5,000	2,600	Additional piping to hydrant will produce required flow.
Southwest 26th Street and Campus Way	J157	5,000	4,800	Required flow can be met by other hydrants in area.
Philomath Avenue and Technology Loop	J480	3,500	3,100	Future pipe looping will produce required flow.
Southwest Airport Avenue and Lowe Street	J343	3,500	2,100	Future pipe looping will produce required flow.
Southwest Airport Place and Runway	J342	4,500	1,400	Future pipe looping will produce required flow.

Low Demand, High Water Treatment Plant Production. The existing system was modeled to reflect a fill period for the North Hills first level reservoir. System demand was set at 20 percent of average day demand, the second service level pumping stations were turned off, the altitude valve on the pipeline to the Baldy reservoir was closed, the water level in the North Hills first level reservoir was set at 96 percent full, production from the Rock Creek WTP was set at 2 million gallons per day (mgd), and production from the H.D. Taylor WTP was set at 14 mgd. The results of the model run showed pressure exceeding 95 psi in the area located southeast of downtown Corvallis. Because the system has low demand and the Baldy reservoir is full (pipe closed), all water which is not supplying a demand will flow into the North Hills first level reservoir. The increased amount of water flowing between the H.D. Taylor WTP and the reservoir will increase the amount of frictional headloss in the piping and an increase in pressure will be required at the H.D. Taylor WTP to pump the water to the reservoir. This confirms the problem identified by the operators when they try to fill the reservoir during periods of low demand. The modeling showed that water preferentially moved to the Baldy Reservoir and the high system pressure started after the altitude valve at Baldy Reservoir closed.

By adding a north/south transmission line between the H.D. Taylor WTP and the reservoir, the high pressure problems will be corrected. A 30-inch and 36-inch transmission line will be required.

Pumping Stations

The existing pumping stations adequately supply the existing demands on the system and also provide adequate backup capacity. The South Hills third level pumping station was recently upgraded with an additional fire flow pump. Several of the existing pumping stations will need to be upgraded to meet future demands. The proposed upgrades are discussed later under proposed system improvements.

Storage

The existing system meets the existing storage capacity requirements for equalization, fire protection, and emergency storage.

Service Pressure Levels

Currently the third service pressure level extends from elevation 410 fmsl to elevation 560 fmsl. The Timberhill reservoir has an overflow elevation of 660 fmsl which creates a static head difference between the overflow elevation and lowest elevation on the third service level of 250 feet (108 pounds per square inch [psi]). Therefore, the maximum pressure of 85 psi will be exceeded at elevations below 464 fmsl, when the reservoir is full. The computer model verifies the high pressure in the area served by the Timberhill reservoir.

At locations where system pressures are above acceptable levels, pressure reducing valves have been placed on individual service lines.

PROPOSED SYSTEM IMPROVEMENTS

Improvements to the water distribution system need to be made in order to insure the system can meet the existing demands. These improvements include distribution network upgrades to adequately supply existing fire hydrants and to decrease the filling time of the North Hills first level reservoirs, while keeping system pressures at acceptable levels.

Future growth within the urban growth boundary will also necessitate improvements to the system. System improvements are sized based on the requirements of the system at build out. The CYBERNET model was used to characterize the flow of water in the system, to identify areas of high and low pressures, and to provide information for sizing pipelines. A model of the proposed system at build out was run using maximum hour demand as well as maximum day demand plus fire flow and low demand with high water treatment plant production.

Figure A in Appendix B shows the existing water distribution system as well as the proposed improvements. The figure is the graphical presentation of the system generated by CYBERNET.

Distribution Network

Proposed distribution piping network improvements include new pipelines to serve all future demands within the urban growth boundary. Besides supplying all users within the urban growth boundary, the proposed network provides the following major components:

1. An intertie between the North Hills third level pumping station and the Timberhill reservoir. This will allow the Timberhill reservoir to be supplied by the North Hills third level pumping station as well as by the Queens View pumping station.

2. The addition of a 36-inch-diameter north/south transmission line from the H.D. Taylor WTP to the North Hills first level reservoir. This will allow the reservoir to fill at a faster rate during low demand periods, while decreasing head losses through the system and maintaining system pressures below 85 psi. The transmission line will allow up to 35 mgd to be pumped into the system from the H.D. Taylor WTP.

Storage

Total storage requirements are obtained by adding together the requirements for equalizing, fire-fighting reserve, and emergency reserve storage. As stated in Chapter 5, the recommended equalization storage volume is equal to 20 percent of the maximum day demand. Fire-fighting reserve storage requirements have been determined using the Insurance Services Office, Inc., (ISO) guidelines and were presented in Table 5-1. The fire storage required for each area served represents the largest volume which a single fire in the area may demand. Applying the assumptions presented in Chapter 5, the emergency storage requirements at build out will be 30 MG.

The major proposed storage improvements are as follows:

1. To supply the future demands from the Crescent Valley second service level, it is proposed that a 1.0 MG reservoir be built at the north end of the valley. The reservoir will allow for minimizing the diameter of the distribution piping between the North Hills and the north end of Crescent Valley.

2. A 0.5 MG reservoir is proposed to be located at the northwest side of Crescent Valley. It will supply the demands of the Crescent Valley third service level.

3. A 1.0 MG reservoir is proposed for the Mulky Creek area. The proposed location is near Bald Hill. The reservoir will supply demands from the second service level and will minimize the distribution piping diameters between the Woodland Park area and the Mulky Creek area.

4. An additional 17.2 MG of emergency storage is required. The proposed location for the additional storage is 8.6 MG at the Baldy reservoir site and 8.6 MG at the North Hills first level reservoir site. Siting for first level reservoirs needs to be refined and additional sites may be required. This improvement will increase the total emergency storage to 30 MG at build out. The proposed storage sites are preliminary. Additional site location studies are required. In order to maintain water quality, the reservoirs need to be managed to continuously utilize and replace (turnover) the entire stored volume. This can be achieved by designing the system such that the reservoir becomes part of the transmission system. All water would be transmitted to the reservoir before it is transmitted to the users. This will provide a continuous flow of water through the reservoir. Using baffles in the reservoir to force the water through channels will insure that all water entering the reservoir will flow out in a timely manner and stagnant areas will be avoided.

5. An emergency curtailment plan needs to be developed which will allow the city to respond to a loss of water supply by informing the public on the emergency measures that must be followed.

6. An evaluation is needed of the Rock Creek Water Treatment Plant capacity for water production during an emergency. High summer production has a higher risk for introducing bad tasting water into the distribution system. A booster pumping station along the Rock Creek supply line will also be required.

Table 6-3 presents the proposed storage improvements. All of the emergency storage is recommended for the first level. This reduces pumping costs and does not compromise system operation since the pumping stations for the upper levels will have the needed capacity to serve the second and third level.

Pumping Stations

In order to supply the future water demands at build out, it is recommended that six existing pumping stations be upgraded and five new pumping stations be built. Table 6-4 presents the proposed improvements.

The pumping stations which are supplying areas with storage reservoirs are sized to provide the maximum day demand with one pump out of service.

Table 6-3. Proposed Storage Improvements at Build Out

Service level	Area served	Maximum day demand (mgd)[a]	Storage required to serve area			Existing reservoirs serving area		Proposed additional reservoirs to serve area		Storage available for emergency (mg)
			Equalizing (mg)[b]	Fire (mg)[c]	Total (mg)	Name	Capacity (mg)	Name	Capacity (mg)	
1st	First service level	25.00	5.00	2.3	7.3	Baldy North Hills 1st LVL TOTAL:	7.5 10.0 17.5	Baldy (expansion) North Hills 1st LVL (expansion)	8.6 8.6 17.2	27.4
2nd	Woodland, Sylvan, North Hills	4.2	0.9	0.6	1.5	North Hills 2nd LVL Woodland 2nd LVL Sylvan TOTAL:	2.5 1.2 0.2 4.0	None		2.5
	Crescent Valley	2.5	0.5	0.6	1.1	North Hills 2nd LVL Woodland 2nd LVL Sylvan		Crescent Valley 2nd LVL	1.0[d]	-0.1
	Mulky Creek	3.5	0.7	0.6	1.3	North Hills 2nd LVL Woodland 2nd LVL Sylvan		Baldy 2nd LVL	1.0[e]	-0.3
	South Hills	0.5	0.1	0.2	0.4	South Hills	0.5	None		0.2
3rd	Crescent Valley	1.0	0.2	0.2	0.4	None		Crescent Valley 3rd LVL	0.5	0.1
	Timberhill, Skyline West, North Hills, Woodland Park	3.3	0.7	0.2	0.9	Timberhill Woodland 3rd LVL TOTAL:	1.0 0.1 1.1	None		0.2
TOTAL		40.0	8.0	4.70	12.7		23.1		19.7	30.0

[a] Max day demand equals 1.54 times average day demand for the first service level and equals 4.0 times average day demand for residential areas (2nd and 3rd service levels).
[b] Equalizing storage equals 0.20 times maximum day demand.
[c] Refer to Table 5-1 for fire storage requirements. The fire storage required represents the largest volume which a single fire in the area served may demand.
[d] Assuming 0.10 mg of storage requirements will be met by excess capacity of North Hills 2nd LVL, Woodland 2nd LVL, and Sylvan reservoirs.
[e] Assuming 0.3 mg of storage requirements will be met by excess capacity of North Hills, 2nd LVL, Woodland 2nd LVL, and Sylvan reservoirs.

Table 6-4. Proposed Pumping Station Improvements at Build Out

Service level	Area served	Pumping demands at build out: Demand[a] (gpm)	Fire	Total	Existing pumping stations	Pump capacity (gpm)	Proposed new pumping stations and existing station expansion	Pump capacity (gpm)
1st	Booster Pumping Station along the Rock Creek supply line	3,472[b]	---	3,472	---	---	Rock Creek	1,750
								1,750
								1,750
2nd	North Hills, Woodland Park, Sylvan Acres, Crescent Valley, Mulky Creek	10,024	---	10,024	North Hills 2nd LVL	500	North Hills 2nd LVL	2,150
						900		2,150
						4000		2,150
					Hoover	750	Hoover	750
						750		750
								750
					38th and Grant	300	38th and Grant	300[d]
						300		300[d]
							Baldy 2nd LVL	2,150
								2,150
								2,150
	Lewisburg[c]	300	1,000	1,300	---	---	Lewisburg	300
								300
								500
								500
	South Hills	368	---	368	South Hills 2nd LVL	225	South Hills 2nd LVL	225
						225		225
								150
	Southwest[c]	175	1,000	1,175	---	---	Southwest	175
								175
								500
								500
	West Hills[c]	250	1,000	1,250	---	---	West Hills	250
								250
								500
								500
3rd	Timberhill, North Hills, Skyline West	2,232	---	2,232	Queens View	790	Queens View	790
						790		790
								790
					North Hills 3rd LVL	350	North Hills 3rd LVL	350
						350		350
						110		350
					Woodland Park	750	Woodland Park	750[d]
					Witham Hill	500	Witham Hill	500[d]
	South Hills[c]	175	1,000	1,175	South Hills 3rd LVL	150	South Hills 3rd LVL	175
						500		175
						500		500
								500
	Crescent Valley	676	---	676	---	---	Crescent Valley	350
								350
								350
	Sylvan Acres[c]	47	---[e]	47	Sylvan Acres	50	Sylvan Acres	50[d]
						110		110[d]

[a] For areas served with storage available, the demand equals maximum day residential demand (4.0 times average day demand). For areas served with no storage available, the demand equals peak hour residential demand (11.75 times average day demand).
[b] The Rock Creek booster pumping station is sized to match the capacity of the Rock Creek WTP (5 mgd).
[c] No storage available.
[d] [redacted]ansio[redacted]red.
[e]

The pumping stations which serve areas which do not have storage reservoirs are sized to provide the peak hour demand with one pump out of service. These areas have a low density, single family residential land use classification with a recommended fire flow of 1,000 gpm. Two additional 500 gpm pumps to supply fire flow requirements are included.

The area served by the Witham Hill pumping station will continue to be served by the Woodland Park third level pumping station and by the Timberhill third service level, through the Elmwood PRV. The area served by the 36th and Grant pumping station will continue to be served by the Hoover and North Hills second level pumping stations.

For the full 5 mgd capacity of the Rock Creek WTP to be realized, a booster pumping station on the Rock Creek supply line will be required. This station will boost the pressure at an intermediate point along the line, increasing the line capacity to 5 mgd. Predesign of this system will be required to define the requirements for these improvements.

Figure 6-1 shows the proposed storage and pumping station improvements.

Pressure Reducing Valves

The existing Elmwood PRV functions well to provide flow from the Timberhill third service level to the Woodland Park third service level and should be maintained in the future.

Another location in the system where a PRV may be warranted is between the North Hills second service level and the Crescent Valley High School area, which is a part of first service level. The ground elevation near the Crescent Valley High School is approximately 269 fmsl, whereas the overflow elevation of the North Hills second level reservoir is 496 fmsl. This represents a static pressure difference of 98 psi, and is the highest pressure which may be experienced at the High School if there were no headloss through the distribution piping. The CYBERNET model shows that due to headloss through the distribution piping, the pressure at the High School will stay below 85 psi at build out. Currently, the maximum pressure will exceed 85 psi. In the future, it is proposed that the Crescent Valley High School area be connected to distribution piping from the first service level, and at that time, disconnected from the second service level. This will reduce headloss from the second to first service levels, and therefore, reduce pumping costs.

The current third level service area extends from elevation 410 fmsl to elevation 560 fmsl. If a minimum pressure of 40 psi is maintained at elevation 560 fmsl, the pressure at elevation 410 will be 105 psi.

In the future, areas which are at the lower end of the third service level and which experience system pressures above acceptable levels will have pressure reducing valves placed on individual service lines.

PHASES OF PROPOSED IMPROVEMENTS

The proposed improvements to meet the demands at build out have been broken down into phases in which they will be constructed. These phases are shown in Table 6-5. The timing and location of improvements needed to serve future growth is highly dependent on where growth actually occurs.

Table 6-5. Phases of Construction for Proposed Improvements

Phase	Population served (people)
I	50,000
II	50,000 to 60,000
III	60,000 to 80,000
IV	80,000 to 120,000

The following paragraphs describe the proposed construction during these phases. Estimated costs for the proposed improvement phases are presented in Chapter 7.

Phase I

The improvements for Phase I include upgrading the system to provide fire flows during maximum day demands. This will be achieved by: adding pipelines in the airport area, to loop dead end lines, and adding a pipeline serving a hydrant in the vicinity of the Western View Intermediate School.

Also proposed for Phase I is a portion of the new 30-inch-diameter north/south transmission pipeline between the H.D. Taylor WTP and the North Hills first level reservoir. This will eliminate high system pressures when filling the reservoir during periods of low system demand.

Additional storage required to meet the emergency storage needs presently and in the future are shown in Table 6-6. The emergency storage volumes required are based on the assumptions that the H.D. Taylor WTP could be out of service for up to three days. During this time the city-wide water use would be the maximum day demand for the first half day and after public notification it would drop to one-half the average day demand for the remaining two and one-half days. The only source of water to the city would be from the Rock Creek WTP with an existing delivery capacity of 3.5 mgd and a future capacity of 5 mgd.

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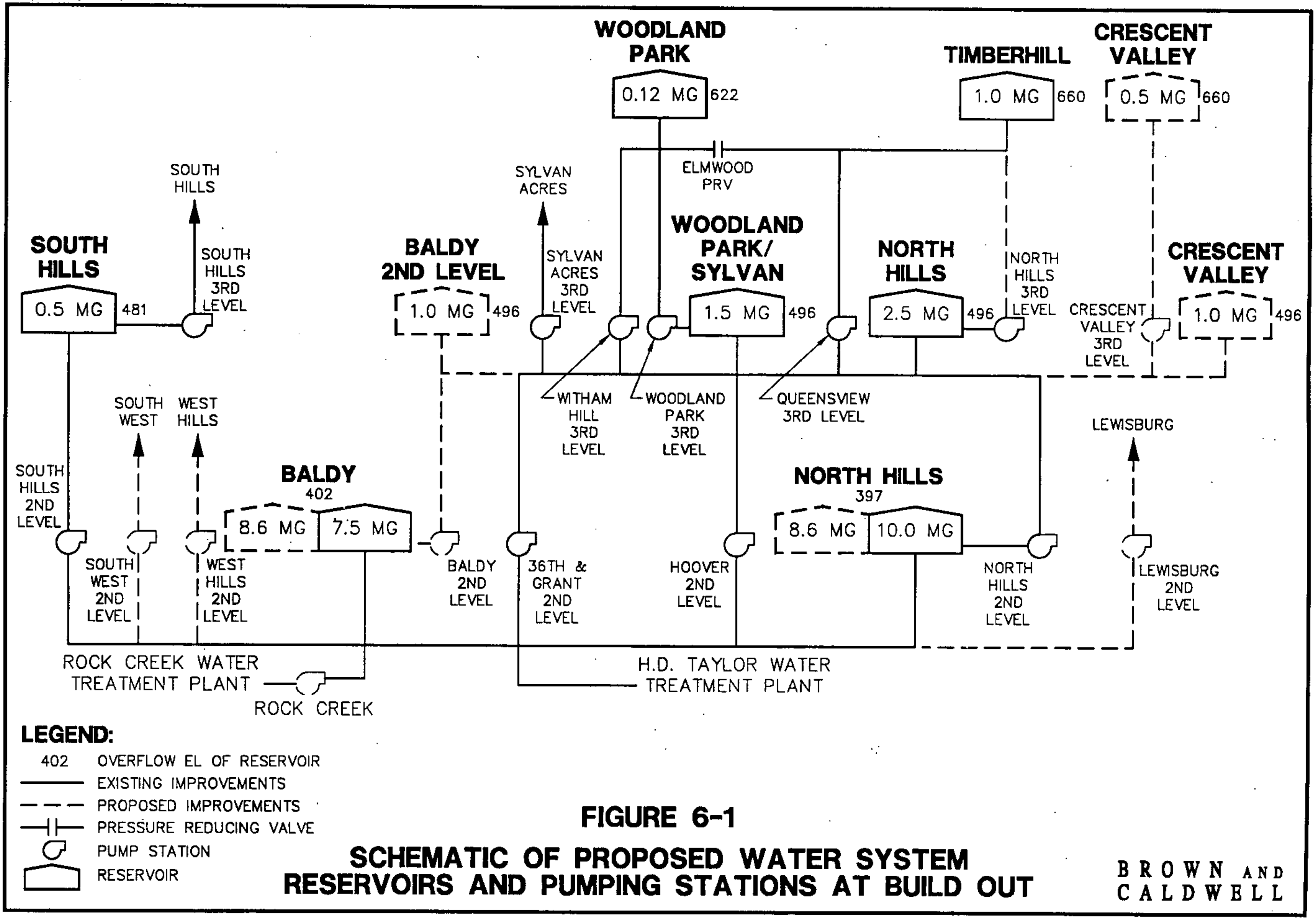


FIGURE 6-1

SCHEMATIC OF PROPOSED WATER SYSTEM RESERVOIRS AND PUMPING STATIONS AT BUILD OUT

BROWN AND CALDWELL

Table 6-6. Emergency Storage Requirements

Population	Storage required (MG)				Storage available (MG)	Additional emergency storage required[b] (MG)
	Equalizing	Fire	Emergency[a]	Total		
50,000	3.0	3.3	4.9	11.2	23.1	0
60,000	4.0	3.3	10.5	17.8	23.1	0
80,000	5.4	3.9	18.4	27.7	24.1[c]	3.6
120,000	8.0	4.7	30.00	42.7	25.6[d]	17.2

[a] Assuming Rock Creek supplies 3.5 mgd in Phases I, II, and III and 5 mgd in Phase IV (with the addition of the Rock Creek booster pumping station).
[b] Volume equals total storage required minus storage available. Required volume is additional to the existing volume (1997).
[c] Assuming that the proposed 1.0 MG Baldy 2nd LVL reservoir is built.
[d] Assuming that the proposed 1.0 MG Crescent Valley 2nd LVL reservoir and the 0.50 MG Crescent Valley 3rd LVL reservoirs are built.

Under normal demand conditions, the city would be out of water within a day. Thus, the need to notify all users of the emergency will be critical. Furthermore, a prolonged disruption of the Willamette River source would cause a severe water shortage. Upgrading the capacity of the Rock Creek supply line should be considered as an early project.

It is recommended that additional emergency storage be built during two phases as follows:

Phase I	No expansion
Phase II	No expansion
Phase III	8.6 MG at North Hills reservoir site
Phase IV	8.6 MG at Baldy reservoir site
Total	17.2 MG

Phase II

Phase II includes improvements which will bring full service to the first and second service levels in the southwest part of the city within the urban growth boundary, as well as to the second and third service levels in the Timberhill area and the first service level between the North Hills and Crescent Valley High School. These areas are expected to have the most potential for growth in the next 10 to 20 years.

The proposed construction includes the 1 MG Baldy second level reservoir, expansions of the North Hills second and third service level pumping stations, Hoover pumping station, and Queens View pumping station; Baldy second level, Southwest, and West Hills pumping stations and distribution pipelines.

Also recommended for Phase II is the completion of the 30-inch and 36-inch-diameter north/south transmission pipeline between the H.D. Taylor WTP and the North Hills first level reservoirs.

Phase III

Improvements under Phase III include adding 8.6 MG of emergency storage capacity at the North Hills first level reservoir site and adding distribution piping to the second service level in the Mulky Creek Area, southwest of Sylvan Acres, and the Crescent Valley area; and to the first service level in the southeast part of the city. These are the expected areas of new growth in 20 to 40 years.

Phase IV

Phase IV includes improvements which will provide service to the first, second, and third service levels north of Crescent Valley and which will tie the Mulky Creek second service level together with the Edgewood and Sylvan Acres second service level. These are the expected areas of new growth when the population is between 80,000 and 120,000.

The Phase IV improvements include additional emergency storage of 8.6 MG at the Baldy reservoir site, the 1.0 MG Crescent Valley second service level reservoir, the 0.5 MG Crescent Valley third service level reservoir, the Lewisburg second service level pumping station, the Crescent Valley third service level pumping station, the Rock Creek booster pumping station, and distribution piping.

CHAPTER 7

COST OF PROPOSED SYSTEM IMPROVEMENTS

This chapter details how project costs are derived and presents a breakdown of the estimated costs for the proposed system improvements.

COST DATA

In every engineering study concerned with water system development it is necessary to make estimates of construction costs for the proposed works. The basic cost data must be obtained or developed for each type of construction involved in the plan in sufficient detail to permit determination of total project cost. Cost data used in this report were derived from actual projects designed by Brown and Caldwell, and from professional and construction publications. The cost data for water main construction correlates with the costs of pipeline projects which have recently been built within the City of Corvallis.

Precision of Cost Estimates

The precision of a cost estimate is a function of the detail with which the proposed improvements have been developed and the techniques used in preparing the actual estimate. An order-of-magnitude estimate, as defined by the American Association of Cost Engineers, is appropriate for the comparison of alternatives. An order-of-magnitude estimate is approximated and is prepared without detailed engineering data. Techniques such as cost capacity curves, scale-up or scale-down factors, and ratios are used in developing such an estimate. An order-of-magnitude cost estimate has an expected accuracy of +50 percent to -30 percent. All estimates presented in this report are order-of-magnitude estimates.

Basis of Cost Estimates

In considering the cost estimates, it is important to realize that changes during final design, as well as future changes in the cost of materials, labor and equipment, will cause comparable changes in the cost estimates presented in this report. Decisions made on the basis of these current cost estimates will remain valid, however, since the relative economy of alternative projects will change only slightly in response to changes in specific goods or services.

A good indicator of changes in construction costs is the Engineering News-Record (ENR) Construction Cost Index. Cost data presented in this report are based on an ENR Construction Cost Index for Seattle of 6,747. This is as reported in the July 6, 1998, issue of Engineering News-Record. Cost data can be adjusted to any time in the past or future by applying a ratio of the then-prevailing ENR index for Seattle to the index of 6,747.

Capital Cost

The total capital investment necessary to complete a project consists of expenditures for land acquisition, construction, all required engineering services, contingencies, and such overhead items as legal and administrative services and financing. The various components of capital cost are discussed in the following sections.

Land Acquisition. In most locations, constructing water distribution mains will not require the significant acquisition of privately-owned land because pipeline routes follow public roads and easements. Where private land is being developed, the developer should be required to create the necessary easements and conveyances for construction of water mains and other utilities. The costs of land acquisition for water mains was not included in estimates, and is not considered to be a significant percentage of project costs. Conversely, land requirements for reservoirs, pumping stations and treatment plants can be significant. Land requirements for reservoirs are estimated to be 0.5 acres per million gallons of storage capacity. Land requirements for pumping stations are estimated to be 0.5 acre per each installation. For this study, land costs were estimated to be $143,000 per acre.

While this plan assumes first level reservoirs will be located on existing sites, the siting needs to be refined and additional sites may be required.

Construction Cost. Construction costs include the materials, labor, and services necessary to build the proposed project. Prices used in this study were obtained from a review of pertinent sources of reliable construction cost information. Construction cost data presented in this report are not intended to represent the lowest possible cost of completing the work. Instead, the costs represent the median costs that would result from responsible bids received from competent local contractors given a moderate amount of competition in the vicinity. Actual project costs can vary substantially based on the time of year that the work is bid, and the amount of competition for the work as a factor of the local economy.

Distribution system pumping station costs are determined not only by the type of pumps, station capacity and head and station layout, but by the architectural design and landscaping provided. Estimated average construction costs for distribution pumping stations are listed in Table 7-1. The cost data are based on enclosed pumping stations with architectural and landscaping finishes suitable for residential areas. The costs include installation of one standby pumping unit. The costs include auxiliary power generation equipment and instrumentation.

Table 7-1. Construction Costs for Distribution Pumping Stations

Pumping station capacity, mgd[a]	Construction cost[b], $1,000[c]
1	161
2	194
5	281
10	394
25	663

[a] Costs for capacities not shown may be interpolated.
[b] Cost do not include engineering, overhead, and contingency.
[c] Costs based on ENR CCI for Seattle of 6,747.

The estimated costs for water storage reservoirs, are provided in Table 7-2, and include spread footing foundations, site preparation, inlet and outlet piping with appropriate controls, and reservoir overflow provisions. The estimates assume that the tanks are above-grade and that rock excavation is not required.

Table 7-2. Construction Costs for Storage Reservoirs

Tank volume, million gallons[a]	Construction cost[b] concrete reservoirs, $1,000[c,d]
0.5	562
1	770
2	1,157
5	2,193
10	3,855
15	5,450
20	6,847

[a] Costs for volumes not shown may be interpolated.
[b] Costs do not include engineering, overhead, and contingency.
[c] Costs are based on ENR CCI for Seattle of 6,747.
[d] Costs are based on prestressed concrete tank design.

The estimated costs for water mains are listed in Table 7-3 on the basis of cost per lineal foot of installed pipe. The unit cost assumes the use of cement-mortar lined ductile iron pipe. Estimated costs are shown for two general categories of water main construction, new construction, and installation beneath existing asphalt-paved streets. Pipeline costs include right-of-way preparation, trench excavation, placing and jointing pipe, installing valves and fittings, placing imported initial backfill, and placing subsequent backfill. The subsequent backfill was assumed to be imported or select granular material for street and shoulder construction, and native for open areas. Costs for surface restoration and relocation or replacement of conflicting utilities are included. Pipeline costs also include the costs of valves, thrust blocks and fire hydrants. The cost for new or reconnected house services from the pipeline to the property line, for average suburban residential development densities, is included in the costs for pipelines up to 18-inches in diameter. These costs are not included for larger diameter pipelines, which serve mainly as transmission lines. The unit costs do not include engineering, overhead, and contingencies. These costs increase if rock is encountered. The cost of rock excavation cannot be accurately estimated due to local rock conditions, but rock excavations can add 50 to 200 percent or more, to the per linear foot cost of pipeline installation. For this study, the costs of proposed pipelines within rocky areas are estimated to be those given in Table 7-3 multiplied by a factor of 2.

Table 7-3. Construction Costs for Water Mains

Pipeline diameter, inches	Construction cost,[a,b] dollars per lineal foot new construction	Construction cost including pavement replacement
8	52	66
10	60	74
12	74	86
14	77	96
16	81	102
18	85	112
20	96	121
24	114	135
30	148	186
36	189	233
42	223	269

[a] Costs are based on ENR CCI for Seattle of 6,747.
[b] Cost do not include engineering, overhead, and contingency.

Costs for water distribution lines up to 12 inches in diameter assume that fire hydrants will be provided every 400 feet. The costs assume that the end of each pipeline would be provided with a cross and three valves for future extension. Outlets for future connections would be provided with a valve.

Engineering, Overhead, and Contingencies. In addition to the base construction cost, project cost estimates also must include costs associated with engineering, overhead, and contingencies (E/O/C). The sum of construction cost and E/O/C is the total project cost. Engineering services may include all of the following: preliminary investigations and reports, site and route surveys, foundation explorations, preparation of drawings and specifications, general supervision of construction, detailed inspection, construction surveying and staking, sampling and testing of materials, start-up services, and operations manuals for complex structures. Engineering costs vary based on project complexity and size, but generally range between 12 and 20 percent of construction cost. In this report, 15 percent is used for estimating purposes.

Overhead charges cover such items as legal fees, financing expenses, administrative costs, and interest during construction of a project. Overhead costs are also variable, but a typical value of 5 percent is used in this report for estimating purposes.

Contingency factors are included in a project cost estimate to allow for unforeseen circumstances which may occur during the design and construction of a project. Such conditions may include differing site geology, variations in final quantities, and unforeseen equipment requirements. Normal contingency factors at the planning stage range from 15 percent to 25 percent, with 15 percent used in this study.

The capital costs used in this report include a total markup for engineering, overhead, and contingencies of 35 percent.

COST OF PROPOSED IMPROVEMENTS

Tables 7-4 through 7-7 present the estimated capital costs for the proposed improvements to the Corvallis water distribution system, phases I through IV. Refer to Chapter 6 for a discussion of the proposed improvements phases. These improvements will meet the needs within the Corvallis area growth boundary through each phase.

The estimated capital cost for pipelines are shown in greater detail in Appendix A.

Table 7-4. Estimated Cost of Proposed Improvements for Phase I[a]

Proposed improvements	Capital cost[b] ($1,000)
Pipeline Projects[c]	
Airport	376
North Hills	3,328
Timian Street	61
35th Street	100
Subtotal of pipelines	3,865
Total capital cost	3,865

[a] Provides service to the existing population (50,000 people).

[b] Capital costs include construction cost plus 35 percent for engineering, overhead, and contingency.

[c] Refer to Appendix A for the capital cost for each proposed pipeline.

Table 7-5. Estimated Cost of Proposed Improvements for Phase II[a]

Proposed improvements	Capital cost[b] ($1,000)
Reservoirs	
Second service level	
Baldy (1.0 MG)	1,138
Pumping stations	
Second service level	
Baldy	512
Hoover expansion	94
North Hills expansion	311
South Hills	39
Southwest	318
West Hills	338
Third service level	
Queens View expansion	94
Subtotal of pumping stations	1,706
Pipeline Projects[c]	
Baldy Supply	2,757
Conifer	1,720
Eastside	1,629
High School	2,561
Mary's River Crossing	807
North Hills Loop	2,782
Skyline West	539
Southwest Loop	1,130
Wake Robin	2,054
West Hills	2,690
45th Street	647
53rd Street	865
Subtotal of pipelines	20,181
Total capital cost	23,025

[a] Provides service to a population between 50,000 and 60,000 people.
[b] Capital costs include construction cost plus 35 percent for engineering, overhead, and contingency.
[c] Refer to Appendix A for the capital cost for each proposed pipeline.

Table 7-6. Estimated Cost of Proposed Improvements for Phase III[a]

Proposed improvements	Capital cost[b] ($1,000)
Reservoirs	
First service level	
North Hills expansion (8.6 MG)	4,577
Pipeline Projects[c]	
Airport	2,106
Crescent Valley	1,166
Crystal Lake	3,075
Mary's Creek Crossing	1,781
Sylvan Acres	426
Weltzin/Ingalls Loop	1,925
West Hills	1,930
Subtotal of pipelines	12,409
Total capital cost	16,986

[a] Provides service to a population between 60,000 and 80,000 people.

[b] Capital costs include construction cost plus 35 percent for engineering, overhead, and contingency.

[c] Refer to Appendix A for the capital cost for each proposed pipeline.

Table 7-7. Estimated Cost of Proposed Improvements for Phase IV[a]

Proposed improvements	Capital cost[b] ($1,000)
Reservoirs	
First service level	
Baldy expansion (8.6 MG)	4,577
Second service level	
Crescent Valley (1.0 MG)	1,138
Third service level	
Crescent Valley (0.5 MG)	807
Subtotal of reservoirs	6,522
Pumping stations	
First service level	
Rock Creek	477
Second service level	
Lewisburg	351
Third service level	
Crescent Valley	316
Subtotal of pumping stations	1,144
Pipeline Projects[c]	
Crescent Valley	2,578
High School	2,045
Lewisburg	1,793
Upper Crescent Valley	1,196
West Hills	2,095
Subtotal of pipelines	9,707
Total capital cost	17,373

[a] Provides service to a population between 80,000 and 120,000 people.
[b] Capital costs include construction cost plus 35 percent for engineering, overhead, and contingency.
[c] Refer to Appendix A for the capital cost for each proposed pipeline.

CHAPTER 3

EXISTING WATER SYSTEM

The Corvallis water system serves the area within the present city limits, but also serves some properties outside the city limits such as the airport to the south and Crescent Valley High School to the north. The system is composed of two water treatment plants (WTPs), ten booster pumping stations, nine reservoirs, and one pressure reducing valve. Figure 3-1 shows the location of the system's major components. Water transmission and distribution piping, twelve inches in diameter and larger, are shown.

SOURCE OF SUPPLY

The Corvallis water system receives its water from two sources: the Rock Creek WTP, located west of the City of Philomath, and the H.D. Taylor WTP, located along the Willamette River in southeast Corvallis.

The Rock Creek WTP was built in 1956 and is supplied from Rock Creek and Griffith Creek, tributaries of the Mary's River. Surface water is collected and flows by gravity through a system of pipes to the water treatment plant located at the lower end of the Rock Creek watershed. The plant is a conventional water filtration treatment plant and has a capacity of 5 million gallons per day (mgd). The treated water flows by gravity through a series of 16- and 20-inch-diameter transmission pipelines to the City of Corvallis. The hydraulic capacity of the transmission pipelines is limited to 3.5 mgd.

The H.D. Taylor WTP is supplied by the Willamette River. Water from the river is pumped to the plant where it is filtered, chlorinated, fluoridated, and subsequently pumped with high service pumps into the water distribution system. The plant was constructed in 1949. Over the years it has undergone several expansions. The most recent expansion of the Taylor WTP was completed in 1997. These upgrades provide a maximum plant production capacity of 21 mgd. The *H.D. Taylor Water Treatment Plant Facilities Plan* was completed in June 1993.

DISTRIBUTION AND STORAGE SYSTEM

Pipeline Network

The Corvallis water transmission and distribution system consists of approximately 200 miles of existing pipeline. The pipelines are cast iron or ductile iron and range in size from 2 inch to 30 inch diameter.

Service Pressures

There are currently three service pressure levels in the Corvallis water distribution system. The first level serves the lower pressure zone between elevations 210 and 290 feet above mean sea level (fmsl). The second level serves the middle pressure zone between elevations 290 and 410 fmsl. The third service level serves areas of elevations between 410 and 560 fmsl. Figure A included in Appendix B shows the location of the service levels.

Fire Protection

Fire protection in Corvallis is provided by the City of Corvallis Fire Department. The Insurance Services Office, Inc., (ISO) has established guidelines for determining the recommended fire flows for various land use classifications. Using these guidelines, fire flow requirements within the Corvallis urban growth boundary have been established by the Corvallis Fire Department. The ISO has developed a rating system for municipal fire protection in which water supply, fire department, fire service communications, and fire safety control are graded and given a classification based on system deficiency. Classes 1 through 10 have been developed, with Class 1 being the least deficient and Class 10 being the most deficient. The ISO has given the City of Corvallis a fire insurance rating of Class 2. Fire protection requirements are further addressed in Chapter 5.

Reservoirs

The nine reservoirs within the Corvallis water distribution system provide a total of 23.13 million gallons of treated water storage. Information on the reservoirs is shown in Table 3-1.

Pumping Stations

There are ten booster pumping stations in the Corvallis water distribution system that pump water from the first service level to the second service level and from the second service level to the third service level. Information on the booster pumping stations is summarized in Table 3-2.

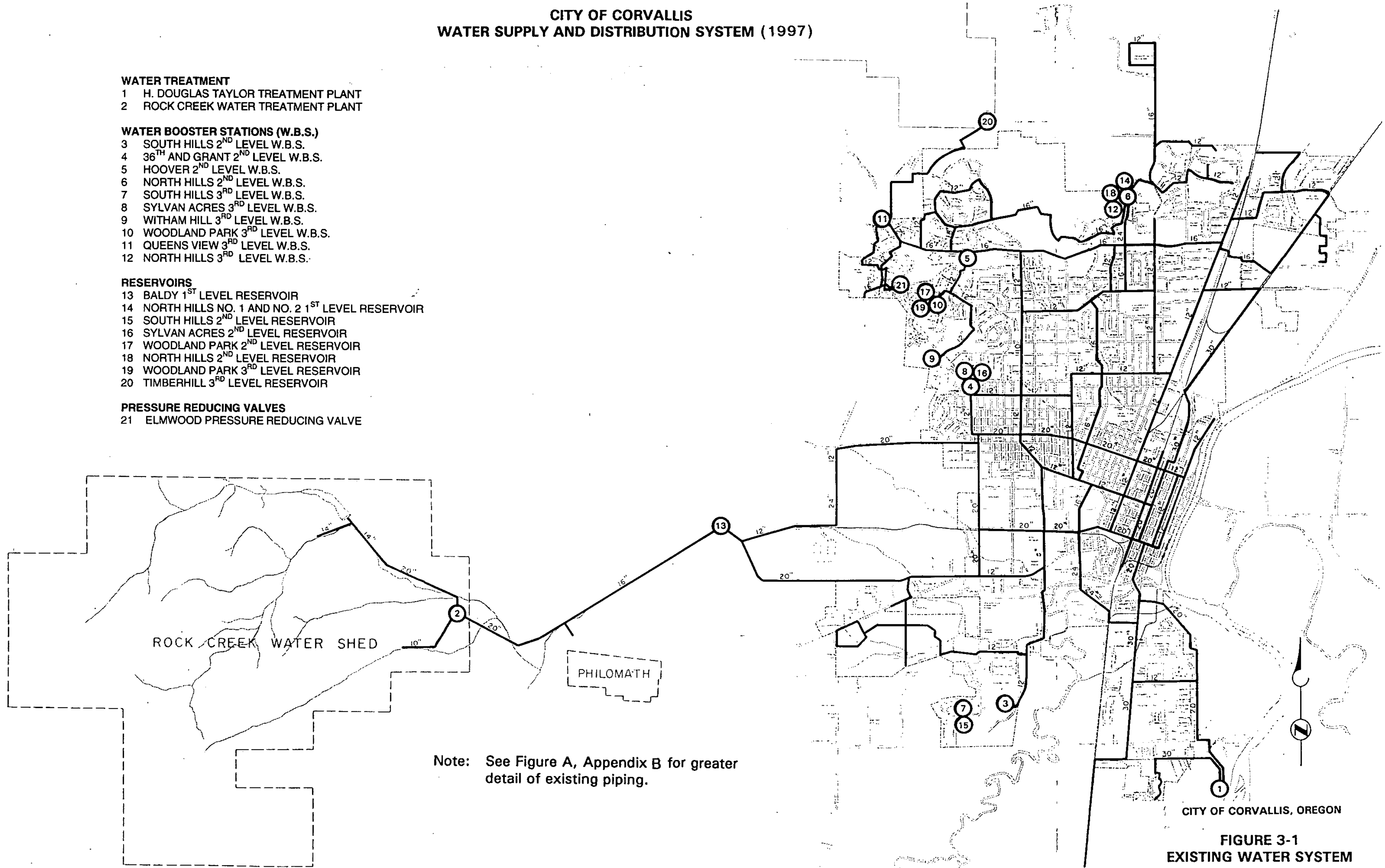
CITY OF CORVALLIS
WATER SUPPLY AND DISTRIBUTION SYSTEM (1997)

WATER TREATMENT
1 H. DOUGLAS TAYLOR TREATMENT PLANT
2 ROCK CREEK WATER TREATMENT PLANT

WATER BOOSTER STATIONS (W.B.S.)
3 SOUTH HILLS 2ND LEVEL W.B.S.
4 36TH AND GRANT 2ND LEVEL W.B.S.
5 HOOVER 2ND LEVEL W.B.S.
6 NORTH HILLS 2ND LEVEL W.B.S.
7 SOUTH HILLS 3RD LEVEL W.B.S.
8 SYLVAN ACRES 3RD LEVEL W.B.S.
9 WITHAM HILL 3RD LEVEL W.B.S.
10 WOODLAND PARK 3RD LEVEL W.B.S.
11 QUEENS VIEW 3RD LEVEL W.B.S.
12 NORTH HILLS 3RD LEVEL W.B.S.

RESERVOIRS
13 BALDY 1ST LEVEL RESERVOIR
14 NORTH HILLS NO. 1 AND NO. 2 1ST LEVEL RESERVOIR
15 SOUTH HILLS 2ND LEVEL RESERVOIR
16 SYLVAN ACRES 2ND LEVEL RESERVOIR
17 WOODLAND PARK 2ND LEVEL RESERVOIR
18 NORTH HILLS 2ND LEVEL RESERVOIR
19 WOODLAND PARK 3RD LEVEL RESERVOIR
20 TIMBERHILL 3RD LEVEL RESERVOIR

PRESSURE REDUCING VALVES
21 ELMWOOD PRESSURE REDUCING VALVE



Note: See Figure A, Appendix B for greater detail of existing piping.

CITY OF CORVALLIS, OREGON

FIGURE 3-1
EXISTING WATER SYSTEM
(COURTESY OF CORVALLIS UTILITIES DEPARTMENT)

Table 3-1. Existing Reservoirs

Reservoir location	Map No.[a]	Service level	Year of construction	Construction materials	Capacity (mg)	Bottom elevation (USGS, ft)	Overflow elevation (USGS, ft)
Baldy	13	1st	1936	Concrete	7.5	384.03	401.78
North Hills #1	14	1st	1959	Concrete	5.0	371.00	397.0
North Hills #2	14	1st	1969	Concrete	5.0	371.00	397.0
South Hills	15	2nd	1973	Concrete	0.5	457.50	481.0
Sylvan Acres	16	2nd	1952	Steel (standpipe)	0.25	435.57	495.57
Woodland Park	17	2nd	1969	Concrete	1.25	472.30	495.81
North Hills	18	2nd	1974	Concrete	2.5	471.00	496.0
Woodland Park	19	3rd	1969	Steel (elevated)	0.125	598.94	621.94
Timberhill	20	3rd	1982	Concrete	1.0	641.00	660.00

[a] Refer to Figure 3-1.

Table 3-2. Existing Booster Pumping Stations

Pumping station location	Map No.[a]	Service level	Serving areas with reservoirs	Quantity of pumps	Pump motor size (hp)	Capacity of each pump (gpm)
South Hills	3	2nd	Yes	2	15	225
36th and Grant	4	2nd	Yes	2	25	300
Hoover	5	2nd	Yes	2	40	750
North Hills	6	2nd	Yes	1	25	900
				1	20	500
				1	100	4,000
South Hills	7	3rd	No	1	5	150
				2	15	500
Sylvan	8	3rd	No	1	2	50
				1	3	110
Witham Hill	9	3rd	Yes	1	25	500
Woodland Park	10	3rd	Yes	2	40	750
Queens View	11	3rd	Yes	2	50	790
North Hills	12	3rd	No	2	25	350
				1	7.5	110

[a] Refer to Figure 3-1.

SYSTEM OPERATION

The methods of system operation are detailed in the following paragraphs.

H.D. Taylor Water Treatment Plant

The high service pumping station at the H.D. Taylor WTP has seven pumps which provide a total capacity of 22 mgd at a total dynamic head of approximately 250 feet.

The discharge from the high service pumping station is controlled by manually turning on and off the pumps. The plant operators monitor the water level in the pumping station clearwell and the water levels in the first service level reservoirs to determine at what rate the pumping station should discharge and which pumps should be on.

Rock Creek Water Treatment Plant

The discharge from the Rock Creek WTP flows by gravity from the clearwell at the plant to the transmission pipeline. In order to achieve adequate chlorine contact time for disinfection, a minimum water depth in the clearwell must be maintained. To maintain water depth, a control valve in the transmission pipeline modulates open or closed as needed, controlled by a signal from a level sensor in the clearwell.

Booster Pumping Stations Serving Areas With Reservoirs

Generally, the booster pumping stations which pump to storage reservoirs (see Table 3-2) are controlled automatically off the water level in the reservoirs. Signals from level sensors in the reservoirs are used to control the pumps. The lead pump will turn on when the reservoir water level reaches a preset low level. It will pump until the reservoir fills to a high water level, at which point it will turn off. If the reservoir level continues to drop below the preset low level with the lead pump on, the follow pump will turn on when a preset lower level is reached and both pumps will pump until the reservoir fills to a high level, at which point both pumps will turn off.

Booster Pumping Stations Serving Areas Without Reservoirs

Booster pumping stations which pump to areas without reservoirs (see Table 3-2) are controlled automatically off the discharge pressure at the pumping stations. The lead pump will turn on when the discharge pressure reaches a preset low pressure. The pump will run and will turn off when the discharge pressure exceeds a preset high pressure. If the discharge pressure continues to drop below the preset low pressure with the lead pump on, the follow pump will turn on when a preset lower level is reached. The follow pump will pump for 15 minutes, controlled with a timer, at which point it will turn off. If the system pressure has not been restored above the lower pressure level, the follow pump will again turn on for 15 minutes and an alarm will be sent to the operations staff. The follow pump will continue to cycle until the system pressure has been restored.

Reservoirs

The Baldy and Sylvan Acres reservoirs have altitude valves to control flow into and out of the reservoirs. The valves open when the pressure on the tank side of the valve exceeds the pressure on the other side. The other reservoirs do not require altitude valves.

With the location of Baldy reservoir on the west side of the service area where supply is plentiful and demand relatively low, the Baldy reservoir fills sooner than the North Hills reservoir. Once the Baldy reservoir is full, service pressures near the H.D. Taylor WTP increase during periods of low demand, high production.

Pressure Reducing Valves

The Elmwood pressure reducing valve (PRV) is the only PRV in the system. It regulates the flow from the Timberhill third service level to the Woodland Park third service level. The flow occurs due to the difference in overflow elevations between the Timberhill reservoir (elevation 660 feet above mean sea level (fmsl) and the Woodland Park third service level reservoir (elevation 622 fmsl). The valve is electrically controlled and opens when the Woodland Park reservoir is 75 percent full (elevation 616.19 fmsl) and closes when the reservoir is 90 percent full (elevation 619.64 fmsl). The system allows the Woodland Park reservoir to be filled using the existing Queens View pumping station along with the existing Woodland Park third level pumping station. If the pressure on the Timberhill reservoir side of the valve is lower than the Woodland Park reservoir side, the valve can be opened to fill the Timberhill reservoir.

Estimated Capital Costs of Proposed Pipelines

City of Corvallis
Water Distribution System Facility Plan
Estimated Capital Costs of Proposed Pipelines

Construction Phase I

Project/ Pipe Number	Node		Pipe Size (In.)	Length (Ft)	Location Map Quarter	Location Vicinity	Capital Cost/Ft (Dollars)	Total Capital Cost (Dollars) (a)	Notes (b)
	From	To							
Airport									
933	331	343	16	439	SE	Airport Place	138	61,000	
940	331	342	20	1,931	SE	Plumley St.	163	315,000	
Subtotal:								376,000	
North Hills									
571	244	507	36	1,410	NE	Hwy. 99 W.	315	444,000	
572	244	251	36	1,232	NE	Garfield Ave.	315	388,000	
576	103	104	42	407	NE	N. Hills 1st Lvl. Res.	363	148,000	
577	93	103	36	1,454	NE	13th St.	315	458,000	
578	93	237	36	554	NE	Walnut Blvd.	315	175,000	
582	236	237	36	1,258	NE	15th St.	315	396,000	
584	263	573	36	1,050	NE	15th St.	315	331,000	
597	251	263	36	2,541	NE	11th St.	315	800,000	
599	236	573	36	598	NE	15th St.	315	188,000	
Subtotal:								3,328,000	
Timian Street									
800	324	780	8	344	SW	Timian St.	178	61,000	CARE
Subtotal:								61,000	
35th Street									
640	48	400	12	862	SW	35th St.	116	100,000	
Subtotal:								100,000	

Total Phase I Capital Cost: 3,865,000

a. Costs are based on a ENR CCI for Seattle of 6,747. Capital costs include construction cost plus 35 percent for engineering, overhead, and contingency.
b. CARE = Cost Assumes Rock Excavation.

Construction Phase II

Project/ Pipe Number	Node From	Node To	Pipe Size (In.)	Length (Ft)	Location Map Quarter	Location Vicinity	Capital Cost/Ft (Dollars)	Total Capital Cost (Dollars) (a)	Notes (b)
Baldy Supply									
594	44	571	24	5,994	SW	Winding Way	365	2,188,000	CARE
586	43	570	24	468	NW	Baldy Reservoir	365	171,000	CARE
587	43	44	30	998	NW	Bald Hill	399	398,000	CARE
Subtotal:								2,757,000	
Conifer									
570	103	264	30	1,372	NE	Conifer Blvd.	251	344,000	
588	275	503	24	488	NE	Hwy. 99 W.	184	90,000	
598	275	276	24	1,022	NE	Hwy. 99 W.	184	188,000	
590	276	267	24	1,390	NE	9th St.	184	256,000	
591	266	267	30	946	NE	Conifer Blvd.	251	237,000	
592	265	266	30	946	NE	Conifer Blvd.	251	237,000	
593	264	265	30	1,466	NE	Conifer Blvd.	251	368,000	
Subtotal:								1,720,000	
Eastside									
551	505	506	16	3,292	NE	Hewlett Packard	138	454,000	
796	273	508	30	3,849	NE	SPRR	251	966,000	
797	86	440	16	1,516	NE	Seavy Ave.	138	209,000	
Subtotal:								1,629,000	
High School									
550	503	550	24	2,715	NE	Hwy. 99 W.	184	500,000	
552	550	551	20	4,148	NE	Jackson Creek	163	676,000	
553	291	551	20	3,021	NE	Jackson Creek	163	492,000	
564	503	551	24	4,852	NE	Hwy. 99 W.	184	893,000	
Subtotal:								2,561,000	
Mary's Creek Crossing									
787	26	36	36	1,042	SE	Hwy. 99 W.	315	328,000	
788	36	37	36	1,520	NE	Hwy. 99 W.	315	479,000	
Subtotal:								807,000	
North Hills Loop									
652	651	652	12	1,119	NE	Lester Ave.	100	112,000	
656	652	656	20	4,826	NE	Timberhill	260	1,255,000	CARE
777	651	660	16	2,712	NE	North Hills	217	589,000	CARE
792	652	660	16	7,581	NE	North Hills	109	826,000	
Subtotal:								2,782,000	
Skyline West									
631	323	631	16	2,485	NW	Ponderosa Ave.	217	539,000	CARE
Subtotal:								539,000	
Southwest Loop									
473	473	474	16	2,818	SE	53rd St.	138	389,000	
484	473	512	16	3,287	SE	53rd St.	138	454,000	
596	512	571	24	507	SE	Country Club Dr.	184	93,000	
779	473	501	12	910	SW	South West	116	106,000	
814	474	806	12	757	SW	Country Club Dr.	116	88,000	
Subtotal:								1,130,000	

Construction Phase II (Continued)

Project/ Pipe Number	Node From	Node To	Pipe Size (In.)	Length (Ft)	Location Map Quarter	Location Vicinity	Capital Cost/Ft (Dollars)	Total Capital Cost (Dollars) (a)	Notes (b)
Wake Robin									
470	471	482	24	2,731	SW	Wake Robin Ave.	184	503,000	
471	471	472	24	5,132	SE	Agate Ave.	184	944,000	
472	472	473	24	2,636	SE	Nash Ave.	184	485,000	
499	482	483	24	662	SW	SPRR	184	122,000	
Subtotal:								2,054,000	
West Hills									
606	T612	605	20	2,267	NW	Bald Hill	260	589,000	CARE
608	570	606	24	816	NW	Bald Hill	365	298,000	CARE
609	850	607	18	3,853	SW	West Hills Road	115	443,000	
610	607	608	16	2,497	SW	Winding Way	109	272,000	
875	606	850	24	938	NW	Reservoir Ave.	184	173,000	
876	57	58	12	1,817	NW	Reservoir Ave.	116	211,000	
877	607	45	12	3,303	SW	West Hills Road	116	383,000	
879	57	850	24	1,140	NW	Reservoir Ave.	184	210,000	
880	57	851	16	1,020	NW	Reservoir Ave.	109	111,000	
Subtotal:								2,690,000	
45th Street									
478	391	478	12	1,291	SE	Country Club Dr.	116	150,000	
481	472	478	12	3,248	SE	45th St.	116	377,000	
483	808	478	12	1,033	SE	Country Club Dr.	116	120,000	
Subtotal:								647,000	
53rd Street									
475	475	476	20	1,978	SE	53rd St.	163	322,000	
476	59	476	20	2,222	SE	53rd St.	163	362,000	
482	324	805	12	738	SE	Philomath Blvd.	116	86,000	
778	398	772	12	816	SW	West Hills Road	116	95,000	
Subtotal:								865,000	

Total Phase II Capital Cost: 20,181,000

a. Costs are based on a ENR CCI for Seattle of 6,747. Capital costs include construction cost plus 35 percent for engineering, overhead, and contingency.
b. CARE = Cost Assumes Rock Excavation.

Construction Phase III

Pipe Number	Node From	Node To	Pipe Size (In.)	Length (Ft)	Location Map Quarter	Location Vicinity	Capital Cost/Ft (Dollars)	Total Capital Cost (Dollars) (a)	Notes (b)
Airport									
450	451	830	24	1,468	SE	Hwy. 99 W.	184	270,000	
451	450	830	24	1,539	SE	Hwy. 99 W.	184	283,000	
452	451	452	16	2,700	SE	Hwy. 99 W.	138	373,000	
453	452	453	16	2,435	SE	Hwy. 99 W.	138	336,000	
461	451	457	12	1,414	SE	Herbert Ave.	116	164,000	
462	452	458	12	1,490	SE	Weltzin Ave	116	173,000	
622	357	450	24	383	SE	Hwy. 99 W.	184	70,000	
850	829	830	24	2,373	SE	Kiger Island Dr.	184	437,000	
Subtotal:								2,106,000	
Crescent Valley									
681	680	711	20	1,252	NE	Crescent Valley Dr.	130	163,000	
711	710	712	12	1,199	NE	Highland Dr.	100	120,000	
712	711	712	24	1,576	NE	Crescent Valley Dr.	155	244,000	
713	289	712	24	4,121	NE	Highland Dell Dr.	155	639,000	
Subtotal:								1,166,000	
Crystal Lake									
793	15	25	36	9,499	SW	H.D. Taylor WTP	315	2,992,000	
794	15	19	36	265	NE	H.D. Taylor WTP	315	83,000	
Subtotal:								3,075,000	
Mary's Creek Crossing									
786	25	26	36	956	SE	Hwy. 99 W.	315	301,000	
790	80	81	36	448	NE	Hwy. 99 W.	315	141,000	
795	81	507	36	4,252	SE	Hwy. 99 W.	315	1,339,000	
Subtotal:								1,781,000	
Sylvan Acres									
615	196	213	16	2,954	NW	Witham Hill Dr.	109	322,000	
616	185	613	12	1,044	NW	Elizabeth Place	100	104,000	
Subtotal:								426,000	
Weltzin/Ingalls Loop									
455	454	455	20	2,364	SE	Ingalls St.	163	385,000	
456	455	458	20	2,775	SE	Weltzin Ave.	163	452,000	
930	332	454	20	1,992	SE	Airport Ave.	163	325,000	
931	333	334	20	1,027	SE	Airport Place	163	167,000	
932	332	333	20	854	SE	Airport Place	163	139,000	
934	330	458	12	670	SE	Hout Street	116	78,000	
935	330	345	12	1,025	SE	Hout Street	116	119,000	
936	331	345	12	1,218	SE	Hout Street	116	141,000	
937	334	342	16	860	SE	Plumley Street	138	119,000	
Subtotal:								1,925,000	
West Hills									
604	602	603	12	1,827	NW	Oak Creek Dr.	199	364,000	CARE
605	602	605	18	5,066	NW	Mulky Creek	230	1,165,000	CARE
607	605	606	18	1,744	NW	Bald Hill	230	401,000	CARE
Subtotal:								1,930,000	

Total Phase III Capital Cost: 12,409,000

a. Costs are based on a ENR CCI for Seattle of 6,747. Capital costs include construction cost plus 35 percent for engineering, overhead, and contingency.
b. CARE = Cost Assumes Rock Excavation.

Construction Phase IV

Pipe Number	Node From	Node To	Pipe Size (In.)	Length (Ft)	Location Map Quarter	Location Vicinity	Capital Cost/Ft (Dollars)	Total Capital Cost (Dollars) (a)	Notes (b)
Crescent Valley									
682	681	711	20	1,619	NE	Crescent Valley Dr.	130	210,000	
683	681	713	16	1,562	NE	Crescent Valley Dr.	109	170,000	
684	681	683	18	1,902	NE	Crescent Valley Dr.	115	219,000	
685	682	683	16	1,626	NE	Crescent Valley Dr.	109	177,000	
686	682	717	16	3,294	NE	Crescent Valley Dr.	109	359,000	
687	683	685	12	3,221	NE	Lewisburg Ave.	100	322,000	
688	685	717	12	2,087	NE	Lewisburg Ave.	100	209,000	
714	711	713	18	3,460	NE	Crescent Valley Dr.	115	398,000	
715	682	713	20	1,992	NE	Crescent Valley Dr.	130	259,000	
723	T718	717	18	2,221	NE	Sulphur Springs Road	115	255,000	
Subtotal:								2,578,000	
High School									
554	292	553	12	2,167	NE	Highland Dr.	116	251,000	
555	553	554	12	4,342	NE	Highland Dr.	116	504,000	
559	551	557	24	2,831	NE	Shasta Ave.	184	521,000	
560	554	557	24	2,262	NE	Shasta Ave.	184	416,000	
562	553	557	16	2,561	NE	Highland Dr.	138	353,000	
Subtotal:								2,045,000	
Lewisburg									
556	550	555	20	4,699	NE	Hwy. 99 W.	163	766,000	
557	555	556	20	1,929	NE	Hwy. 99 W.	163	314,000	
558	554	556	16	1,990	NE	Lewisburg Ave.	138	275,000	
561	555	557	16	2,244	NE	Hwy. 99 W.	138	310,000	
709	556	709	12	1,276	NE	Locke Cemetary	100	128,000	
Subtotal:								1,793,000	
Upper Crescent Valley									
690	682	686	16	1,965	NE	Frazier Creek Road	109	214,000	
691	686	687	16	4,237	NE	Belhaven Dr.	109	462,000	
692	686	T689	16	2,225	NE	Frazier Creek Road	109	243,000	
693	686	688	12	957	NE	Brownly Height Dr.	100	96,000	
721	684	687	16	761	NE	Lewisburg Ave.	109	83,000	
722	687	720	16	896	NE	Sulphur Springs Road	109	98,000	
Subtotal:								1,196,000	
West Hills									
601	600	601	18	2,400	NW	Walnut Blvd.	230	552,000	CARE
602	601	613	16	4,095	NW	Walnut Blvd.	109	446,000	
603	601	602	18	4,768	NW	Oak Creek Dr.	230	1,097,000	CARE
Subtotal:								2,095,000	

Total Phase IV Capital Cost: 9,707,000

a. Costs are based on a ENR CCI for Seattle of 6,747. Capital costs include construction cost plus 35 percent for engineering, overhead, and contingency.
b. CARE = Cost Assumes Rock Excavation.