

isultants, Inc.

Lab Report No. 74484

Client P.O. _____

Date Received 1/8/97 1000

IS REPORT

Attention Rick Mumpower Collected Date 1/7/97 Time 1430
 Client City of Florence Collected by Tom Cannon
PO Box 340 Source Sludge / Digester
Florence, OR 97439 Location WWTP, Florence, OR

SEWAGE SLUDGE DEQ LIST

PARAMETER	METHOD	RESULTS
Arsenic (Total)	EPA 206.2/7060	<u>14.7</u> mg/kg dry weight
Cadmium (Total)	EPA 213.2/7131	<u>3.1</u> mg/kg dry weight
Chromium (Total)	EPA 218.2/7191	<u>17.9</u> mg/kg dry weight
Copper (Total)	EPA 220.1/7210	<u>613</u> mg/kg dry weight
Lead (Total)	EPA 239.2/7421	<u>33.2</u> mg/kg dry weight
Mercury (Total)	EPA 245.1/7470	<u>2.2</u> mg/kg dry weight
Molybdenum (Total)	EPA 246.2/7481	<u>7.4</u> mg/kg dry weight
Nickel (Total)	EPA 249.2/7521	<u>24.1</u> mg/kg dry weight
Selenium (Total)	EPA 270.2/7740	<u>ND@ 10.0</u> mg/kg dry weight
Zinc (Total)	EPA 289.1/7950	<u>712</u> mg/kg dry weight
Total Nitrogen (TKN)	EPA 351.3	<u>11.6</u> % dry weight
Nitrate Nitrogen	EPA 353.3	<u>ND@ 0.01</u> % dry weight
Ammonia Nitrogen	EPA 350.2	<u>6.35</u> % dry weight
Total Phosphorus	EPA 365.3	<u>4.73</u> % dry weight
Potassium (Total)	EPA 258.1/7610	<u>1.48</u> % dry weight
pH	EPA 150.1/9040	<u>7.7</u>
Total Solids	EPA 160.3	<u>2.72</u> %
Volatile Solids	EPA 160.4	<u>72.3</u> %

ND means "not detected"

APPROVED

Rory E. White

DATE 2/10/97

APPENDIX E

SUMMARY OF BIOASSAY FOR WASTEWATER TREATMENT PLANT EFFLUENT

NORTHWESTERN AQUATIC SCIENCES

A Division of NAS Associates, Inc.

P.O. Box 1437, Newport, Oregon 97365 (503) 265-7225



August 27, 1997

Mr. Rick Mumpower
City of Florence
P.O. Box 340
Florence, OR 97437

Dear Mr. Mumpower:

Enclosed, please find copies of Repts. No. 573-1 and 573-2 giving the results of a dual-endpoint fathead minnow test and a rainbow trout acute test, respectively. No acute toxicity was observed in either test.

In the chronic toxicity test, due to a fungal problem encountered, there was significant mortality and also growth inhibition in the 50% effluent treatment. There was no significant mortality or growth inhibition at the 100% effluent treatment. Therefore, it was concluded that the effects seen in the 50% effluent treatment were an anomaly due to the fungus and not to effluent toxicity. The no-observed-effect-concentration (NOEC) was concluded to be 100% effluent (no chronic toxicity). In addition, although the computation methods indicated an IC25 (point estimate for chronic toxicity) of 38.4% effluent, we conclude, for the reasons given above, that this does not indicate that effluent toxicity occurred at this concentration.

If you have any questions, please feel free to call me at 541-265-7225.

Sincerely,

A handwritten signature in black ink, appearing to read "Gary A. Buhler". The signature is fluid and cursive, with the first name "Gary" being more prominent.

Gary A. Buhler
Project Manager

Encl.

TOXICITY TEST REPORT

TEST IDENTIFICATION

Test No.: 573-1Title: Fathead Minnow dual endpoint (acute/chronic) toxicity test using static exposure to City of Florence effluent.Protocol No.: NAS-XXX-PP2, September 15, 1990, Revision 2 (6-1-96).Based on U.S. EPA. 1994. Method 1000.0, Fathead minnow, Pimephales promelas, larval survival and growth test, pp. 58-113, In: Short-term methods for estimating the chronic toxicity of effluents and receiving waters to freshwater organisms. Third edition. EPA/600/4-91/002.

STUDY MANAGEMENT

Study Sponsor: City of Florence, P.O. Box 340, Florence, OR 97439.Sponsor's Study Monitor: Mr. Rich MumpowerTesting Laboratory: Northwestern Aquatic Sciences, P.O. Box 1437, Newport, OR 97365.Test Location: Newport Laboratory.Laboratory's Study Personnel: G.A. Buhler, B.S. Proj. Man./Study Dir.;

L.K. Nemeth, B.A., QA Officer; M.S. Redmond, M.S., Aq. Toxicol.; G.J. Irissarri, B.S., Aq. Toxicol.; E. Coffey, B.S., Tech.

Study Schedule:

Test Beginning: 7-24-97, 12:00 p.m.

Test Ending: 7-31-97, 10:30 a.m.

Disposition of Study Records: All specimens, raw data, reports and other study records are stored according to Good Laboratory Practice regulations at: Northwestern Aquatic Sciences, 334 SW 7th Street, Suite B, Newport, OR 97365.Good Laboratory Practices: The test was conducted following the principles of Good Laboratory Practices (GLP) as defined in the EPA/TSCA Good Laboratory Practice regulations revised August 17, 1989 (40 CFR Part 792).Statement of Quality Assurance: The test data were reviewed by the Quality Assurance Unit to assure that the study was performed in accordance with the protocol and standard operating procedures. This report is an accurate reflection of the raw data.

TEST MATERIAL

Description: City of Florence unchlorinated secondary effluent. Details and water quality conditions at time of sample receipt are as follow:

NAS Sample No.	8694E	8703E	8701E
Collection Date	7-23-97	7-25-97	7-28-97
Receipt Date	7-24-97	7-26-97	7-29-97
Conductivity (umhos/cm)	370	360	380
pH	7.1	7.2	7.4
Hardness (mg/L)	60	40	40
Alkalinity (mg/L)	140	130	150
Total ammonia-N (mg/L)	4.5	7.5	8.0

Treatments: Samples briefly temperature equilibrated prior to use.Storage: Stored at 4°C in the dark until used.

DILUTION WATER

Source: Moderately hard synthetic water prepared from Milli-Q water.

Dates of Preparation: 7-23-97, 7-28-97

Water Quality: Conductivity, 300/280 $\mu\text{mhos/cm}$; pH, 7.9/7.8; hardness, 100/100 mg/L as CaCO_3 ; alkalinity, 80/70 mg/L as CaCO_3 .

Pretreatment: Aerated ≥ 24 hr

TEST ORGANISMS

Species: *Pimephales promelas*, fathead minnow.

Age: 24 to 48-hr-old.

Source: Aquatox, Inc., Hot Springs, Arkansas

Acclimation: Holding conditions during the 24 hours prior to testing were: temperature, 22.4°C; dissolved oxygen, 8.3 mg/L; pH, 7.8; conductivity, 475 $\mu\text{mhos/cm}$.

TEST PROCEDURES AND CONDITIONS

Test Chambers: 600 ml glass beakers containing 250 ml of test solutions.

Test Concentrations: 100, 50, 25, 12.5, 6.25, and 0 % (control).

Replicates/Treatment: 4

Organisms/Treatment: 40

Loading (based on final weight of control organisms): 0.02 g/L

Water Volume Changes per 24 hr: 1

Aeration: None

Feeding: Approximately 0.15g newly hatched *Artemia salina* nauplii per beaker twice daily, 6 hours apart, except on day 7.

Effects Criteria: The effect criteria used were: 1) mortality, and 2) growth inhibition. Mortality was defined as lack of visible movement during a 30 second observation period. Growth inhibition was measured as the difference in weight of fish between a treatment level and the control.

Water Quality and Other Test Conditions: Temperature, $24.6 \pm 0.2^\circ\text{C}$; dissolved oxygen, 7.1 ± 1.3 mg/L; conductivity, 454 ± 41 $\mu\text{mhos/cm}$ (100% effluent), 297 ± 17 $\mu\text{mhos/cm}$ (control); pH, 7.5 ± 0.2 ; hardness, 40 ± 0 mg/L as CaCO_3 (100% effluent), 100 ± 0 mg/L as CaCO_3 (control); alkalinity, 124 ± 15 mg/L as CaCO_3 (100% effluent), 77 ± 5 mg/L as CaCO_3 (control); and photoperiod 16:8 hr, L:D.

DATA ANALYSIS METHODS

Percent survival and the average weight per larva were calculated for each treatment replicate from the raw data and the means were obtained for each treatment level. Average weights were calculated two ways: 1) based on the number of surviving fish (the historical method), and 2) based on the initial number of fish (as per EPA 600/4-91/002). The LC50 (survival) was calculated, where data permitted, either by the probit or the Trimmed Spearman-Kärber method. The IC25 (growth) was calculated using the Linear Interpolation Method with bootstrapping. NOEC and LOEC values for survival and growth were computed using ANOVA and an appropriate post hoc test (Dunnett's test, T-Test with Bonferroni's adjustment, Steels Many-One Rank Test, or Wilcoxon Rank Sum Test with Bonferroni Adjustment). The appropriate test was selected after evaluating the data for normality and homogeneity of variance. Weight data were excluded from the ANOVA calculation if there was a survival effect. An arcsine transformation was performed on the survival data prior to statistical analysis. The statistical software employed for these calculations was ToxCalc, v. 5.0.15, Tidepool Scientific Software.

PROTOCOL DEVIATIONS

None

REFERENCE TOXICANT TEST

Test No.: 999-788

Reference Toxicant and Source: $\text{CdCl}_2 \cdot 2.5\text{H}_2\text{O}$, Mallinckrodt, Lot No. TNZ.

Test Date: 7-24-97

Dilution Water Used: Moderately hard synthetic water.

Result: 7-day LC50, 21.4 ug/L Cd; 7-day IC25, 15.3 ug/L Cd; NOEC, 10 ug/L Cd. These test results are within the laboratory's control chart warning limits.

TEST RESULTS

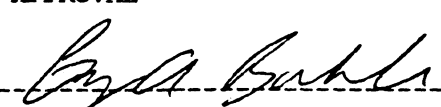
Acute Endpoint: A detailed tabulation of the acute test results is given in Table 1. There was no mortality of fathead minnows exposed to 100% effluent after 48 hours. Therefore, the effluent passed the acute test according to Oregon DEQ guidelines (DEQ Whole Effluent Toxicity Testing Guidance Manual, January 1993).

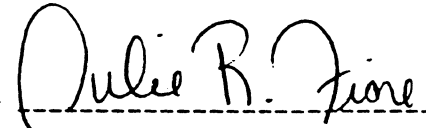
Chronic Endpoint: A detailed tabulation of the chronic test results is given in Table 2. The biological effects, given as the NOEC and LOEC for survival and growth, and the LC50/IC25 for survival/growth, are shown below.

Comments: Control survival (97.5%) met the test acceptability criterion (80%). Control weights (0.58 mg) also met acceptability criterion (0.25 mg). The reference toxicant test results were within control chart limits. Therefore, this toxicity test is considered a valid test. It should be noted however, that there was significant fungal growth in most test chambers on days 5, 6, and 7 of the test with the heaviest growth in the 50% chambers and no fungal growth in the 100% chambers. All deaths except two occurred on day 7 of the test. Since this fungus also grew in control test chambers, it is likely that the fungus came with the batch of fish and not from the effluent. From examining the data, it appears that any toxic effect was likely due to the fungus.

	Survival	Growth based on survivors	Growth based on initial no.
NOEC (%)	100	100	100
LOEC (%)	>100	>100	>100
7-Day LC50/IC25 (%)	>100	>100	38.4
(95% conf. int.)	--	--	(23.2 - 51.5)
Method	Data Inspection	Linear Interpolation	Linear Interpolation

STUDY APPROVAL

 8/27/97
 Study Director Date

 8-26-97
 Quality Assurance Unit Date

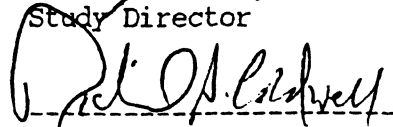
 8/27/97
 Manager, Toxicology Date

Table 1. Survival of fathead minnow larvae exposed for 48-hours to City of Florence effluent.

Effluent concentration (%)	Repl.	Number of larvae			Percent survival	Mean* percent survival
		Exposed	Dead	Surviving		
100	1	10	0	10	100.0	100.0
	2	10	0	10	100.0	
	3	10	0	10	100.0	
	4	10	0	10	100.0	
Control	1	10	0	10	100.0	100.0
	2	10	0	10	100.0	
	3	10	0	10	100.0	
	4	10	0	10	100.0	

* An asterisk next to a treatment mean indicates that it is significantly ($P < 0.05$) less than the control mean.

Table 2. Survival and growth of fathead minnow larvae exposed for seven days to City of Florence effluent.

Effluent conc. (%)	Repl.	Number of larvae			% surv.	Mean % surv.	Ave. ¹ wt./larva (mg)	Mean wt. (mg)	Ave. ² wt./larva (mg)	Mean wt. (mg)
		Exposed	Dead	Surv.						
100	1	10	0	10	100.0		0.517		0.517	
	2	10	0	10	100.0		0.534		0.534	
	3	10	0	10	100.0		0.449		0.449	
	4	10	1	9	90.0	97.5	0.561	0.515	0.505	0.501
50	1	10	9	1	10.0		0.350		0.035	
	2	10	4	6	60.0		0.468		0.281	
	3	10	4	6	60.0		0.392		0.235	
	4	10	5	5	50.0	45.0*	0.520	0.433*	0.260	0.203*
25	1	9a	0	9	100.0		0.546		0.546	
	2	10	0	10	100.0		0.569		0.569	
	3	10	0	10	100.0		0.466		0.466	
	4	10	1	9	90.0	97.5	0.494	0.519	0.445	0.506
12.5	1	9a	0	9	100.0		0.526		0.526	
	2	10	0	10	100.0		0.556		0.556	
	3	10	0	10	100.0		0.585		0.585	
	4	10	0	10	100.0	100.0	0.500	0.542	0.500	0.542
6.25	1	10	0	10	100.0		0.530		0.530	
	2	10	2	8	80.0		0.526		0.421	
	3	10	0	10	100.0		0.543		0.543	
	4	10	0	10	100.0	95.0	0.567	0.542	0.567	0.515
Control	1	9a	0	9	100.0		0.632		0.632	
	2	10	0	10	100.0		0.608		0.608	
	3	10	1	9	90.0		0.481		0.433	
	4	10	0	10	100.0	97.5	0.585	0.577	0.585	0.565

* An asterisk next to a treatment mean indicates that it is significantly (P<0.05) less than the control mean.

¹Average weight based on number of surviving fish larvae per replicate.

²Average weight based on initial number of fish larvae per replicate.

^aInitial number of fish reduced by one to compensate for missing fish during the test.

TOXICITY TEST REPORT

TEST IDENTIFICATION

Test No.: 573-2Title: Rainbow trout, *Oncorhynchus mykiss*, 96-hr acute bioassay using static exposure to City of Florence effluent.Protocol No.: NAS-XXX-QM1, June 23, 1990, Revision 2 (3-8-95). Based on Weber, C.I. 1991. Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms (Fourth Edition), EPA 600/4-90/027.

STUDY MANAGEMENT

Study Sponsor: City of Florence, P.O. Box 340, Florence, OR 97437.Sponsor's Study Monitor: Mr. Rick MumpowerTesting Laboratory: Northwestern Aquatic Sciences, P.O. Box 1437, Newport, OR 97365.Test Location: Newport laboratory.Laboratory's Study Personnel: G.A. Buhler, B.S., Proj. Man./Study Dir.; L.K. Nemeth, B.A., QA Officer; M.S. Redmond, M.S., Aq. Toxicol.; Coffey, B.S., Tech.Study Schedule:

Test Beginning: 7-24-97, 2:00 p.m.

Test Ending: 7-28-97, 2:20 p.m.

Disposition of Study Records: All specimens, raw data, reports and other study records are stored according to Good Laboratory Practice regulations at: Northwestern Aquatic Sciences, 334 S.W. 7th Street, Suite B, Newport, OR 97365.Good Laboratory Practices: The test was conducted following the principles of Good Laboratory Practices (GLP) as defined in the EPA/TSCA Good Laboratory Practice regulations revised August 17, 1989 (40 CFR Part 792).Statement of Quality Assurance: The test data were reviewed by the Quality Assurance Unit to assure that the study was performed in accordance with the protocol and standard operating procedures. This report is an accurate reflection of the raw data.

TEST MATERIAL

Description: Unchlorinated final effluent (24HC). Details and water quality conditions at time of sample receipt are as follow:

NAS Sample No.	8694E	8702E
Collection Date	7-23-97	7-25-97
Receipt Date	7-24-97	7-26-97
Receipt Temperature (°C)	8.0	6.1
Dissolved oxygen (mg/L)	9.2	8.6
Conductivity (umhos/cm)	370	360
pH	7.1	7.2
Hardness (mg/L)	60	40
Alkalinity (mg/L)	140	130
Total ammonia-N (mg/L)	4.5	7.5

Treatments: Samples thermal equilibrated prior to use in test.Storage: Stored refrigerated (4°C) in sealed container until tested.

DILUTION WATER

Source: NAS spring water

Date of Collection: 7-24-97

Water Quality: Conductivity, 260 $\mu\text{mhos/cm}$; pH, 7.7; hardness, 100 mg/L as CaCO_3 ; and alkalinity, 60 mg/L as CaCO_3 .

Pretreatment: Aerated prior to use.

TEST ORGANISMS

Species: Rainbow trout, *Oncorhynchus mykiss*

Size/weight: 0.31 g/fish.

Source: Purchased 7-18-97 from Mt. Lassen Trout Farms, Red Bluff, CA.

Hatch date, 6-15-97.

Acclimation: Trout were held in flow-through tanks supplied with fresh spring water and aeration. Trout were fed twice daily until 24 hours prior to testing. Water quality conditions for the week prior to testing averaged: Temperature, $11.5 \pm 0.9^\circ\text{C}$; dissolved oxygen, 12.0 ± 2.0 mg/L; pH, 7.2 ± 0.5 ; conductivity, 295 ± 13 $\mu\text{mhos/cm}$; hardness, 95 mg/L as CaCO_3 ; and alkalinity, 95 mg/L as CaCO_3 . Acclimation tank loading: 1.0 g/L.

TEST PROCEDURES AND CONDITIONS

Test Chambers: 2-gallon glass aquaria containing 4.0 L of test solution.

Test Concentrations: 100, 50, 25, 12.5, 6.25, and 0% (control).

Replicates/Treatment: 2

Organisms/Treatment: 20

Loading: 0.78 g/L

Duration: 96 hours

Aeration: None.

Feeding: None during the test or for 24 hours prior to testing.

Water Volume Changes: One at 48 hrs.

Effect Criterion: Mortality, defined as the lack of respiratory movement in response to tactile stimulation.

Water Quality and Other Test Conditions: Temperature, $12.0 \pm 0.2^\circ\text{C}$; dissolved oxygen, 10.3 ± 0.5 mg/L; conductivity, 450 ± 21 $\mu\text{mhos/cm}$ (100% effluent), 296 ± 22 $\mu\text{mhos/cm}$ (control); pH, 7.5 ± 0.2 ; hardness, 60 mg/L as CaCO_3 (100% effluent), 100 mg/L as CaCO_3 (control); alkalinity, 150 mg/L as CaCO_3 (100% effluent), 60 mg/L as CaCO_3 (control); and photoperiod 16:8 hr, L:D.

DATA ANALYSIS METHODS

Percent survival was calculated for each treatment replicate from the raw data and the means were obtained for each treatment level. The LC50 was calculated, where data permitted, either by the Probit or the Trimmed Spearman-Kärber method. The statistical software employed for these calculations was ToxCalc, v.5, Tidepool Scientific Software.

PROTOCOL DEVIATIONS

None.

REFERENCE TOXICANT TEST

Test No.: 999-787

Reference Toxicant and Source: SDS, sodium dodecyl sulfate (Sigma Chemical, Lot No. 17H-0459).

Test Date: 7-24-97

Dilution Water Used: NAS spring water.

Result: LC50, 12.0 mg/L. This result is within the laboratory's control chart warning limits.

TEST RESULTS

A detailed tabulation of the test results is given in Table 1. The biological effect, given as the LC50 is as shown below.

96-hr LC50 (%)

>100

(95% C.I.)

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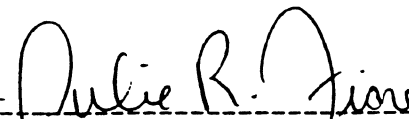
Method

By Data Inspection

STUDY APPROVAL



8/27/97



8-26-97

Study Director

Date

Quality Assurance Unit

Date



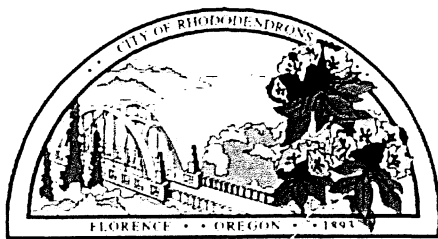
8/27/97

Manager, Toxicology

Date

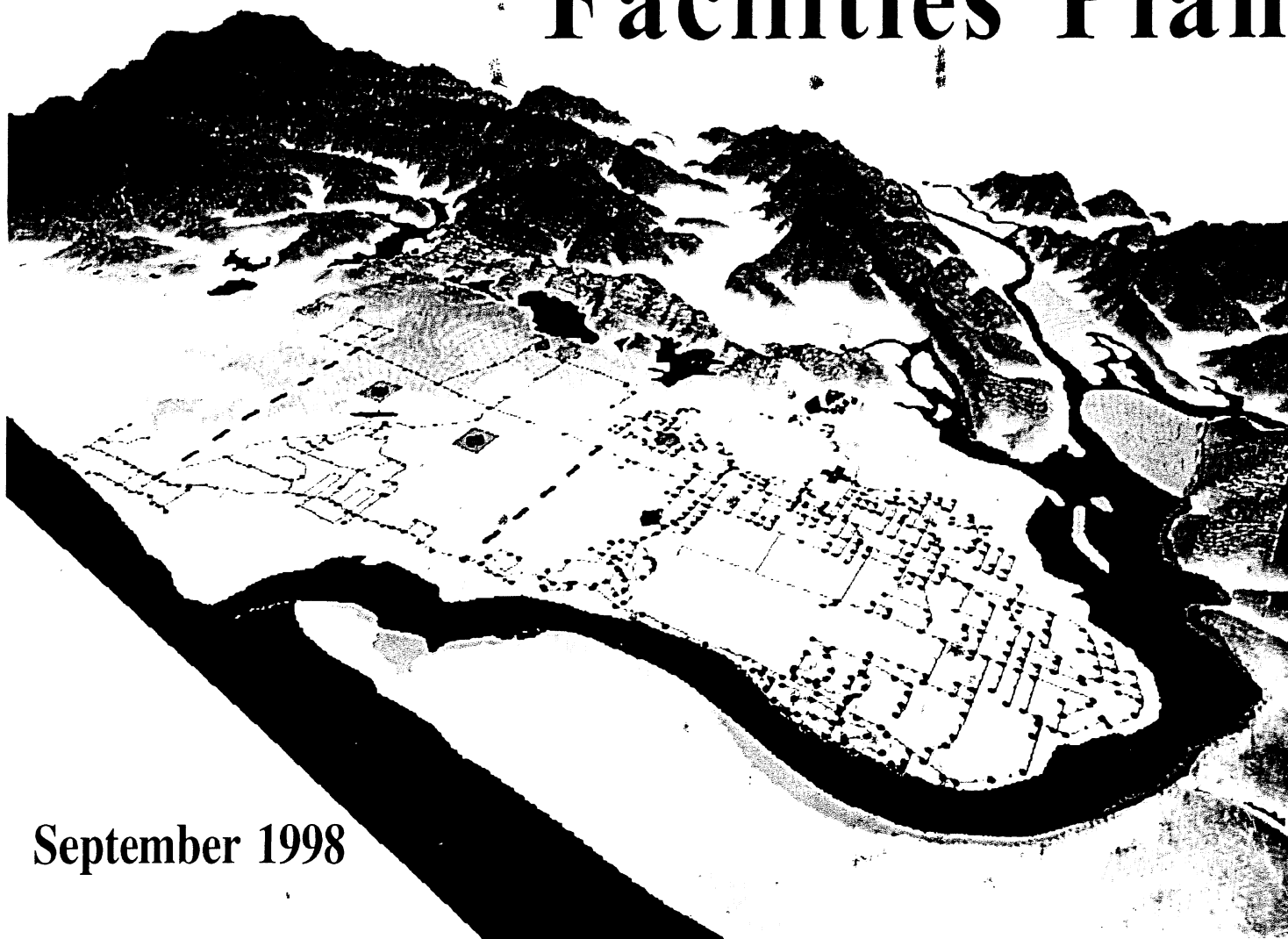
Table 1. Survival of Oncorhynchus mykiss exposed to City of Florence effluent.

Effluent Conc. (%)	Repl.	Number of trout surviving					96-hr % Survival	
		0-hr	24-hr	48-hr	72-hr	96-hr	individual	mean
100	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
50	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
25	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
12.5	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
6.25	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
Control	1	10	10	10	10	10	100.0	
	2	10	10	10	9	9	90.0	95.0



City of Florence

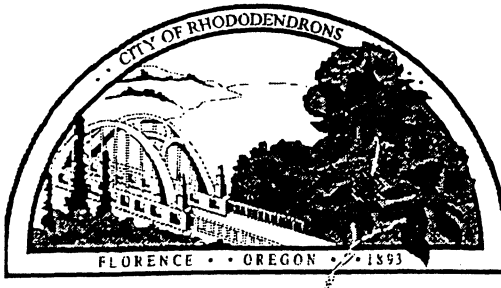
Water Facilities Plan



September 1998

**BROWN AND
CALDWELL**

A Complete Copy of This Document Can Be
Viewed In Florence At City Hall, And At The
County Administration Office Or The Land
Management Division Office, Both Located At:
125 E 8th Avenue, Eugene



City of Florence

Water Facilities Plan

September 1998



EXPIRES: 12/31/99

B R O W N A N D
C A L D W E L L

B R O W N A N D
C A L D W E L L

September 22, 1998

Mr. Ken Lanfear
Director of Public Works
City of Florence
250 Highway 101
Florence, Oregon 97439

13-4686

Dear Mr. Lanfear:

We are pleased to present the completed Florence Water Facilities Plan. It has been a pleasure working closely with you and your staff on this project. This Plan is an update of the draft completed in October 1997. We have addressed comments on the draft and made other minor revisions to bring the plan up-to-date.

Brown and Caldwell looks forward to working with you in the implementation of the recommended plan. Thank you for the opportunity to be of service.

Very truly yours,

BROWN AND CALDWELL



Jon Beer
Project Manager

JDB:ps

EXECUTIVE SUMMARY

The City of Florence has elected to take a long-range view of their potable water system needs for the next 20 years. Through this water system facilities plan, the groundwork is laid for major expansions to Florence's water sources, treatment, distribution, and storage. This plan begins with a detailed assessment of Florence's current water system. Future water needs for the study area are described next. The following section explains the regulatory requirements which govern operations of both existing and future facilities. Alternatives are developed in the next section, which includes both groundwater and surface water sources and treatment. Finally, the alternatives are evaluated and a recommended plan is presented.

ASSESSMENT OF EXISTING FACILITIES

The City of Florence is jointly served by facilities within Florence, as well as the Heceta Water District.

CITY OF FLORENCE

Florence is currently supplied with water from a well system which produces relatively high levels of iron. The city operates six of their seven wells a time. Chemicals are added to oxidize the soluble iron in water which passes through six pressure filters. These filters contain greensand media which remove the iron and other sediments. The facility produces an average of 1.0 million gallons per day (mgd) with a peak capacity of 1.8 mgd. Upon inspection, the oldest filter was noted to be corroded and inefficient. Opportunities were found to optimize valving, controls and chemical feed systems.

Storage is provided in four reservoirs—an elevated 250,000 gallon tank near the city shop which is slated for demolition, a 500,000 gallon steel tank on the east hills, and two 2,000,000 gallon tanks recently constructed near the Sand Pines Golf Course.

HECETA WATER DISTRICT

Florence relies heavily upon the Heceta Water District for water supplies to supplement their production. Annually, over half of the District's production is sold to Florence. Water sales to Florence can vary from approximately 45 to 200 million gallons per month. During the summer months, over half of Florence's peak needs are met by the District.

Heceta operates a surface water facility on Clear Lake. High water quality in the lake has meant that no treatment other than disinfection has been necessary. Soon new regulations will require some type of advanced treatment such as filtration. The maximum daily flow of this facility is 2.0 mgd. The District includes four reservoirs totaling about 1.8 gallons of storage.

Heceta's distribution system extends from Florence's city limits north approximately 6 miles. Their system includes about 45 miles of piping. There are currently about 1,700 connections to the District which represents a service population of 4,240, exclusive of Florence.

FUTURE WATER SYSTEM NEEDS

Future water supply needs are determined in part by growth projections for the study area. Growth projections are then used to predict future water consumption patterns. The study area encompasses the proposed Florence UGB. The proposed UGB is the same as the existing UGB, with the exception of two additions on the east side.

POPULATION

Within this study area, the current service population is estimated to be 6,401. Service is currently limited to areas within the city limits. The estimated population for the entire UGB is 7,856. An average annual growth rate of 3.5 percent was assumed for the study period. This is within the 2.3 to 3.7 percent range assumed for the Comprehensive Plan. The projected populations presented in the table below are based on applying the assumed growth rate to the current estimated populations. Because the water service area is expected to expand to the entire UGB, the design population is selected as the projected UGB population of 17,937.

Florence City and UGB Population Projections.

Year	Population*	
	City	UGB
1996	6,401	7,856
2020	14,617	17,937

Note: *Projections based on 3.5 percent annual growth rate.

BASIS OF PLANNING

The function of a water system is to deliver water in adequate quantity and quality at an acceptable pressure from the source of supply to the customers. The water supply system should be capable of meeting all demands without reducing pressure below acceptable limits. The following standards are used to guide the city in development of a high quality water system which meet regulatory requirements.

WATER SYSTEM DEMAND CRITERIA

Annual average per capita demand	188 gpcd
Peak month to annual average peaking factor	1.44
Peak day to annual average peaking factor	1.75
Peak hour to annual average peaking factor	3.50
Fire flow, Q, gpm as a function of population P, thousands	$Q=1020(P)^{1/2}(1-0.01(P)^{1/2})$
Fire flow duration, hours	15

Fire reserve volume, V, mg

$$V=(15/24)Q$$

Maximum system flow is peak hour or peak day plus fire flow, whichever is greater.

Assume system losses are 10 percent of the total demand

CURRENT WATER DEMAND

The average daily per capita usage is 182 gallons per person per day which compares well with prior studies. Comparing the historical maximum monthly flow and the peak day flow to the annual average flow yields peaking factors of 1.44 and 1.75 respectively. Maximum hour peaking factors are not available as Heceta monitors the amount of water sold to Florence on a monthly basis. A peak hour to average annual flow peaking factor of 3.50 was selected. This is the same peaking factor used in the 1992 Water Facilities Plan and is typical for many municipalities.

Unmetered water use accounted for 15.1 percent of all water produced from 1988 to 1990. It is expected that this percentage may be slightly reduced in the future as leaks are repaired and higher quality home service connections are installed. Unmetered water usage is included in the following projections for future water demands.

FUTURE WATER DEMANDS

Using the population projections developed earlier and applying the demand factors shown above, future water use projections can be developed. The maximum month and maximum day usage for the year 2020 are projected to be 4.86 and 7.0 respectively.

REGULATORY REQUIREMENTS

Oregon State drinking water requirements are established in Oregon Administrative Rules (OAR) 333-061 Public Water Systems. The OARs are administered by the Oregon Department of Human Resources Division of Health (Division). These rules are based upon the original Environmental Protection Agency (EPA) Safe Drinking Water Act (Act) enacted in 1986 and subsequently revised. The last major revision to the Act was in 1996.

Responsibilities of Water Suppliers, OAR 333-061-0025, defines the requirements for Florence and the Heceta Water District as purveyors of drinking water. Requirements for sampling, reporting, recordkeeping, complaint investigation, and operator training are explained.

Maximum Contaminant Levels and Action Levels, OAR 333-061-0030, defines the acceptable limits for inorganic, organic, and volatile organic contaminants.

TURBIDITY

The maximum contaminant levels for turbidity for all nonfiltered public water systems using surface water sources or groundwater sources under the direct influence of surface water is 5 NTU unless a specific variance has been granted by the Division.

Water supplies from systems employing conventional filtration treatment must be less than 0.5 NTU for at least 95 percent of all monthly samples and may exceed 1 NTU in no more than 5 percent of monthly samples.

For water systems using slow sand filtration and diatomaceous earth, the turbidity level must be less than 1 NTU in at least 95 percent of the samples taken each month and at no time shall the turbidity exceed 5 NTU.

Systems using other filtration technologies must meet the same turbidity limits as slow sand filtration and diatomaceous earth.

MICROBIOLOGICAL CONTAMINANT LEVELS

Microbiological contaminant levels are based upon the presence or absence of total coliforms in a sample. For a system which is required to collect 40 or more samples per month, total coliform positive samples shall not exceed 5.0 percent of the samples collected each month. For systems that collect fewer than 40 samples per month, no more than one positive sample is allowed each month.

SECONDARY CONTAMINANT LEVELS

A number of secondary contaminants could adversely affect water quality. these include color, corrosivity, pH, copper, hardness and iron. the secondary contaminate level for iron is of particular interest to the city of Florence as all the current and future groundwater sources produce iron bearing waters.

STANDARDS FOR SURFACE WATER, GROUNDWATER UNDER SURFACE WATER INFLUENCE, AND GROUNDWATER

Beginning January 1, 1992, OAR 333-061-0032 required that all surface water systems and groundwater systems under the influence of surface water had to meet a requirement of at least 99.9 percent (3-log) removal and/or inactivation of *Giardia lamblia* cysts and a 99.99 percent (4-log) removal and/or deactivation of viruses.

The OARs make major distinctions between filtered and nonfiltered treatment systems with regards to plant performance and monitoring.

TREATMENT REQUIREMENTS AND PERFORMANCE STANDARDS FOR CORROSION CONTROL

Corrosion control is mandated by the Division under OAR 333-061-0034 to minimize the potential for the release of harmful metals into a public water supply. Medium-sized water systems (serving 3,301 to 50,000 persons) were to conduct initial sampling by July 1, 1992, to determine if corrosion control is necessary. A water system which exceeded the lead or copper action level was required to implement optimal corrosion control. A water system which exceeds the lead or copper action levels must also provide public notice through a public education program.

In the case of Florence, testing has shown consistent compliance with this standard. Heceta Water District has reportedly not complied with this standard on occasion.

VOLUNTARY WELLHEAD PROTECTION PROGRAM

Protection of Oregon's groundwater resources is the primary goal of the Oregon Wellhead Protection Program. Established under the original EPA Safe Drinking Water Act, this program is jointly administered by the Department of Environmental Quality and the Health Division. Under this voluntary program, a community identifies the recharge area for its ground water supplies, determines the potential sources of contamination and makes decisions regarding how the groundwater resource will be managed.

FUTURE REGULATORY IMPACTS

The 1996 revisions to the Safe Drinking Water Act will strongly influence the operation of both existing and future water treatment and distribution systems in Oregon. Over the next 8 years, the Division expects to implement new standards for disinfection, radon, arsenic and sulfate.

Over the next 6 to 12 months, new and expanded programs will be promulgated by the EPA will also include a state revolving fund (SRF) program for water system construction much like the wastewater SRF program. Nineteen million dollars per year is expected through the year 2003. This money is earmarked for water system construction and implementation of new and existing regulatory programs.

IMPACT OF REGULATIONS

Current and future regulations will highly influence both the near-term and long-term planning decisions by Florence. Requirements for maximum contaminant levels, turbidity, disinfection, sampling, operations, and monitoring will influence selection of appropriate technology. At present, the city effectively addresses all the maximum contaminant level requirements. In particular, the secondary contaminant limit for iron at 0.3 mg/L will continue to be a major factor when evaluating treatment options. The current requirements for unfiltered surface water systems apparently will preclude using Clear Lake as a surface water source without filtration. The upcoming enhanced surface water treatment rules will likely reduce allowable turbidity from 0.5 NTU to 0.1 NTU. More stringent operational control of such process streams as backwash flows are also expected under the enhanced rules.

ALTERNATIVE DEVELOPMENT

Design criteria are presented based upon the evaluation of historical data. These criteria are then used to develop new water source alternatives. The alternatives evaluated included an expansion to the existing water treatment facility, an expanded treatment plant on Clear Lake, and a new groundwater source in the northwestern portion of the UGB. An analysis of both the distribution system and storage facilities was conducted to identify deficiencies.

As established above under Demand Criteria, the water source and treatment alternatives must be able to meet the projected maximum day demand with 10 percent of the capacity out of service. The sizing requirement for the maximum day source and treatment is then $(1.1)(7.0 \text{ mgd}) = 7.7 \text{ mgd}$.

It was estimated that 3.0 mgd is a reasonable capacity to expect from the existing treatment system following optimization and expansion. The Clear Lake Water Source Assessment dated May 21, 1997, showed approximately 2.5 mgd could be withdrawn from Clear Lake during the summer months while limiting drawdown to approximately 1.5 feet. For the purposes of this study, it is assumed the peak summer capacity of Clear Lake is 2.5 mgd.

Based on this information, two options are evaluated to determine the most appropriate combination of water sources and treatment to provide the required 7.7 mgd. These options are summarized below:

Treatment Capacity Expansion Options

Item	Capacity, mgd	
	Option 1	Option 2
Expanded Florence treatment plant	3.0	3.0
New Clear Lake surface water plant	2.5	—
New groundwater source and treatment	2.2	4.7
Total	7.7	7.7

NEW WATER SOURCE AND TREATMENT ALTERNATIVES

Three alternatives for providing additional water supplies include expansion and optimization of Florence's existing water treatment facility, development of the Clear Lake surface water supply, and development of a new groundwater supply.

Florence Water Treatment Plant Optimization and Expansion

It was estimated that the existing facilities could reliably produce 2.5 mgd following optimization. Additional treatment facilities would be required to expand the plant capacity to 3.0 mgd. The improvements and costs associated with plant optimization include modifications to the horizontal

contact chamber, upgrades to the filters and associated valves and piping, and changes to the chemical feed system and the wash water ponds. The estimated cost for plant optimization is \$371,400.

Plant expansion will be required to provide a total of 3.0 mgd treatment capacity at the existing facility. Additional wells and greensand filters will be required. Seven existing wells yield approximately 220 gallons per minute (gpm) each for a total of 1,540 gpm (2.2 mgd). Three more wells will be necessary to provide sufficient raw water capacity. These wells will be constructed with approximately the same depth and diameter as the seven existing wells.

Three additional greensand filters are also recommended. The costs for the wells, filters, and associated appurtenances is \$1,065,000.

New Clear Lake Treatment System

A new water treatment facility at Clear Lake would be designed to provide a minimum of 2.5 mgd during summer periods. At least 3 mgd would be produced by the facility during winter periods when lake level drawdown is not a concern. The raw water pumps would be equipped with variable frequency drives to allow variable production.

Four treatment alternatives were considered in this evaluation. These include direct filtration, diatomaceous earth filtration, microfiltration, and ultrafiltration. Given the pristine raw water quality of Clear Lake, any of these technologies would meet the requirements of the drinking water standards. The most appropriate choice can therefore be determined by a present worth analysis which considers both the capital and long-term operational cost of each treatment alternative. After selection of the most economical surface water treatment technology, a complete treatment system alternative was developed for a cost comparison to the groundwater source and treatment option. A brief description of each surface water treatment alternative is provided below.

Direct Filtration. The direct filtration process for treatment of surface water consists of three basic unit processes. The first process disperses and rapidly mixes coagulant chemicals with raw water. After mixing, water is conveyed to a flocculation zone where floc particles (destabilized colloidal particles) are allowed to form and coagulate into larger, filterable particles. The final unit process is filtration where floc particles are physically removed.

Diatomaceous Earth Filtration. Diatomaceous earth (DE) filtration consists of a pressure filter in which the filtration media is a layer of diatomaceous earth. DE is composed of silicate skeletons of diatoms which are mined from open pits. It is injected as a slurry into the raw water stream ahead of the DE filter. This slurry forms a porous DE cake which physically strains particulate matter from the raw water.

Microfiltration and Ultrafiltration. Both microfiltration and ultrafiltration are relatively new filtration processes that rely on porous membranes to physically strain particles in water. These systems use thin sheets of synthetic organic materials (cellulose acetate, for example) that have very small pore sizes capable of screening protozoan cysts and bacteria.

The total capital costs of the four alternatives are shown below. It is clear from the table that the direct filtration alternative has the lowest capital cost.

**Capital Cost Summary for
Surface Water Treatment Alternatives**

Treatment alternative	Capital cost, dollars
Direct filtration	4,058,000
Diatomaceous earth filtration	5,577,000
Microfiltration	8,543,000
Ultrafiltration	10,249,000

Of equal importance to capital cost, is the annual operating cost. The annual operating costs for these four alternatives ranged from a low of \$186,680 for the direct filtration alternative to a high of \$281,031 for diatomaceous earth. These costs are based on a plant flow of 3 mgd and are expressed in 1997 dollars. Since direct filtration was the lowest cost alternative based on both capital and annual operating cost, a present worth analysis was conducted to determine the life cycle cost for comparison with exclusively groundwater treatment as shown below.

Capital, Annual, and Present Worth Costs of Direct Filtration System

Capital cost, dollars	Total annual cost, dollars/year^a	Present worth cost, dollars^b	Dollars/1,000 gal treated (at 3 mgd flow)
4,058,000	600,000	5,891,000	0.55

Notes: ^a Includes amortization of capital cost.

^b Present worth based on 8 percent discount rate and 20-year period.

New Groundwater Source and Treatment

Based on previous hydrogeological studies, groundwater resources are probably sufficient to meet expected water supply demands. Water quality is generally acceptable with the exception of high iron concentrations.

The source of the iron is believed to be solubilization of the iron from low pH water created by the breakdown of organic material in the dunal sands. High iron increases water supply costs because of water treatment costs and well redevelopment costs due to increased fouling of the groundwater supply wells. Therefore, the primary factor in selection of new wellfield will be minimizing iron concentrations in the raw water. In addition, new groundwater supply development should occur outside of the hydrologic boundaries of Clear Lake.

Based on groundwater sample results for iron in groundwater, there appears to be a correlation between open dunal sands and reduced concentrations of iron in groundwater. Potential wellfield areas that have reduced iron concentrations and are outside of the hydrologic boundary of Clear Lake include:

- The area west of the junction of Munsel Lake Road and U.S. Highway 101.
- The area west and east of Mercer Lake Road and U.S. Highway 101.

New groundwater wells should be specifically designed to minimize iron concentrations and the need for maintenance due to clogging. Results of limited groundwater sampling indicate that iron concentrations may be lower in the upper zone of the aquifer than at depth. Well screening can be avoided at specific depths which are prone to high iron concentration. The total depth of the well should be limited to the depth of the dunal sands (approximately 100 to 150 feet below the ground surface). Higher groundwater quality can be maintained by using more wells drilled to shallower depths. These wells would probably produce approximately 200 to 250 gpm.

Two groundwater supply systems are developed for comparison. A 2.2 mgd facility would be needed in conjunction with the Clear Lake supply. A 4.7 mgd facility would be needed if the Clear Lake supply were not available. Assuming approximately 220 gpm from each well, 7 wells would be needed to supply a 2.2 mgd facility and 14 wells needed to supply 4.7 mgd. As an initial assumption, 1,000 foot spacing is planned between each well. A buffer of 500 feet is assumed around the perimeter of the wellfield to provide a measure of wellhead protection. A total of 150 acres of land is included for the 2.2 mgd wellfield and 300 acres of land included for the 4.7 mgd wellfield.

Wellhead Protection. In development of a new wellfield, it is important to evaluate the potential for contamination by performing a well head protection study. This evaluation is an assessment of potential contamination sources in a production well capture zone and focuses on protecting the aquifer within this area. Because groundwater is present at shallow depths (less than 50 feet) and the dunal sands have high permeability, it is likely that an accidental release of contamination at the surface would result in groundwater contamination.

Groundwater Treatment and Costs. Although low iron levels are expected, some degree of iron removal should be anticipated. Greensand iron removal technology is assumed for these alternatives. This is the same technology currently used by the city at their existing water treatment plant.

The following table shows the estimated capital, annual, and present worth costs associated with both a 2.2 mgd and a 4.7 mgd groundwater facility.

Present Worth Costs for Groundwater Treatment Systems

20-Year life cycle ^a				
Treatment capacity, mgd	Capital cost, dollars	Total annual cost, dollars/yr	Net present worth, dollars	\$/1,000 gallons treated (at 3 mgd)
2.2	3,566,000	598,000	5,874,000	0.75
4.7	7,006,000	1,166,000	11,446,000	0.68

Notes: ^a Life cycle costs are calculated based on 8 percent interest rate.

To determine the most cost effective combination of surface and groundwater, the present worth costs of both surface and groundwater treatment are compared below.

Present Worth Cost Comparison of Options

Option	Description	Capital cost, dollars	Total equivalent annual cost, dollars/yr ^a	Present worth cost, dollars ^a
Option 1	Clear Lake surface water—2.5 mgd Groundwater—2.2 mgd	7,624,000	1,198,000	11,765,000
Option 2	Clear Lake surface water—0 Groundwater—4.7 mgd	7,006,000	1,166,000	11,446,000

Notes: a Present worth and equivalent annual costs based on 8 percent interest rate over 20 years.

The total present worth costs for both options are between 11 and 12 million dollars. Although the present worth cost for Option 2 is slightly lower, the 3 percent difference between the costs of the two options is not significant. Noncost criteria become particularly important and are considered at the conclusion of this study.

WATER DISTRIBUTION SYSTEM EVALUATION

The existing water distribution system was analyzed using a computer model called CYBERNET was used to simulate present average day demand and maximum water use conditions. The model shows the existing distribution system maintains pressure above 30 psi at the peak hour demand except in a few localized areas where pressures fall below 30 psi.

Multiple computer runs were made to assess the future system analysis by inserting various new piping configurations into the existing system. It was determined that with the addition of two new 12-inch distribution lines running from east to west across the northern service area, adequate pressures could be maintained in the future. It is recommended that basic two level pressure zone system be retained and expanded. A booster pumping station will also be required on 42nd Street.

STORAGE SYSTEM IMPROVEMENTS

Storage must be provided in reservoirs to accommodate peak hour flows as well as fire flow. The current requirement for storage 5.5 million gallons. This storage is based on 3 days of annual average demand plus fire flow for 15 hours. The current storage capacity totals 4.75 million gallons. At present, the city relies on additional storage from the Heceta water system when the intertie with their system is operative.

By the year 2020, a total of 11.2 million gallons of storage will be required to serve the study area—an increase of approximately 6.5 million gallons. This storage should be distributed throughout the service area to maximize reliability in the event of fire or natural disaster.

The city may have the opportunity to acquire property from the county immediately to the west of Clear Lake. This site includes a ridge near an existing Heceta Water District reservoir. Other possible locations are within the limits of a new wellfield in the northeastern portion of the study area. Typically, steel tanks are more cost effective to construct in the smaller sizes. However,

they often require painting every 5 to 7 years, especially in a marine environment. Concrete tanks require little maintenance but usually have a higher first cost, depending upon the availability of an experienced concrete tank contractor.

RECOMMENDED PLAN

The recommended plan includes extensive improvements and expansions to the water treatment, distribution, and storage systems.

WATER TREATMENT IMPROVEMENTS

Water treatment improvements will include optimization and expansion of the existing groundwater treatment plant, a new surface water source on Clear lake, and a new groundwater source in the northern portion of the study area.

Optimization and Expansion of the Existing Water Treatment Plant

This option includes modifications to the following process areas in the existing plant:

- Contact chamber
- Filters
- Chemical feed system
- Operations building

The total cost of these improvements is approximately \$371,400 and will result in increased reliability, safety, as well as higher output from the existing units. It is expected that production from the existing units can be increased by 30 percent with a reduction in chemical use and operator attention.

In addition, three new wells will be added providing approximately 0.5 mgd additional source capacity, with one well in reserve. Three greensand filters, in conjunction with the modified existing filters, will provide a total of 3.0 mgd with one filter out of service. The cost for the new wells and filters is expected to be approximately \$1,065,000. The total project cost for the plant optimization and expansion is \$1,436,400.

New Water Supply and Treatment

Limitations on the aquifer at the existing treatment plant site will require that at least one additional source be developed. Clear Lake was assessed as a potential surface water source and it is expected that 2.5 mgd can be withdrawn from the lake during peak summer periods without excessive drawdown. With such a plant on Clear Lake, 2.2 mgd additional capacity would be required by the end of the planning period to provide a maximum of 7.7 mgd. Without a plant on Clear Lake, 4.7 mgd of additional capacity will be required. A summary of the life cycle costs associated with each option is shown in the present worth summary below:

<u>Option</u>		<u>Present worth</u>
1	A 2.5 mgd surface water facility on Clear Lake supplemented with a 2.2 mgd groundwater source and treatment system	\$11,765,000
2	A 4.7 mgd groundwater source and treatment system	\$ 11,446,000

Evaluation of Options. The present worth costs of each option are essentially the same. Noncost factors should then weigh heavily in the selection of the recommended option. Environmental impacts, ease of implementation, reliability, permitting, and regulatory aspects, flexibility, and aesthetics should all be considered.

Environmental Impacts. Both options can be developed to minimize environmental impacts. During severe drought periods, an option using Clear Lake would need to address the potential for drawdown in the lake. This would be managed by limiting water withdrawals during critical periods. An option relying solely on groundwater could potentially pose concerns for environmental impacts to local private wells depending upon localized influences to the aquifer.

Ease of Implementation. Although the Division of Health has mandated that filtration be provided to any surface water source on Clear Lake, it is not clear that an agreement can be reached between the city and Heceta Water District. Yet siting a new wellfield in the north Florence area, even a 150-acre facility, may also pose a significant challenge. For this reason, both alternatives could be ranked equal with regards to ease of implementation.

Reliability. Being able to rely on both surface and groundwater sources would be ideal for a water utility. Contamination of both type sources simultaneously would be highly unlikely. However, it is unknown what drawdown due to pumping would be considered acceptable to all parties associated with Clear Lake. If the drawdown level was less than the 2.5 feet assumed, the economics of the Clear Lake alternative would shift. An option which includes two widely separated ground water systems would still offer significant reliability. The technologies associated with treatment of both surface and groundwater are equally reliable.

Permits and Regulatory Aspects. Development of a wellfield will require a groundwater protection plan regardless of size. No major permitting difficulties are anticipated in development of a surface water plant on Clear Lake.

Flexibility. A surface water plant on Clear Lake would increase the flexibility of the Florence water supply program. Contamination from an accidental spill in the region of a wellfield could curtail pumping for an extended period. A surface water source could add

to the diversity of Florence's water supply. However, with the dual wellfield option, the separation of the existing and new wellfields and the large number of wells proposed would offer less, but adequate flexibility.

Aesthetics. A new water treatment plant on Clear Lake would require visual screening and architectural treatment to blend with the natural surroundings. Similarly, a new wellfield and groundwater treatment facility should also be designed to fit into the surrounding area. The larger size of the groundwater system may pose a more significant challenge.

Selection of Recommended Supply and Treatment Option. The ranking of the options with respect to the criteria discussed above are summarized below. A ranking of 1 is the highest.

Summary of Treatment Alternative Rankings

	Option 1 - Clear Lake and 2.2 mgd wellfield	Option 2 - 4.7 mgd wellfield
Environmental impact	2	2
Ease of implementation	2	2
Reliability	2	3
Permits and regulatory aspects	2	2
Flexibility	2	3
Aesthetics	2	2
Total	12	14

The above table marginally supports the selection of Option 1, a surface water treatment plant on Clear Lake with a new 2.2 mgd wellfield in the north Florence area. Assuming the necessary political and institutional agreements can be reached, this alternative would be the preferred choice. However, with costs and noncost criteria ranking so closely, either alternative should be considered workable.

The total capital cost for the surface water plant and the 2.2 mgd wellfield would be approximately \$7,624,000. The surface water plant would be equipped with a laboratory, chemical storage and feed areas, and a filtration process area. It is assumed that a full basement clearwell would be utilized to provide the required chlorination detention time.

Provisions should be made during the design process to maintain continued operation of the existing Clear Lake facility during construction. After the new surface water plant is completed, the entire Heceta Water District will be served by the new Clear Lake facility. Most of the planned expansion within the Heceta Water District is included in the Florence study area. The Water Facilities Plan for Heceta Water District projects the peak flow requirements for the District outside of the Urban Growth Boundary to be only 0.48 mgd or about 6 percent of Florence's future water demand. These flows could be readily accommodated within the recommended plan.

The wellfield and groundwater treatment facility would be similar to the existing facility. It is expected that larger filter units will be used in the new facility to minimize the number of units and the associated piping and controls costs. The current chemical feed, flow measurement, and filter operations would be refined in designing the new groundwater treatment system.

FUTURE WATER DISTRIBUTION SYSTEM

According to both the distribution modeling effort and discussions with city staff, the existing water distribution system is essentially adequate. Minor modifications are necessary to address current localized low pressure areas. As growth continues, the majority of the new connections will be in the north Florence area. At present, a serviceable distribution grid exists in this area.

Two east-west interties will be necessary to meet peak hour demands at the design year. One intertie will extend west approximately 5,400 feet from 43rd and Highway 101 west to Rhododendron Drive. The other will extend approximately 7,700 feet west from Heceta Beach Road east to Highway 101. The project costs for these mains are shown below.

Water Distribution Main Improvements Project Costs

	43rd/ Rhododendron	Heceta Beach/ Hwy 101	Combined project costs
Pipeline construction, \$70/linear foot	378,000	540,000	
Contingency, 25 percent	95,000	135,000	
Subtotal	473,000	675,000	
Engr, admin, legal, 20 percent	95,000	135,000	
Total project cost	568,000	810,000	\$1,378,000

With these additions, adequate pressures can be maintained when future development infills normal distribution piping within the north Florence area.

FUTURE STORAGE RESERVOIRS

Providing a diversity of storage is key to maintaining a reliable municipal water supply system. When the 250,000 gallon reservoir is removed near the city shop, all the city storage will be located in two locations. While this situation is adequate for current needs as long as the Heceta Water District storage is intertied, future growth in the north Florence area will require additional storage in the north portion of the city. Approximately 6.5 million gallons (mg) of new storage will be required by the end of the planning period. The actual locations and sizing of future reservoirs must be selected in response to actual growth patterns. However, there are two likely locations which should be given strong consideration as development proceeds.

Recognizing high level storage is always preferable, the city may wish to purchase county property that includes a ridge above Clear Lake. The proposed surface water treatment plant could pump directly to a 2.0 mg reservoir at this location. There is a limited amount of high level ground in the north Florence area to site additional reservoirs. An option would be to site the

remaining reservoirs adjacent to the new north Florence wellfield discussed previously. These reservoirs would be equipped with pumping stations to maintain pressure throughout the first level pressure zone. The costs for these facilities are shown below.

Storage Reservoir Costs

Item	Cost
2.0 mg Reservoir	520,000
(2) 2.25 mg Reservoirs	1,170,000
(1) Pumping station	150,000
Subtotal	1,840,000
Contingency, 25 percent	460,000
Construction subtotal	2,300,000
Engineering, administration and legal, 20 percent	460,000
Total project cost	2,760,000

RECOMMENDED PLAN COSTS

Cost have been developed for each of the components of the recommend plan. A description of the improvements and a summary of the project costs are presented below.

Water Supply and Treatment

Existing water plant optimization	\$ 371,400
Existing water plant expansion	\$1,065,000
New Clear Lake surface water treatment	\$4,058,000
New north Florence wellfield	\$3,566,000

Water Distribution

43rd/Rhododendron	\$ 568,000
Heceta Beach/Hwy 101	\$ 810,000
42nd St. booster pumping station	\$ 30,000

Water Storage

2.0 mg reservoir near Clear Lake	\$ 780,000
Two 2.25 mg reservoirs in north Florence area	\$1,755,000
Booster pump station	\$ <u>225,000</u>

Total estimated costs for recommended improvements including contingency, engineering, administration, and legal costs	\$13,228,400
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PHASING OF RECOMMENDED IMPROVEMENTS

The need for new supply and storage will be determined to a large degree by growth within the study area. A following phasing plan would represent a logical progression of improvements:

Phase 1 would begin immediately and include the plant optimization and the new treatment facility on Clear Lake. It is assumed that the Health Division will approve of this timing and allow the existing Clear Lake facility to operate in the interim. At least a 2.0 million gallon reservoir would be built near Clear Lake during the same project. Design and construction of these improvements could be completed by the end of the year 2000. The total estimated project cost would be approximately \$5,209,400.

Phase 2 would not be needed until approximately the year 2007. The existing treatment plant should be expanded at that time. Interties for the water distribution system will likely be necessary. A booster pumping station on 42nd Street will also be required. The land acquisition process for the new well field and reservoir location should be pursued during this period. Design and construction of these facilities would require approximately 2 years. The total project cost exclusive of land would be approximately \$2,473,000.

Phase 3 source capacity would not be needed until about the year 2010. This phase would include development of the wellfield and groundwater treatment plant, the two 2.25 million gallon reservoirs, and the booster pumping station. Two and one half years should be allocated for the design and construction of this facility. The total estimated project cost is \$5,546,000.

In the event it is not feasible for the city and Heceta Water District to develop a facility at Clear Lake, the phasing shown in the following figure could be modified to include a large wellfield in two phases in the north Florence area in lieu of a Clear Lake treatment plant and a smaller wellfield.

TABLE OF CONTENTS

CHAPTER 1 ASSESSMENT OF EXISTING FACILITIES

Introduction	1-1
Florence Water System Description.....	1-1
Florence Wellfield	1-1
Florence Water Treatment Plant.....	1-2
Treatment Plant Components	1-2
Treatment Process	1-3
Florence Distribution and Storage System	1-4
Chlorine Residual Evaluation	1-5
Assessment of Existing Treatment Facilities.....	1-6
Horizontal Contact Chamber.....	1-6
Filtration	1-7
Filter Description	1-7
Filter Productivity and Efficiency	1-7
Options to Improve Filter Efficiency.....	1-9
Chemical Feed Systems.....	1-12
Chlorine.....	1-12
Air	1-12
Potassium Permanganate.....	1-12
Caustic Soda.....	1-13
Soda Ash	1-13
Sodium Fluoride	1-13
Sodium Hexametaphosphate	1-13
Wash Water Ponds.....	1-14
Operations Building and Other Support Facilities	1-14
Heceta Water System	1-15
Treatment	1-15
Distribution.....	1-16
Storage	1-16
Water Quality Issues	1-16

CHAPTER 2 FUTURE WATER SYSTEM NEEDS

Water Study Area.....	2-1
Location and Boundaries.....	2-1
Planning Period.....	2-1
Physical Environment.....	2-1
Topography, Geology, and Soils	2-2
Climate	2-3
Water Resources.....	2-3
Socioeconomic Environment	2-5
Population	2-5
Land Use	2-7

TABLE OF CONTENTS-Continued

Basis of Planning	2-8
Water System Standards	2-8
Water System Demand	2-8
Water System Facilities	2-9
Water System Unit Costs	2-9
Current and Future Water Requirements	2-10
Peaking Factors	2-10
Current Water Usage	2-10
Future Water Demands	2-13

CHAPTER 3 REGULATORY STANDARDS

Responsibilities of Water Suppliers [OAR 333-061-0025]	3-1
Maximum Contaminant Levels and Action Levels [OAR 333-061-0030]	3-1
Turbidity	3-3
Maximum Microbiological Contaminant Levels	3-4
Secondary Contaminant Levels	3-4
Treatment Requirements and Performance Standards for Surface Water, Ground- water Under Surface Water Influence, and Groundwater [OAR 333-061-0032]	3-5
Nonfiltered Systems	3-5
Filtered Systems	3-6
Groundwater Systems	3-6
Treatment Requirements and Performance Standards for Corrosion Control [OAR 333-061-0034]	3-6
Voluntary Wellhead Protection Program	3-6
Future Regulatory Impacts	3-7
Impact of Regulations	3-8
Process Options	3-9

CHAPTER 4 ALTERNATIVE DEVELOPMENT

Design Criteria	4-1
Water Source and Treatment Alternative Sizing	4-1
New Water Source and Treatment Alternatives	4-3
Florence Water Treatment Plant Optimization and Expansion	4-3
Plant Optimization	4-3
Plant Expansion	4-5
New Clear Lake Treatment System	4-5
Description of Surface Water Treatment Alternatives	4-6
Estimated Costs for Surface Water Treatment Alternatives	4-7
New Groundwater Source and Treatment	4-12
New Groundwater Source	4-13
Wellhead Protection	4-14
Groundwater Treatment and Costs	4-14

TABLE OF CONTENTS-Continued

Water Distribution System Evaluation	4-16
Analysis Approach	4-16
Existing System Analysis	4-17
Future System Analysis	4-17
Storage System Improvements	4-18
Current Storage Requirements	4-18
Future Storage Requirements	4-18
Storage Costs	4-18
CHAPTER 5 RECOMMENDED PLAN	
Recommended Water System Improvements	5-1
Water Treatment Improvements	5-1
Optimization and Expansion of the Existing Water Treatment Plant	5-1
New Water Supply and Treatment	5-2
Future Water Distribution System	5-4
Future Storage Reservoirs	5-4
Recommended Plan Costs	5-5
Water Supply and Treatment	5-5
Water Distribution	5-5
Water Storage	5-6
Phasing of Recommended Improvements	5-6
APPENDIX A	Remote Chlorine Filter Brochure
APPENDIX B	Monthly Average Performance Data From April 1995 to December 1996
APPENDIX C	Clear Lake Water Source Assessment Technical Memorandum
APPENDIX D	Notice of Violation and Remedial Order
APPENDIX E	Distribution System Modeling Results

LIST OF FIGURES

	Page
1-1 Existing Treatment Process	1-3
1-2 Florence Distribution System Service Zones	*1-4
1-3 Florence Distribution System Chlorine Residual	*1-6
2-1 Florence Water Study Area	*2-2
2-2 Zoning Map of Florence Area	*2-8
2-3 Monthly Average Water Use (1992-96)	*2-10

* Follows page listed

TABLE OF CONTENTS-Continued

4-1	Possible New Well Field Locations	*4-14
4-2	Existing Distribution System Analysis	*4-18
4-3	Future Distribution System Analysis.....	*4-19
5-1	Proposed Clear Lake Surface Water Treatment Plant.....	*5-4
5-2	Future Florence Water System.....	*5-4
5-3	Florence Water Demand and Capacity Projections	*5-6

LIST OF TABLES

1-1	Florence WTP Filtration Performance-Average Monthly Values.....	1-8
1-2	Heceta Water District Production and Sales in Million Gallons.....	1-17
2-1	Florence Area Temperature Summary.....	2-4
2-2	Florence City and UGB Population Projections	2-6
2-3	Summary of Current Land Use Areas.....	2-7
2-4	City of Florence Monthly Average Water Demands (1992-1996)	2-11
2-5	City of Florence 1992-1996 Annual Average Water Demands.....	2-12
2-6	City of Florence Projected Water Demands (1997-2020).....	2-13
3-1	Inorganic Maximum Contaminant Levels	3-1
3-2	Organic Maximum Contaminant Levels.....	3-2
3-3	Volatile Organic Chemical Maximum Contaminant Levels	3-3
3-4	Secondary Contaminant Maximum Levels.....	3-4
4-1	Design Criteria Guidelines.....	4-2
4-2	Treatment Capacity Expansion Options.....	4-2
4-3	Estimated Plant Optimization Project Costs	4-4
4-4	Estimated Costs for Existing Plant Expansion	4-5
4-5	Estimated Project Costs for New Clear Lake Treatment System Direct Filtration Alternative.....	4-7
4-6	Estimated Project Costs for New Clear Lake Treatment System Diatomaceous Earth Filtration System	4-8
4-7	Estimated Project Costs for New Clear Lake Treatment System Microfiltration System.....	4-9
4-8	Estimated Project Costs for New Clear Lake Treatment System Ultrafiltration System.....	4-10
4-9	Capital Cost Summary for Surface Water Treatment Alternatives.....	4-10
4-10	Annual Costs for Surface Water Treatment Options.....	4-11
4-11	Chlorination System Present Worth Comparison	4-12

* Follows page listed

TABLE OF CONTENTS-Continued

4-12	Capital, Annual, and Present Worth Costs of Direct Filtration System.....	4-12
4-13	Groundwater Treatment System Capital Costs	4-15
4-14	Present Worth Costs for Groundwater Treatment Systems.....	4-15
4-15	Present Worth Cost Comparison of Options.....	4-16
4-16	Steel Storage Reservoir Construction Costs	4-19
4-17	Concrete Storage Reservoir Construction Costs.....	4-19
5-1	Summary of Treatment Alternative Rankings	5-3
5-2	Water Distribution Main Improvements Project Costs.....	5-4
5-3	Storage Reservoir Costs	5-5

CHAPTER 1

ASSESSMENT OF EXISTING FACILITIES

INTRODUCTION

In this chapter, Florence's existing water system is presented with a description of the water sources, treatment, distribution, and storage. The physical and chemical process used for water treatment are explained next. A detailed assessment of the Florence water treatment plant process follows. The assessment results in a number of detailed recommendations for plant optimization which are included in Chapter 5.

A brief summary of the water treatment, distribution, and storage systems of the Heceta Water District (District) are included since the District routinely supplies water to the City of Florence. Raw and finish water quality from existing groundwater and surface water sources are detailed at the close of this chapter.

FLORENCE WATER SYSTEM DESCRIPTION

The features of the existing Florence water supply, treatment, distribution, and storage systems are described below.

FLORENCE WELLFIELD

The Florence municipal well field is located on 80 acres adjacent to the Ocean Dunes Golf Course. The city currently utilizes 6 of 7 production wells. They were constructed beginning in the mid 1960s with the last four wells constructed in 1994. The construction specifications for most of the wells are shown below.

<u>Well No.</u>	<u>Total depth, ft.</u>	<u>Cased length, ft.</u>	<u>Screen length, ft.</u>	<u>Screen diameter, in.</u>	<u>Screen slot size, in.</u>
1					
2	120.0	56.5	63.5	10	0.01
3	141.5	151.5	30.0	12	0.008
4	143.0	82.5	40.0	12	0.008
5	143.0	78.0	40.0	10	0.008
6	140.0	78.0	40.0	10	0.008
7	182.0	119.0	40.0	10	0.008

The average yield of these wells is 220 gallons per minute (gpm) (0.32 million gallons per day (mgd)). The total yield from the wellfield is approximately 1,540 gpm (2.24 mgd) although all wells are not used simultaneously. Due to the presence of iron and other sediment, the well

screen sections must be cleaned periodically. When production decreases by approximately 20 percent, after about 5 years, each well is taken out of service, chlorinated, cleaned and surged to remove debris.

Each well is equipped with a multistage submersible pump and some of the wellheads are enclosed in fiberglass housings. Flow is controlled with a throttling discharge valve. Wells No. 4 through 7 are equipped with a flow meter with a 4-20 milliamp readout at both at the wellhead and in the plant control room. It is reported the local readouts are functional while the remote flow readouts at the plant are not reliable.

On occasion, some of the existing wells experience excessive drawdown which indicates insufficient production by the aquifer. In some cases, air is pulled into the casing and the affected well must be throttled to limit production. Output of wells in the throttled condition may be reduced by 20 percent.

FLORENCE WATER TREATMENT PLANT

The existing City of Florence Water Treatment Plant (WTP) is designed to remove high levels of iron and some manganese from the city's groundwater supply. Concentrations of iron in the raw water range from 5 to 6 milligrams per liter (mg/L). Finished water from the treatment plant has iron concentrations ranging from 0.2 to 0.3 mg/L.

Data from April 1995 to December 1996 shows that the existing plant net flow production ranged from 0.1 mgd to 1.8 mgd, with a 1996 annual average net flow production of 1.0 mgd. Approximately 9 percent of the throughput water was used for filter backwashing in 1996.

The most recent major treatment plant expansion was completed in 1992 with the addition of new pressure filters, new feed systems for adding potassium permanganate, chlorine, and sodium carbonate, and an air injection system for oxidation.

TREATMENT PLANT COMPONENTS

The existing WTP is located in Florence at the easterly end of Twenty-fifth Street, near the corner of Willow Street. The major water treatment plant components include a horizontal contact chamber for chlorine contact, injection ports for mixing of iron oxidation chemicals with raw groundwater, six pressure filters, chemical feed systems to feed chlorine, air, potassium permanganate, caustic soda, soda ash, sodium fluoride, sodium hexametaphosphate, and two wash water ponds to collect and decant backwash waste. Decanted backwash wastewater is discharged to a nearby stream.

The WTP Operations Building houses the electrical and instrumentation control systems, the filter control valves, all chemical storage and handling facilities, and a water quality laboratory.

TREATMENT PROCESS

The WTP at Florence is designed to reduce iron and manganese to acceptable concentrations for potable use. The current regulations limit iron concentrations to 0.3 mg/L and manganese to 0.05 mg/L. Raw groundwater is fed chemicals and filtered as shown in Figure 1-1. This figure also shows the cross section of the existing filtration system.

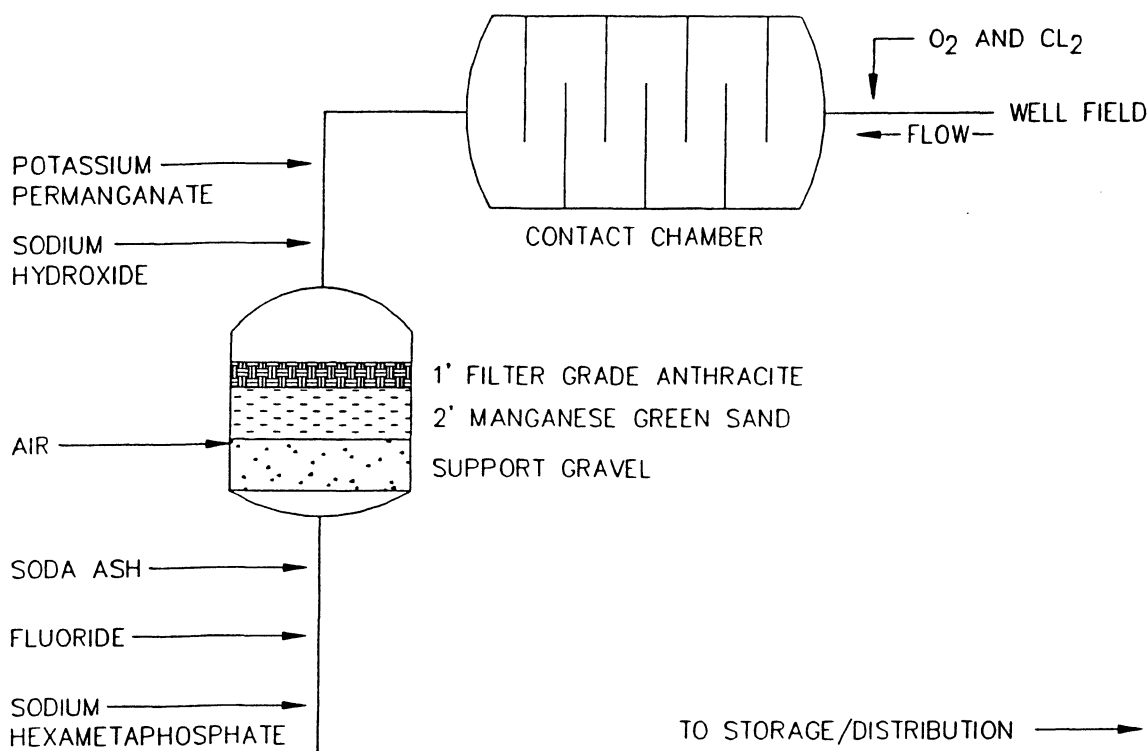
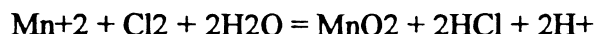
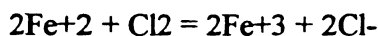


Figure 1-1. Existing Treatment Process

The treatment oxidizes soluble iron (II) to iron (III) and soluble manganese (II) to manganese (IV). Both iron (III) and manganese (IV) are insoluble and can be easily removed by filtration. Iron (III) is complexed with hydroxides to form ferric hydroxide floc and manganese (IV) form insoluble manganese dioxide precipitate. Both these oxidized forms of iron and manganese are subsequently removed by filtration. Typical chemical reactions for the oxidation of iron and manganese are as follows:

Oxidation with chlorine:



Oxidation with Air:



Oxidation with potassium permanganate:



Sodium hydroxide is added after the horizontal contact chamber to raise the pH to increase the oxidation rate of manganese.

Chemical addition causes the raw water to develop ferric hydroxide floc and manganese dioxide precipitates that are subsequently removed in the six downstream green sand filters. The filter media was originally capped with 12 inches of anthracite. Anthracite aides in the removal of manganese and can also help minimize surface blinding. Green sand filter media is a natural zeolite that can itself oxidize iron and manganese to insoluble and filterable states. This oxidative capacity is limited, however, and must be periodically regenerated by the addition of potassium permanganate. Adding potassium permanganate serves to oxidize iron and manganese and helps continuously regenerate green sand filter media.

In the filters, particles are trapped and removed from the water stream. Floc-laden water flows downward through the filter media. The filters must be periodically cleaned due to an accumulation of trapped particles which creates excessive headloss across the filters. Turbidity breakthrough results when particles are no longer retained in the filters and is another indicator that filter cleaning is required. Filters to be cleaned are taken off-line and are backwashed first by an air scour process that helps scrub and loosen particles attached to the filter media and then by a hydraulic backwashing process. The hydraulic backwash process flushes water in an up-flow mode across the filter media and conveys backwash waste into the two wash water ponds located adjacent to the WTP. Wash water is allowed to settle in the wash water ponds. The decanted clear water from the wash water ponds flows into a nearby creek.

After filtration, filtered water is dosed with sodium carbonate for final pH adjustment and corrosion control, sodium fluoride for fluoridation, and zinc hexametaphosphate to inhibit piping corrosion. From this point, finished water is conveyed to the city's water distribution system.

FLORENCE DISTRIBUTION AND STORAGE SYSTEM

Treated water leaving the plant site enters the city's distribution system for immediate consumption by customers or storage in the one of four reservoirs. Figure 1-2 shows the Florence water service area and pressure zones.

The urban area from Old Town north to Thirty-fifth Street is served by the low level pressure zone. Storage is provided by a 250,000 gallon elevated steel reservoir located next to the city shop on Spruce Street. Water is conveyed into the reservoir by submersible pumps at the city

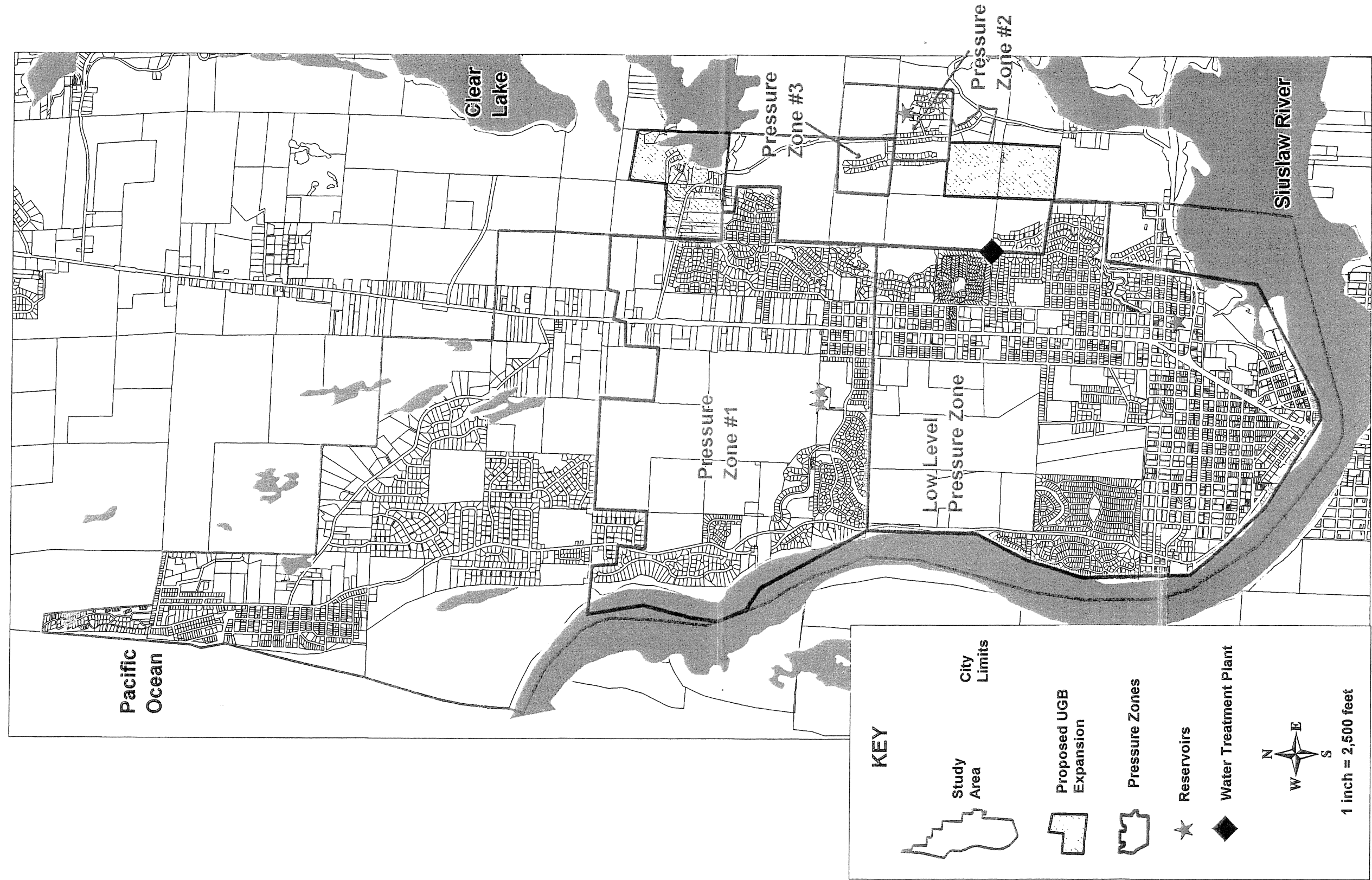


Figure 1-2. Florence Distribution System Service Zones

well field. The reservoir was constructed in 1948 and may be nearing the end of its service life. Minimum pressures in the low level pressure zone are in the 35 to 40 pounds per square inch (psi) range.

Ground elevations are higher in the northern portion of the service area which serves to lower the system pressure. High pressure zone No. 1 was established to serve the area from Thirty-fifth Street north to the city limits. Two 2 million gallon ground level steel storage tanks are located next to the Sand Pines Golf Course. Number 1 booster station is located adjacent to the two reservoirs. This station contains one 7.5-horsepower (hp) and two 40-hp pumps. The discharge pressure of this pumping facility is 60 psi.

High pressure zone No. 2 serves a small area along Munsel Lake Road to the east. This area is served by the 0.5-million-gallon East Side Reservoir. This ground level steel reservoir was built in 1965. The exterior condition of this reservoir is fair, while the condition of the interior coatings is unknown. Number 2 booster station is located next to the reservoir and contains one 20-hp and two 40-hp pumps. The discharge pressure from this number 2 booster station is 85 psi. System pressure is approximately 60 psi at the highest point of use.

High pressure zone No. 3 is located adjacent to high pressure zone No. 2. The booster station for zone No. 3 is located at the entrance to Ocean Dunes Golf Course. This booster station contains one 3-hp, one 5-hp, and one 15-hp pump. The discharge pressure from booster station No. 3 is 65 psi. System pressure is 60 at the highest point of use.

CHLORINE RESIDUAL EVALUATION

Oregon drinking water standards require a minimum 0.2 mg/L chlorine residual be maintained in the distribution system to prevent regrowth of any bacterial or viral contamination in the treated water. Even systems which use an alternate method of disinfection, such as ozone, typically add a low residual of chlorine to maintain water quality in the distribution system. Most treatment plants in Oregon chlorinate finish water to a concentration of about 1.0 mg/L. Florence discharges its water with a residual of about 0.8 mg/L. As the water remains in the distribution system, some oxidation occurs of residual organics, minerals, and other dissolved materials. This process consumes the residual chlorine so that at the ends of the distribution lines chlorine residual levels may be minimal.

One method to address this problem is to increase the initial dosage at the plant. This would likely result in taste and odor complaints from customers close to the plant. In addition, this approach would significantly increase the usage and cost for chlorine. An alternate method is to provide low levels of supplemental chlorination at key locations within the distribution system. Low residuals often result at the outlet of reservoirs during low usage periods and toward the end of long distribution lines. Chlorine can be added at booster pump stations, reservoirs, or directly into a distribution line.

For rechlorination, a weak sodium hypochlorite (NaOCl) can be used instead of chlorine gas. The safety issues relating to the use of chlorine gas in remote locations are obvious. A 12 percent solution of NaOCl will not pose a severe burn hazard, yet will cost effectively add chlorine to the

distribution system, NaOCl is available in 55-gallon drums and can be injected in a number of ways. Locations with electrical power can use a manually adjustable chemical feed pump similar to the numerous feed pumps currently in operation at the water treatment plant.

For locations without power, a flow paced chlorinator using a water metering device to power a NaOCl feed pump may be the appropriate choice. This equipment operates off of line pressure and can be located in a shallow vault adjacent to the distribution line. Although more expensive than simple electrical chemical feed pumps, this equipment can be ideal for isolated applications. For instance, a 20-gallon barrel of 12 percent NaOCl fed at a dose of 0.5 mg/L into a line conveying 0.2 mgd would last at least 2 months. A catalog cut of this self-powered flow proportional feeder is shown in Appendix A.

Figure 1-3 shows locations of recent chlorine residual readings in the Florence distribution system. The lowest reading are indicated in the northeastern portion of the system. While residual chlorine was noted in all cases, a few readings are borderline with regards to the current Health Division requirements of 0.20 mg/L. Supplemental chlorination addition in this area may be appropriate. This would alleviate potential problems during particularly low flow conditions.

ASSESSMENT OF EXISTING TREATMENT FACILITIES

This section of the report describes the condition and performance of the existing facilities at the Florence WTP. The description of the physical condition and performance characteristics of the existing facilities are organized on a unit process basis, generally taken in order of flow through the plant. The primary objectives of this evaluation was to:

1. Assess and document the physical condition of the existing treatment plant.
2. Evaluate the performance of key water treatment unit processes at the existing plant.
3. Develop recommendations to upgrade the existing plant to maximize production.

HORIZONTAL CONTACT CHAMBER

Raw water from the wells is pumped into the horizontal contact chamber where chlorine and air is injected for disinfection and oxidation of iron and manganese. This chamber has no moving parts and is internally baffled. Iron and manganese oxidation agents are added downstream of the horizontal contact chamber. A series of protruding chemical injection ports are attached to this chamber. A total of 20 minutes of contact time is available at 800 gpm according to a recent dye study.

The order in which chemicals are introduced affects the effectiveness of the chemical addition. Air and potassium permanganate generally should be fed in the system as early as possible to increase time available for reaction. Sodium hydroxide should be added ahead of the potassium permanganate to optimize pH for potassium permanganate oxidation of manganese. Currently these chemicals are added adjacent to each other on the same header.

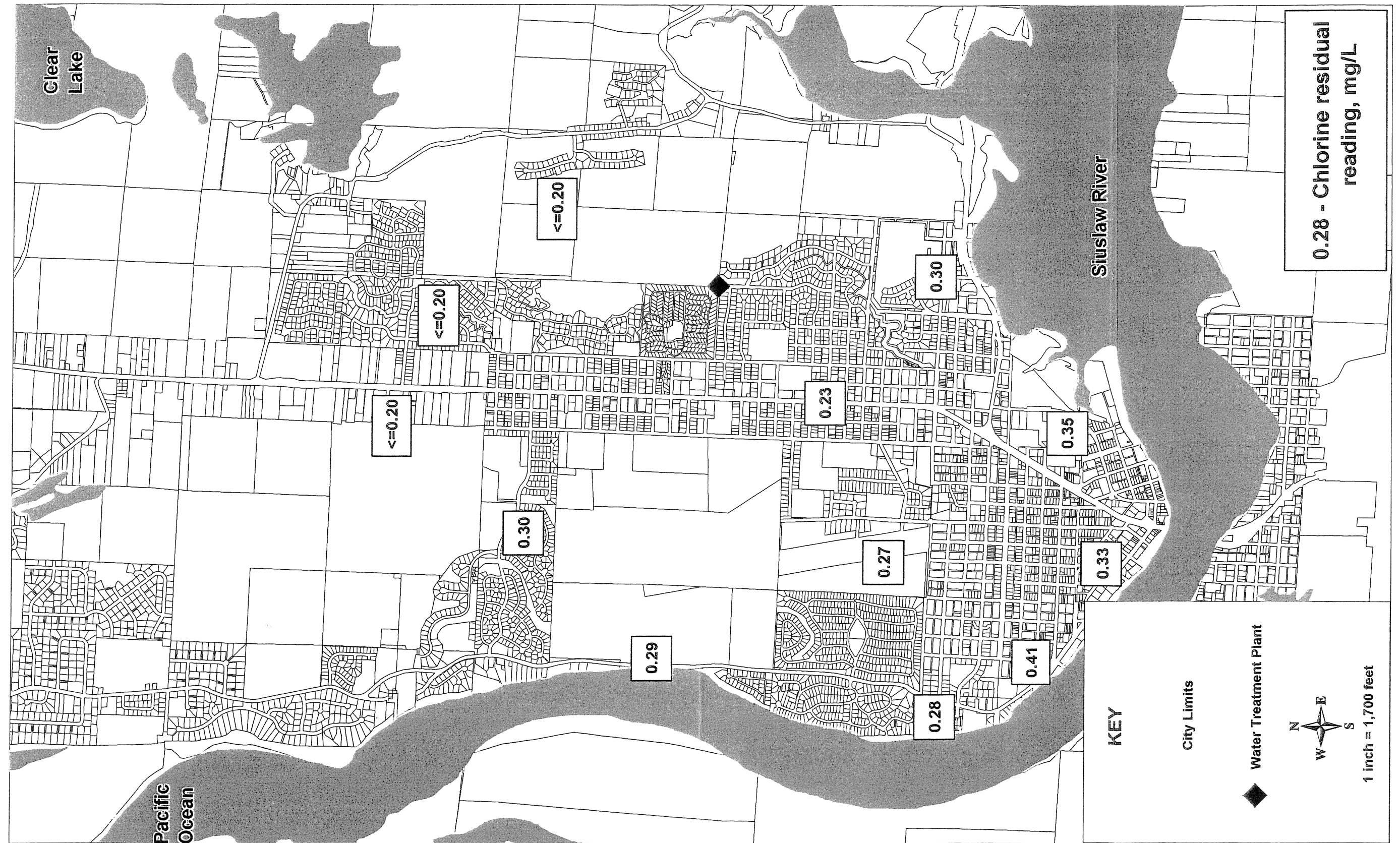


Figure 1-3. Florence Distribution System Chlorine Residual

FILTRATION

The characterization and assessment of the existing filters focus on several key parameters used to judge the overall effectiveness and efficiency of filtration. The first section of this evaluation describes the properties of the filtration system, the second section discusses productivity and efficiency in producing filtered water, the third section explains filter control systems, and the final section evaluates the filter backwashing systems.

Filter Description

Currently there are six outdoor mounted 11-foot-diameter pressure filters (each with a filter area of 95.0 square feet). Originally the filter media profile included 24 inches of green sand and a 12-inch-deep cap of 1-millimeter (mm) effective size anthracite coal. The green sand has an effective size of approximately 0.35 mm and a uniformity coefficient of 1.6. Over time, the anthracite media was lost during backwashing and has been replaced by additional green sand. Anthracite is generally used to remove the bulk of the insolubles so as not to blind the finer green sand media. The filter media is supported by 12 inches of support gravel placed over a pipe underdrain system. The support gravel prohibits the green sand media from clogging the underdrain pipe orifices. Several of the filters differ in height leading to differences in internal clearance between the media surface and the overhead manifold system.

Filters are periodically cleaned by air scour followed by hydraulic backwashing. An air scour manifold located between the support gravel and the green sand is provided in each filter. Similarly, a manifold system is provided on the top of each filter to distribute raw water into the filter and to collect backwash water from each filter. Upon backwashing, a filter-to-waste cycle is used to temporarily waste the initial filter effluent to the wash water ponds when the filter is first brought on-line. This initial flow generally contains high levels of turbidity, iron, and manganese, that must be rejected until proper filter operation is established. This filter-to-waste cycle is generally limited to the first 2 minutes of each filter run. All filter control valves are located inside the operations building.

Filter Productivity and Efficiency

Evaluating filter productivity and efficiency requires the consideration of multiple performance variables. One such variable is Unit Filter Run Volume (UFRV). UFRV is a parameter that is associated with filter efficiency and measures the total amount of acceptable water that is conveyed through a unit area of filter media per filter run. The units of UFRV are gallons per square foot per run. Filtration efficiency is defined by the following relationship:

$$\text{efficiency} = [\text{UFRV} - (\text{unit backwash volume per filter, gal/sf})] / \text{UFRV}$$

Monthly average performance data for the filters from April 1995 to December 1996 is included in Appendix B. This data shows net water production (H₂O usage) and wash water generated (wash water usage). From discussions with plant staff, it is assumed that the data for wash water generated includes the volumes of filter-to-waste. Also from discussions with plant staff, a typical backwash cycle per filter uses approximately 18,000 gallons of water and consists of the following regimen:

<u>Duration, minutes</u>	<u>Sequence</u>	<u>Rate</u>
2.0	Air scour	Undetermined
5.0	Idle-air purge	--
16.0	Water backwash	1,000 gpm (10.5 gallons per minute per square feet (gpm/sf)
0.5	Backwash valve closure	--
0.0	Filter-to-waste	At filtration rate
23.5		

A summary of unit filtration rates, UFRVs, and filtration efficiencies for the existing plant is shown in Table 1-1.

Table 1-1. Florence WTP Filtration Performance-Average Monthly Values

Month	Year	Raw water flow, mgd	Finished water flow, mgd	Backwash plus filter-to-waste flow, mgd	Number of filters on-line	Average filtration rate, gpm/sf	Average filter run time, hours	Average UFRV, gal/sf/run	Filtration efficiency
June	1995	1.19	1.08	0.116	6	1.48	22.3	1,979	0.90
July	1995	1.70	1.53	0.169	6	2.13	15.2	1,935	0.90
August	1995	1.53	1.39	0.143	6	1.91	18.0	2,067	0.91
September	1995	1.35	1.24	0.115	6	1.68	22.4	2,255	0.92
October	1995	1.31	1.19	0.115	6	1.62	22.4	2,180	0.91
November	1995	1.04	0.87	0.162	6	1.30	15.7	1,224	0.85
December	1995	0.91	0.82	0.092	6	1.13	28.0	1,895	0.90
January	1996	0.93	0.85	0.087	6	1.16	29.6	2,055	0.91
February	1996	1.03	0.93	0.103	6	1.28	25.0	1,918	0.90
March	1996	0.99	0.88	0.101	6	1.22	25.4	1,865	0.90
April	1996	0.90	0.82	0.089	6	1.12	28.8	1,936	0.90
May	1996	1.16	1.06	0.098	6	1.43	26.4	2,274	0.92
June	1996	1.42	1.29	0.132	6	1.77	19.5	2,080	0.91
July	1996	1.37	1.23	0.137	6	1.71	18.7	1,922	0.90
August	1996	1.33	1.21	0.123	6	1.66	20.9	2,084	0.91
September	1996	1.13	1.04	0.095	6	1.40	27.1	2,277	0.92
October	1996	1.04	0.95	0.089	6	1.28	29.0	2,236	0.92
November	1996	0.92	0.84	0.082	6	1.13	31.4	2,134	0.91
December	1996	0.68	0.62	0.063	6	0.84	40.9	2,050	0.91

Unit filtration rates ranges from 0.84 to 2.13 gpm/sf. Values for UFRVs and efficiencies range from 1,865 to 2,274 gallon/sf-run and 0.85 to 0.92, respectively. Increase in the UFRV can be obtained by either increasing the unit filtration rate, or lengthening the filtration run time.

Increasing the unit filtration rate is the logical parameter to adjust to obtain more capacity from this plant. Although increasing the run time is beneficial from a standpoint of optimizing filtration efficiency, it may be impractical for this plant. The average run times for the existing filters range from 4 hours to 24 hours. Since this plant is staffed only on one shift per day, backwashing the filters on approximately a 24 hour cycle makes practical sense in this case. If run times were increased beyond 24 hours, backwashing cycles would be activated automatically during non-staffed evening hours. This is generally undesirable since an operator should be on duty to ensure that the backwashing sequence takes place satisfactorily, and that failure of the backwashing system does not result in major problems such as a blown filter underdrain or catastrophic loss of media.

Values for UFRVs and efficiency appear to be insensitive to minor changes in unit filtration rate and seasonal flow variations experienced to date. The data on UFRVs and efficiencies suggests that improvements to filtration productivity and efficiency are possible. Typically, a filtration efficiency of 0.95 or greater is recognized as an indication of optimized filtration.

Since the unit backwash volume per filter is 189 gal/sf, a minimum UFRV of approximately 3,800 gal/sf/run is needed to achieve a 0.95 filtration efficiency. Assuming the backwashing and filter-to-waste cycles remain unchanged, a 24-hour total filtration cycle is used, the UFRV is increased to 3,800 gal/sf-run, the resultant unit filtration rate becomes 2.69 gpm/sf, yielding a net plant capacity of 2.1 mgd. To achieve a 2.5 mgd net plant capacity with a 24 hour total filtration cycle requires a unit filtration rate of 3.24 gpm/sf, resulting in a UFRV of 5,460 gal/sf-run and a filtration efficiency of 0.96. A unit filtration rate of 3.87 gpm/sf with a UFRV of 4,579 gal/sf-run are within the design limits (typically 3 to 5 gpm/sf) of an optimized green sand filtration plant. It is anticipated, therefore, that 2.5 mgd is a realistic target for production with the existing filtration system if improvements are made.

Options to Improve Filter Efficiency

Various methods can be considered to improve filtration efficiency. Variables such as water chemistry, temperature, floc strength, media characterization, filter solids loading rate, air scour and backwash rates, and hydraulic loading rates all affect filtration productivity and efficiency. Only the variables of media characterization, air scour and backwash rates, and hydraulic loading rates are relatively easily controlled by the plant operator. Of these three variables, air scour and backwash rates can improve filter performance to a relatively slight degree, and maximizing the hydraulic loading rate is usually desirable. Improving the characteristics of the filtration media also is a significant variable that can be modified to optimize overall plant capacity.

Filter Operations and Control Systems. Filter operations and controls for these existing facilities are relatively simple. Although the number of raw water pumps are activated automatically in response to distribution system needs, all six pressure filters are kept operational at all times. Filtration rate is directly controlled by the number of raw water pumps that are activated. During periods when new pumps are activated, the unit filtration rate in each filter suddenly increases. This sudden increase, occasionally known as filter "bumping," can dislodge trapped particulate matter on the filters and adversely affect effluent turbidity. In general, it is recommended that filtration rates be kept relatively constant (or be allowed to slowly decline) and that filter bumping be minimized.

With the existing controls, filter backwashing can be initiated either manually or by timer control. Differential pressure, effluent turbidity, or soluble iron or manganese concentrations are not used to automatically initiate backwashing.

The current backwashing protocol is to backwash each filter in succession whenever the timer cycle calls for backwash. During backwashing, one raw water pump is turned off to reduce flow to the plant and minimize filter bumping for the remaining filters still on-line. As soon as one filter is backwashed, it commences filtration again and the next filter is backwashed. Backwashing all filters regularly in this manner limits filter productivity and efficiency. If a filter is on-line and still has productive life left in its filter run, premature backwashing wastes water and reduces the time that the filter is producing filtered water. To address this problem, some iron removal water plants have automated the backwash control of each individual filter. Each filter would need to be equipped with either differential pressure, effluent turbidity or soluble iron or manganese sensors.

This existing backwashing protocol offers the advantages of simplified instrumentation. In addition, all backwashing is performed during staffed periods. The operator currently monitors the performance of each filter and adjusts the length of backwash to adjust for differences in filter performance. Although all filters are nearly identical and should perform in a similar manner, there are small differences in filter media configuration and flow distribution to each filter such that they do not all have the same capacity to filter a given volume of water prior to needing backwashing. It may be desirable at a latter date to consider fully automating the filter backwashing sequence by adding more instrumentation. At present, the preferred approach is to optimize the current system and incrementally automate if system reliability can be maintained.

All inlet and outlet isolation valves are pneumatically operated. Difficulties with the pneumatically operated valves are discussed in a following section of this plan. In addition, each filter outlet is provided with a Cla-Valve to regulate filtered water flow. These Cla-Valves reportedly do not function well and their throttling characteristics have been disabled. Installation of throttling valves would help ensure that all filters operates at approximately identical flow rates. The existing variations in piping geometry and filter media configuration prevents good distribution of flows to the six existing filters, thus compromising overall plant filtration performance.

Air Scour. As discussed earlier, air scour is applied to each filter for the first 2 minutes of the backwash cycle. Air is injected into each filter via a manifold located in the media at the interface between the green sand media and the support gravel. Air discharge nozzles are generally aimed downward along the manifold. The air scour works well in cleaning the media and minimizing water consumption during backwashing. However, the current design of the air distribution manifold system allows green sand media to enter the air piping and clog the air discharge nozzles. One solution to this problem is to turn the nozzles upward to minimize the deposition of media in the air piping manifold. Plant staff have already made this modification to one filter and pipe clogging has diminished. Another solution is to install screens on each nozzle. The Roberts Filter Company has recently installed such screens on the air scour manifold nozzles at the Everett, Washington, WTP.

The existing positive displacement blower providing air for the air scour cycle is not furnished with a pressure relief valve. Since this machine is a positive displacement unit, pressure relief must be installed to prevent damage to the blower or the piping system in the event of line blockage or valve failure.

Hydraulic Backwash. Although the hydraulic backwashing cycle performs adequately in cleaning filter media, the existing filters all suffer from excessive media loss during this portion of the backwashing cycle. During hydraulic backwashing, the media is fluidized causing it to rise toward the backwash exit manifold in the top of the filter. Problems occur in the initial part of the hydraulic backwash cycle when water first enters the bottom of the media. At this time, the media still has entrained air bubbles from the air scour cycle. Some of these air bubbles will purge themselves from the media during the quiescent 5 minute purge cycle between air scour and water wash. However, not all air bubbles will be removed in this manner. When water first enters the bottom of the media containing air bubbles, it lifts great portions of the media in large uneven mats that suddenly boil upwards toward the outlet manifold. This large upward boiling action carries media to the exit manifold and out of the filter. The boiling action ceases when all entrained air bubbles are purged from the system. This problem is further exacerbated by the sudden entry of backwash water into the bottom of the filter. Most filters gradually ramp up the flow of backwash water to minimize boiling and jetting; however, the existing Cla-Valves at the bottom of the filters are unable to finely throttle the backwash water flow rate to achieve this effect. The city installed a media trap downstream of the filters to capture lost media during the backwash cycle. Due to hydraulic constraints on the system, this media trap is currently inoperable.

Several methods can be employed to greatly minimize media loss. Installing a new underdrain system that does not require support gravel lowers the position of the media relative to the exit manifold at the top of the filters. Purging trapped air bubbles more effectively after the air scour cycle by introducing water into the air distribution manifold to help flush out air bubbles from the media will also provide relief to this problem. Additionally, injecting water in the air distribution manifold helps further purge media trapped in the air distribution manifold. By far the most significant improvements to prevent media loss is to install a new backwash flow control valve (preferably electrically actuated) or flow meter with the capability to slowly ramp up and ramp down the flow of backwash water to the filters. Controlling the ramp down cycle helps properly stratify the media (coal and green sand) for the subsequent filtration cycle.

Optimizing Media Configuration. The existing green sand media as described above has a relatively fine-grained structure and can be described as being relatively "tight." This media has a small effective size that can clog easily, thus limiting filter run time, filtration rates, and increasing headloss throughout the media. Increasing the media size would help; however, only the current 35 mm size is commercially available.

A recommendation to help optimize filtration would be to increase the media effective size. However, larger sized media may not be feasible without changing the media type because the greensand media is only commercially available in the current 35 mm size. The use of anthracite as a topping layer should be reconsidered also. While anthracite can aid in manganese removal and reduce filter surface blinding, manganese levels in the raw water are low and replacing the

anthracite with greensand would increase iron removal. It is recommended that an additional foot of greensand be used in some of the filters in lieu of anthracite to determine the optimum filter configuration.

Filter No. 1 was removed from service for repairs recently and it was found that much of the media had escaped due to a breach in the retaining screen holding the media. This is the oldest filter and it is likely that it is in the worst condition. The plant staff is currently planning to retrofit this filter with all new internal components to improve performance.

Finally, additional filter media depth as well as lowering of the filter media can be achieved by replacing the current underdrain and support gravel system with a new underdrain system that does not require support gravel. There are numerous new products commercially available which replace older style underdrains.

CHEMICAL FEED SYSTEMS

This section describes the condition and characteristics of the WTP chemical feed systems. The existing chemical feed systems include chlorine, air, potassium permanganate, caustic soda, soda ash, sodium fluoride, and zinc hexametaphosphate. In general, chemicals are fed into their point of application in solution. Increasing the dispersion of chemicals at the point of application improves chemical effectiveness and often leads to a reduction of chemical consumption. Typically, carrier water flows are approximately 10 times greater than the metered chemical flows.

Chlorine

The existing chlorine feed systems consist of chlorine gas ton cylinders connected to a gas manifold system feeding one chlorinator with a capacity of 200 ppd. Chlorine solution is injected into the horizontal contact chamber and dosage is controlled only by flow pacing. Residual chlorine measurements are not used for automatic process control at this facility. This is generally adequate since chlorine demand is not likely to vary greatly with a groundwater supply. The average dose of chlorine is approximately 9 mg/L. The plant is not provided with a chlorine residual analyzer. This analyzer should be installed with a high and a low residual alarm and a low chlorine residual plant shutdown. This should be provided to help control chlorine underdosing and overdosing. A second chlorinator is also recommended to provide system redundancy.

Air

Compressed air is injected downstream of the horizontal contact chamber to assist with oxidation of iron and manganese. The existing air compressor system consists of one positive displacement compressor with a receiver. This system reportedly functions well and does not need modification for a plant expansion up to 3.0 mgd.

Potassium Permanganate

The existing potassium permanganate feed system consists of two fiberglass bulk storage tanks designed to contain 1.0 percent potassium permanganate solution. A small drum of granular

potassium permanganate is manually added to a mixing tank filled with water to prepare each batch of solution. Solution is applied via metering pumps. This system is apparently functioning well.

Potassium permanganate dosage is flow paced. The current average raw water dose of potassium permanganate is 0.24 mg/L.

Caustic Soda

The existing caustic soda feed system consists of two fiberglass bulk storage tanks and metering pumps designed to contain and deliver the caustic soda solution. This system is relatively new and is apparently functioning well.

Caustic soda dosage is strictly flow paced and is not controlled by pH. Since raw water alkalinity is expected to be relatively stable, automatic pH control is not essential. Plant staff has recommended that caustic soda also be used for final pH adjustment in lieu of soda ash to simplify the addition of pH adjusting chemicals. If caustic soda is used for both pre- and post-filtration pH control, the dosage could be adjusted to serve both functions.

Soda Ash

Soda ash is delivered in bags stored on pallets. The existing soda ash system consists of a pre-packaged 1,000 pound per day capacity Wallace and Tiernan feed system consisting of a bucket elevator, a dry chemical feed hopper, a volumetric screw feeder, a solution tank with mixer, and metering pumps.

Soda ash dosage is flow paced controlled. Since raw water alkalinity is expected to be relatively stable, automatic pH control is not essential. The current average dose of soda ash is 19 mg/L.

Plant staff has suggested that the soda ash feed system be converted to handle sodium fluoride. Caustic soda would be used in lieu of soda ash for final pH adjustment. Sodium fluoride dust is particularly hazardous to human health. Use of this feeder for sodium fluoride will require an enhanced dust collection system be installed on the existing soda ash feed system at the point that sodium fluoride enters the bucket elevator. The soda ash feed system is currently equipped with a hood and an exhaust fan.

Sodium Fluoride

Sodium fluoride solution is prepared in a single fiberglass batch tank. The solution is prepared as a saturated solution of NaF with excess NaF solids in the bottom of the saturator tank. This solution is delivered to the point of application with a metering pump.

Plant staff has suggested that sodium fluoride be fed with the existing sodium carbonate feed system. It is recommended that sodium fluoride dosage be controlled by flow pacing. Currently dosage is controlled manually. The current average fluoride dose is 0.8 mg/L as F.

Sodium Hexametaphosphate

The existing sodium hexametaphosphate feed system consists of one plastic bulk storage tanks designed to contain the sodium hexametaphosphate solution and one metering pump for chemical application. This chemical is supplied by Calgon. This system appears to function adequately.

Sodium hexametaphosphate dosage is controlled manually. Control by flow pacing is recommended.

WASH WATER PONDS

Presently there are two wash water ponds located east of the WTP. Each pond is approximately 6 feet deep with a volume of about 180,000 gallons. Each pond can accommodate the backwash waste from all six filters. Each pond can be filled and dredged separately. Decanted supernatant is discharged to a nearby creek. Currently, dredged solids have yet to be disposed of; however, disposal at the city's wastewater treatment plant should be considered.

A new backwash line to the ponds is recommended to maintain a constant head and flow during the backwash operation. The new backwash line will enter the ponds at the normal high water level so that pond level does not cause variable backpressure on the backwash operation.

In an effort to better utilize decanted supernatant, the recovered water could either be re-injected at the head end of the water treatment plant or used for irrigation of surrounding landscaping. As described earlier, approximately 9 percent of the water passed through the filters is used for backwashing and is ultimately wasted. By recovering this water, overall plant efficiency will improve since much less filter effluent is wasted. However, periodic backwashing agitates the sediment in the backwash ponds which would cause a variable iron load back to the plant. The operators first choice would be use the recovered water to irrigate beach grasses located in the vicinity of the ponds. This practice would help prevent erosion of surrounding property and improve the esthetics of the plant.

While it is recommended that decanted supernatant from this plant be reused as described above, some discharge to the nearby creek should be maintained to prevent the city from losing its discharge permit. It is prudent to retain use of the existing creek discharge permit in the event that a plant upset or plant construction necessitates discharge to the environment. If it became necessary to discharge backwash flows to the wastewater treatment plant, the impact of this 100,000 gallons per day flow stream on that facility must be considered.

OPERATIONS BUILDING AND OTHER SUPPORT FACILITIES

The existing operations building and support facilities, including mechanical, ventilation, and piping systems are adequate for their function for the current plant capacity. Expansion of these facilities to accommodate increased plant capacity can be accomplished in a straight-forward manner. No recommendations to modify these facilities are anticipated.

Upgrades to the plant electrical and instrumentation systems would be beneficial. These two support systems have caused recurring operational problems. For example, many of the major valves controlling the filters are pneumatically actuated. This pneumatic system is powered by a single air compressor with a second compressor as a manual backup. In addition, the pneumatic system is not equipped with an air drier, causing concern that pneumatic lines need more frequent cleaning and purging. It is recommended that if the city wishes to retain its pneumatic control system, two oil-less compressors be used with automatic switch-over in the event of failure of the primary unit. Refrigerant or desiccant air dryers are also recommended. Improved performance

and reliability can be achieved if the most critical of the pneumatically operated valve actuators were replaced with modern electric actuators. Electric actuators are easier to maintain and permit accurate throttling and control of key filtration valves.

To improve system reliability, installation of a telemetry system to monitor plant operations (especially during non-staffed periods) is recommended. These signals would be tied into an automatic phone dialer to report emergencies.

An additional problem that has been reported is the lack of sump pumps in the plant's metering vaults. These vaults occasionally fill up with ground water, which compromises the integrity of the flow meters. Although installation of sump pumps in these vaults would improve integrity of the plant metering system, new, more accurate flow meters in a new location are recommended. This would improve plant performance with more accurate and positive means to flow pace all applied water treatment chemicals.

Standby power for the WTP is probably not needed provided that storage in the distribution system is adequate for relatively short duration power outages.

HECETA WATER SYSTEM

A significant portion of Florence's potable water needs are met by the District. In the mid-1960s, the District began to withdraw water from Clear Lake to serve the initial development in the area north of Florence. In 1970, an intertie was established between the two distribution systems. At present, almost one third of Florence's summer usage is met by the District. It is anticipated that ultimately the two water purveyors will join and the City of Florence will become the sole supplier of water to the area.

TREATMENT

Clear Lake is the sole water supply source for the District. Due to the high water quality of Clear Lake, no treatment other than disinfection has been necessary to this point. A screened intake is provided to convey flow through a 12-inch ductile iron pipe into a cylindrical concrete wetwell. Chlorine solution is produced from vacuum chlorinators fed by 150-pound chlorine cylinders. A residual of 0.8 mg/L chlorine is produced. The treated water is pumped by two 30-hp vertical, constant speed, turbine pumps to a nearby reservoir with a water surface 120 feet above the lake. A government surplus emergency generator is provided for standby power.

Although this facility is well maintained, major modifications will be necessary to comply with current regulations. The existing intake limits the amount of flow which can be pumped. The maximum daily flow produced by this facility is approximately 2 million gallons. Heceta holds four water rights for a total of approximately 4.1 mgd. These water rights are dated 1968, 1974, 1987, and 1994.

DISTRIBUTION

The Heceta distribution system extends from the Florence city limits north approximately 6 miles. There are currently about 1,700 connections to the District, which represents a total service population of about 4,240.

The District currently serves a significant area inside the northern Florence Urban Growth Boundary (UGB), approximately 26 percent of the present District area according to the 1996 Heceta Water District Facilities Plan. This portion of the District within the UGB, located north of Thirty-fifth Street, currently includes 1,028 connections serving approximately 2,570 people. It is anticipated that this area will experience major development within the next 20 years. Table 1-2 shows the use of all water produced by the District for the past 3 years.

The District's Facility Plan identifies approximately 45 miles of distribution piping from 2 inches to 12 inches in diameter. A large portion of the system is asbestos cement pipe installed in the 1960s and 70s. The remainder is more recently installed PVC. The southernmost extension of Heceta's 12-inch distribution main was connected to the Florence water system at a master meter. The intertie is manually control by a valve.

STORAGE

Four reservoirs provide water storage for the District. The reservoirs locations, capacities, and ages are provided below according to the District Facilities Plan.

<u>Reservoir Location</u>	<u>Capacity</u>	<u>Type</u>	<u>Year Constructed</u>
Clear Lake	600,000	steel	1967
Collard Lake	500,000	steel	1969
Sutton Lake	700,000	concrete	1976
Enchanted Valley	30,000	wood	1973

WATER QUALITY ISSUES

Florence currently produces water only from a groundwater source. During much of the year, Florence also purchases surface water from the Heceta Water District. The Florence raw water source is relatively high in iron and it is treated to quite low levels. Little manganese is present in either the Florence or Heceta source. The pH of both supplies is essentially neutral. Although no problems have been noted with the combined Florence and Heceta supplies, some municipalities have noted reactions when groundwater and surface waters are mixed.

In the future, the ratio of groundwater to surface water supplied may change depending upon the selected alternative. This could conceivably result in changes to the composition, taste, or appearance of the water supplied to the customer. A numerical simulation could be run to determine the actual influence of mixing water sources in the future. This simulation would consider the blended water pH, mineral composition, temperature, and the equilibrium conditions for all constituents.

Table 1-2. Heceta Water District Production and Sales in Million Gallons

		Heceta sales to Florence	Heceta usage	Total Heceta production
	Jan-94	2.15	9.29	11.44
	Feb-94	1.96	9.55	11.51
	Mar-94	7.23	10.34	17.57
	Apr-94	4.69	10.33	15.03
	May-94	14.29	13.78	28.07
	Jun-94	19.31	16.68	35.99
	Jul-94	25.21	25.54	50.74
	Aug-94	28.44	24.76	53.20
	Sep-94	14.30	15.52	29.82
	Oct-94	9.43	13.03	22.45
	Nov-94	2.43	10.17	12.60
	Dec-94	2.39	9.45	11.84
Total For 1994		131.81	168.42	300.24
	Jan-95	3.76	9.63	13.38
	Feb-95	82.55	9.07	91.62
	Mar-95	75.16	11.36	86.52
	Apr-95	2.63	10.05	12.68
	May-95	4.46	13.21	17.66
	Jun-95	5.28	14.77	20.05
	Jul-95	10.15	21.72	31.86
	Aug-95	13.03	20.44	33.46
	Sep-95	3.88	14.09	17.98
	Oct-95	0.01	10.90	10.91
	Nov-95	0.01	9.35	9.36
	Dec-95	0.00	9.61	9.61
Total For 1995		200.91	154.18	355.09
	Jan-96	0.00	8.64	8.64
	Feb-96	0.00	8.65	8.65
	Mar-96	0.00	9.91	9.91
	Apr-96	0.00	9.54	9.54
	May-96	0.00	12.04	12.04
	Jun-96	5.23	19.16	24.39
	Jul-96	10.97	20.94	31.91
	Aug-96	13.07	21.12	34.19
	Sep-96	5.95	15.85	21.80
	Oct-96	2.27	11.43	13.70
	Nov-96	1.14	10.23	11.36
	Dec-96	6.58	10.79	17.36
Total For 1996		45.20	158.29	203.49

Once the preliminary design is prepared for the water treatment plant and the estimated chemical dosages have been determined, much more will be known regarding the final composition of the water. For instance, it is likely that some pH adjustment of the Clear Lake water may be necessary in the future to address corrosion concerns. Alum addition would be a reasonable method to raise pH. When these decisions have been made, a numerical simulation can then be run to determine the characteristics of the new combined water source.

CHAPTER 2

FUTURE WATER SYSTEM NEEDS

Development of a long-range water management plan for the Florence area requires consideration of natural, socioeconomic, and environmental characteristics, as well as water supply considerations. The natural environment, including topography, geology, soils, climate, and water resources, affects existing surface and groundwater quality, as well as potential water sources. Factors such as land use and population directly relate to the water use patterns of an area. This chapter describes the study area and discusses the characteristics of both its physical and socioeconomic environment. Future water requirements are established based on current usage patterns. Last, the basis of planning for system improvements is discussed including design standards and typical construction costs.

WATER STUDY AREA

LOCATION AND BOUNDARIES

The City of Florence is situated along the north bank of the Siuslaw River on the central Oregon coast. The city is located in the southern third of the western edge of Lane County. The study area for the facilities plan encompasses the proposed Florence Urban Growth Boundary (UGB). The UGB is currently under review for updating. It is expected to remain essentially unchanged except for minor modifications discussed later in this chapter. Figure 2-1 shows the water facility plan study area (proposed UGB). The current UGB encompasses about 5,400 acres. As can be seen from the figure, geographic barriers (Siuslaw River and Pacific Ocean) preclude development to the south and west. Development is also somewhat limited to the east by lakes, the drinking water supply basin, and steep contours of the Coast Range foothills. Most development is expected to occur to the north.

PLANNING PERIOD

The facilities planning period must be long enough to allow the city to develop and pursue a long-range program. Key limits to the planning period are the precision of population projections and the ability to estimate industrial and commercial growth. A 20-year planning period is typically used in facilities plans. The start of the period is assumed to be the year that the facilities are first put into operation. For this study, the planning period is 20 years—years 2000 through 2020.

PHYSICAL ENVIRONMENT

The physical environment includes the topography, geology, soils, climate, and water resources of the region. This section presents a brief discussion of these items as they relate to the water planning program.

Topography, Geology, and Soils

The topography, geology, and soils of a region will have a significant effect on the design and construction requirements of water facilities. Topography will determine the limits of pressure zones, location of reservoirs, and can determine the route of water lines. The geology and soil conditions of an area will influence both the availability of groundwater as well as construction costs for pipelines and treatment units.

Topography. The city is in a coastal terrace area with gently rolling terrain dominated by dune formations. Slopes range from 0 to 10 percent.¹ The slopes are flat enough for various types of development throughout most of the study area. Land elevations vary from about 10 feet above sea level near the river and coastline to about 100 feet around dune formations. The land generally slopes upward as one proceeds north from the downtown area near the Siuslaw River. The elevation approaches 200 feet further east, along the foothills of the Coast Range mountains.

Geology and Soils. The Florence area is underlain by the Tyee Formation, a thick bed of sandstone and siltstone deposited during the middle Eocene epoch. Although this formation is the dominant outcropping throughout vast areas in the Coast Range, it is overlain by more recent deposits in the Florence area.²

The most recent deposits in the Florence area are sand dune formations from the Holocene epoch. These include active dunes, stabilized dunes, and deflation plains. Stabilized dunes are the most widespread geologic feature in developable areas. The resulting soil type on stabilized dunes is Waldport fine sand. It is a deep, excessively drained soil. Although most of the Florence area development has occurred on stabilized sand dunes, construction on this type of deposit requires caution. The stabilized dune deposits consist of unconsolidated sand with possible layers of compressible organic materials and peat. Because the deposits are compressible, precautions must be taken in foundation design for heavy structures. Additional geotechnical studies are required to determine the extent of compressibility and resulting foundation requirements. Unconsolidated sand is particularly unstable during earthquakes; liquefaction and major additional settling can occur. The permeability of the soil results in rapid movement of groundwater. During construction, wind erosion of unprotected exposed soils can easily occur.

Active dunes are structurally similar to stabilized dunes, except that they are still subject to movement due to blowing sand. In addition to all the limitations and potential problems associated with stabilized dunes, active dunes have blowing sand. Structures in these areas require frequent maintenance including removal of sand and repair of sandblasted surfaces. These areas are generally unsuitable for most development unless stabilized.

Deflation plains are defined as the interdune areas that have been eroded by wind down to the summer groundwater table. These areas have high groundwater in the summer and are usually submerged in the winter. The soil type is Yaquina loamy fine sand which tends to be corrosive to steel and concrete. Although the soil is rather permeable, it is poorly drained because of the high water table. Deflation plains that have been developed generally have been filled and drained. Sewers in deflation plains are subject to high groundwater and corrosive soils.

Estuarine deposits underlie the dune deposits in much of the Florence area. They are old Siuslaw River deposits from the Pleistocene epoch. They are usually found 35 feet or less below the dune