Preliminary Engineering Report

Water Supply Expansion

Prepared for:
City of Port Orford

Consulting Engineers & Geologists, Inc.

275 Market Avenue
Coos Bay, OR 97420-2219
541/266-9890

April 2010
608040.070

Reference: 608040.070

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Prepared for:

City of Port Orford
PO Box 310
Port Orford, OR 97465

Prepared by:

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275 Market Avenue
Coos Bay, OR 97420-2228
541-266-9890

April 2010

REGISTERED PROFESSIONAL ENGINEER #16,604 OREGON JULY 20, 1990 STEVEN K. DONOVAN

EXPIRES: 12/31/11

QA/QC: _____

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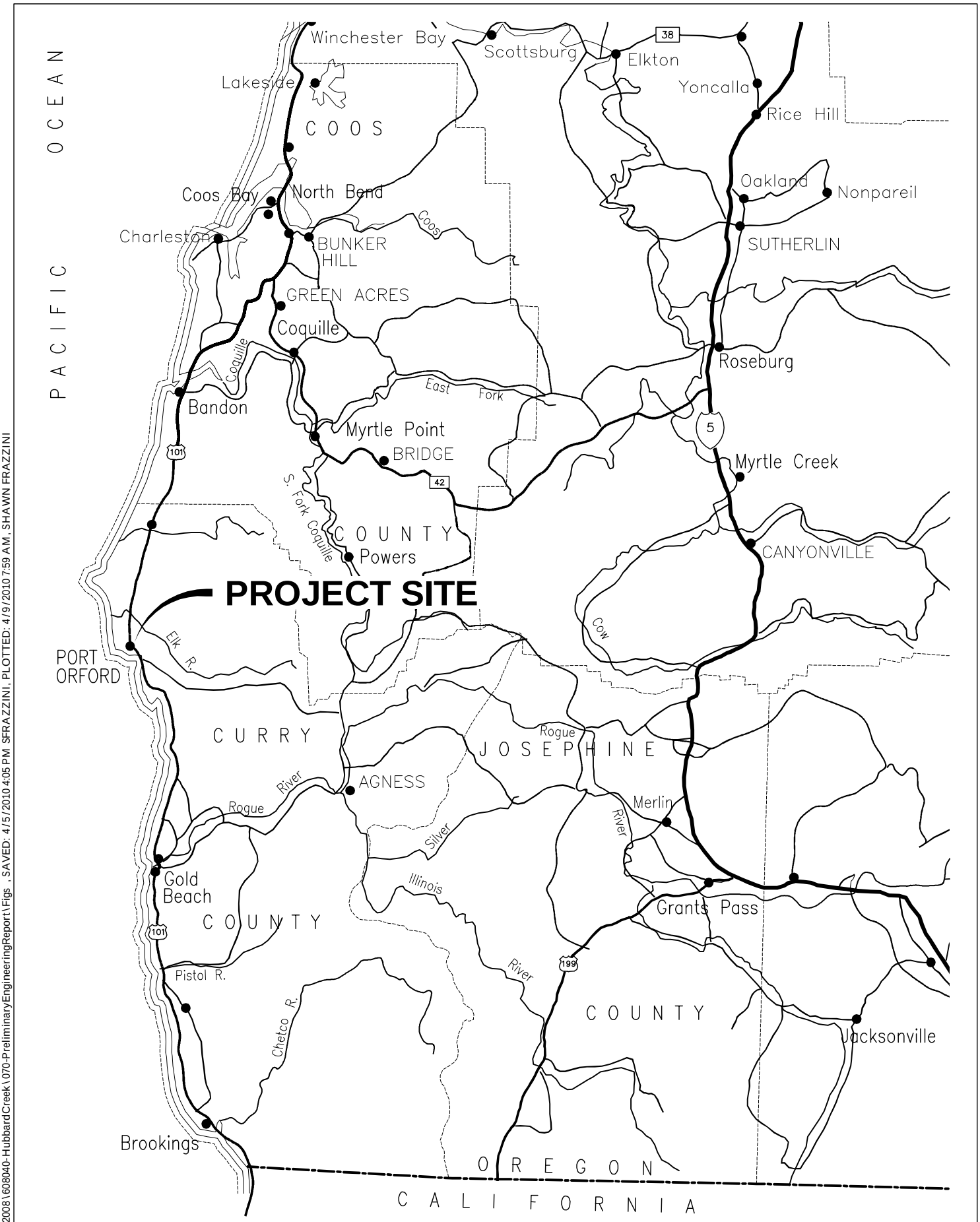
Acronyms and Abbreviations

ADD	Average Daily Demand
cfs	cubic feet per second
CPE	Comprehensive Performance Evaluation
CIP	Capital Improvement Plan
DEQ	Department of Environmental Quality
DIP	Ductile Iron Pipe
EA	Environmental Assessment
EDU	Equivalent Dwelling Unit
ESWTR	Enhanced Surface Water Treatment Rule
gpcd	gallons per capita day
gpm	gallons per minute
MMD	Maximum Monthly Demand
MDD	Maximum Daily Demand
MG	Million Gallons
mgd	million gallons per day
NEPA	National Environmental Policy Act
NTU	Nephelometric Turbidity Units - a measurement of turbidity
OIFA	Oregon Infrastructure Finance Authority
OHD	Oregon Health Division
OWRD	Oregon Water Resources Department
PER	Preliminary Engineering Report
PHD	Peak Hour Demand
ppm	parts per million
psi	pounds per square inch
PVC	Polyvinyl chlorine
RCP	Reinforced Concrete Pipe
SDWA	Safe Drinking Water Act
SWTR	Surface Water Treatment Rule
UGB	Urban Growth Boundary
UFC	Uniform Fire Code
UGB	Urban Growth Boundary
WSDA RD	United States Department of Agriculture Rural Development
WS	Water Surface
WTP	Water Treatment Plant

1.0 Introduction

The City of Port Orford is an incorporated City within the State of Oregon situated on the Southern Oregon Coast in Curry County. The City of Port Orford is located between the City of Bandon to the north and the City of Gold Beach to the south, as shown in Figure 1. The community was estimated to have a population of 1,285 in 2009 as determined by the Center for Population Research and Census at Portland State University. Port Orford owns and operates a municipal water system comprised of a raw water supply reservoir, water treatment plant, finished water storage tank, and distribution system. In February 2004 a Water Master Plan was completed and in December 2005 the Master Plan was updated and adopted by the Council. The Water Master Plan provided an engineering assessment and planning guidance for the successful management of the City's water system over a 20 year period. The plan outlines infrastructure improvements required to maintain compliance with State and Federal standards as well as provide for anticipated growth. One primary conclusion of the Master Plan was that the City is running out of raw water supply due to the limited size of the North Fork Hubbard Creek reservoir. The plan recommended several alternatives to consider increasing the City's water supply, including expansion of the North Fork Hubbard Creek dam, construction of a new dam upstream of the current impoundment, or improve water treatment capabilities to treat brackish water from Garrison Lake. This Preliminary Engineering Report (PER) more clearly defines these alternatives and recommends the improvements needed to provide the City with a water supply which should carry it through the next 50 years.



SHN Consulting Engineers & Geologists, Inc.

Port Orford
Hubbard Creek

Vicinity Map

SHN 608040.070

April 2010

608040-Vicinity-Map-Fig_1

Figure 1

\\Coosbay\projects\2008\608040-HubbardCreek\070-PreliminaryEngineeringReport\Figs , SAVED: 4/5/2010 4:05 PM SFRAZZINI, PLOTTED: 4/9/2010 7:59 AM, SHAWN FRAZZINI

2.0 Study Area

The study area related to this PER is defined by all of the lands contained within the City limits and Urban Growth Boundary (UGB) of the City. Additional limits to the study area consist of those portions of the water supply associated with the North Hubbard Creek water shed, impoundment area and transmission lines, see Figure 2.0.

2.1 Physical Environment

Port Orford's coastal climate is characterized by wet winters and relatively dry summers. Temperatures are moderated by onshore marine breezes, which keep conditions relatively cool even in the summer months. Onshore breezes from the Northwest persist during the summer, while periodic but stronger Southwesterly winds occurring during the winter.

One of the most vital features in Port Orford is Highway 101, which traverses the City north to south. Commercial development is generally centralized along Highway 101. Residential development is located on both sides of the highway; extending to the west to Garrison Lake and the Pacific Ocean and extending to the east into the foothills. The topography of the City ranges from sea level to an elevation above 225 feet near the top of Coast Guard Hill.

2.2 Economic Conditions

The City of Port Orford has an economy based on a small commercial fishing fleet and tourism. RV parks support a seasonal population that, combined with the influx of tourists during the summer, increases the peak water demand. Numerous community events are planned each summer, which can cause the City's population to swell. During the winter the community settles to a more gradual pace and water consumption reflect the demands of the permanent residents.

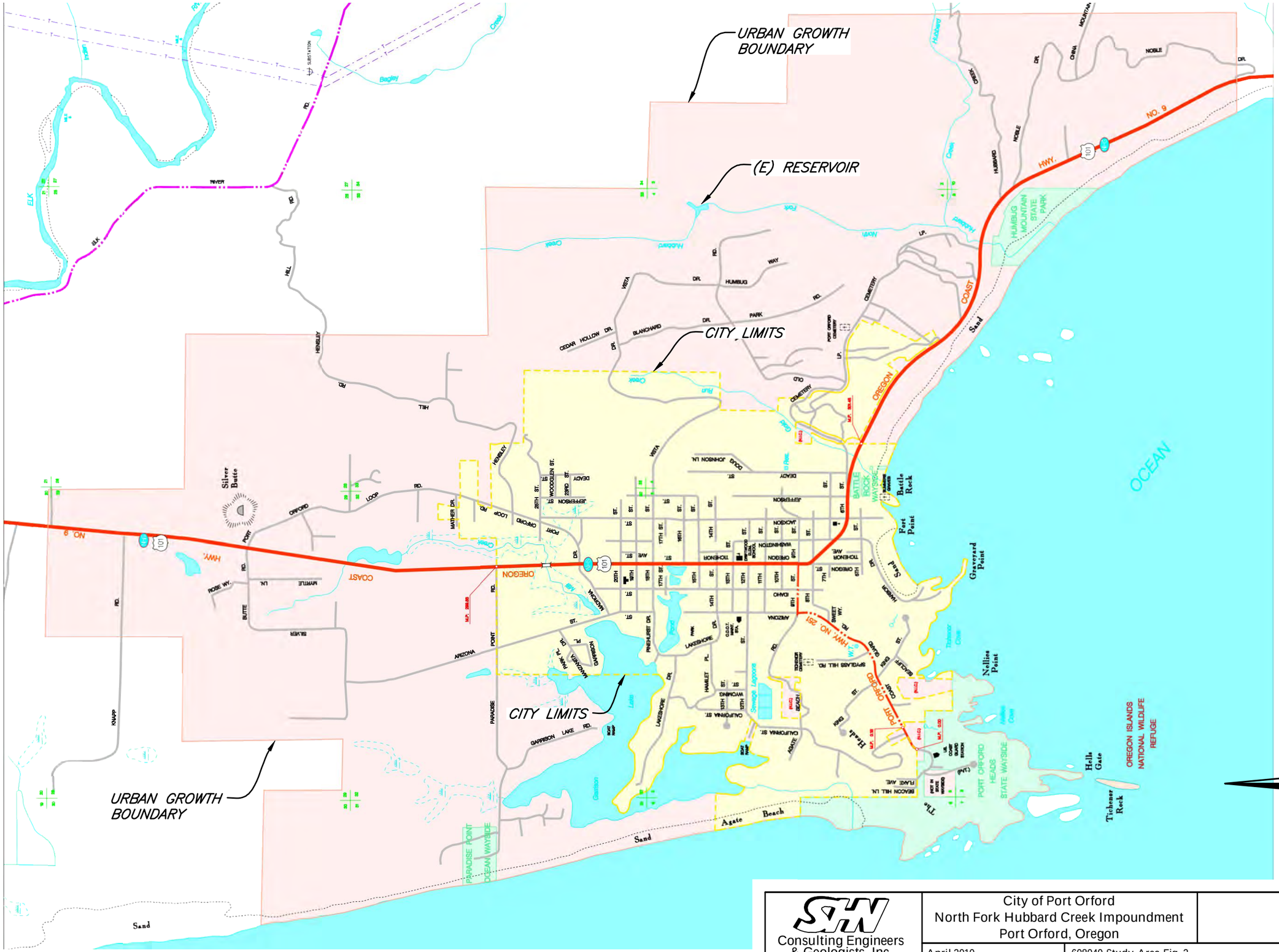
2.3 Population Trends

The Center for Population Research and Census at Portland State University (PSU) prepares annual population estimates for all incorporated cities and each county within the State of Oregon. The Port Orford certified population estimate for July 1, 2009 was 1,285 residents. This population estimate is the most current for the City and will be used as a basis for projecting future population growth.

Historically (1950-2000) City growth rates have averaged approximately 1% per year[1]. Future projections for the 50-year planning period will be based on this historical trend. The Curry County Comprehensive Plan projects an annual growth rate of 1.2% for the City. Planning for the 20-year planning period will be based on the comprehensive plan estimates.

Table 2.1 presents the population projections associated with this Preliminary Engineering Report which includes the 10-year, 20-year, and 50-year planning horizons.

[1] Portland State University data.



SHN Consulting Engineers & Geologists, Inc.

City of Port Orford
North Fork Hubbard Creek Impoundment
Port Orford, Oregon

Study Area

SHN 608040

April 2010

608040-Study-Area-Fig_2

Figure 2

\\Coosbay\projects\2008\608040-HubbardCreek\070-PreliminaryEngineeringReport\Figs , SAVED: 4/5/2010 9:59 AM SFRAZZINI, PLOTTED: 4/5/2010 10:15 AM, SHAWN FRAZZINI

Table 2.1 Population Projections For the City of Port Orford		
Source	**Year**	**Population**
Census	2000	1153
PSU Estimate	2009	1285
Projection[1]	2010	1298
Projection[1]	2020	1434
Projection[1]	2030	1584
Long Range Projection[2]	2060	2134 [2]

1. Based on 1.2% growth rate; Curry County Comprehensive Plan
2. Based on 1.0% growth rate.

3.0 Existing Water System

The City owns and operates a water utility consisting of supply, treatment, storage and distribution. Figure 3.0 presents a schematic layout and general service area map of the City's water system. A detailed description of each major element of the system is provided below.

3.1 Raw Water Sources

The City of Port Orford has three water sources with existing water rights including the North Fork Hubbard Creek, Garrison Lake, and Gold Run Creek. A discussion of these sources is provided below.

3.1.1 North Fork Hubbard Creek

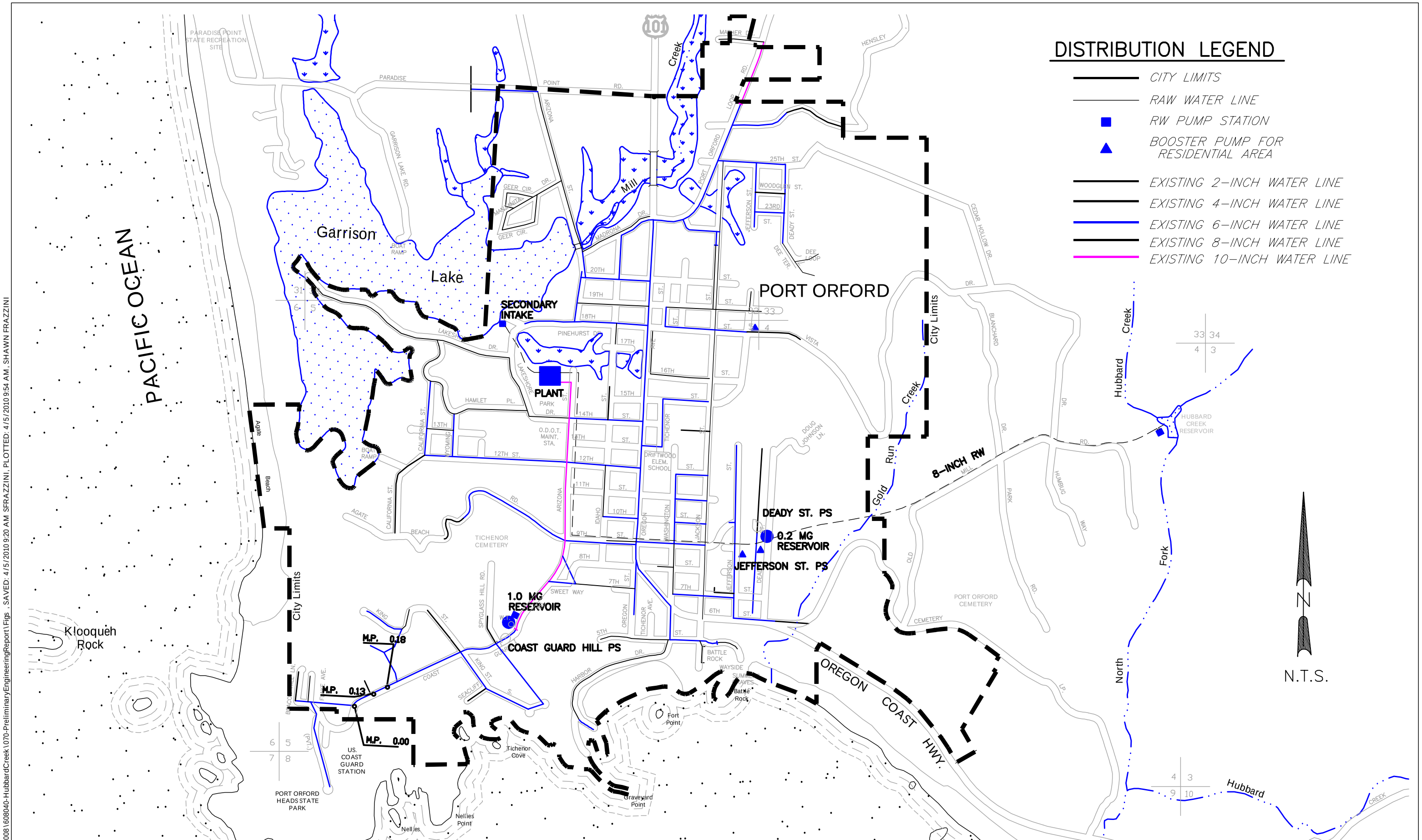
Port Orford's primary water source is the North Fork of Hubbard Creek where the City has a 3.2 acre-foot impoundment. The watershed for this impoundment encompasses approximately 612 acres east and north of the City's UGB. Raw water is drawn from the impoundment from a pump house located adjacent to the reservoir approximately 1 mile east of town as shown in Figure 2. Withdrawal records indicate that the City's has relied upon this water supply as its primary source during the period from 1993-2009.

Timber harvest and related road construction impacted the North Fork Hubbard Creek watershed during the early 1990's. Siltation from runoff from harvested areas reduced the storage volume of the impoundment and had a negative impact on water quality during the years immediately following the timber harvest. Siltation of the reservoir is currently being addressed by a multi-year dredging project being conducted by the City Public Works crews. Dredging activities have provided a temporary improvement to the City's storage needs while a long range plan to improve the City's ownership of the watershed and improve timber harvest practices have been effective at restoring the watershed. Siltation of the impoundment is becoming less problematic as measures to improve the watershed have matured.

3.1.2 Garrison Lake

The City of Port Orford also has a raw water intake on Garrison Lake that had not been used since the winter of 1993 when it was improved to supplement low quality water from the Hubbard Creek source. The Garrison Lake intake was originally (prior to 1980) located in a shallow marshy area near the discharge of an old millpond. In the late 1990's the City constructed a new intake at the end of Pinehurst Dock in an attempt to improve water quality by drawing from a deeper point in the lake. This approach was not successful due to poor water quality (high salinity levels) causing taste and odor problems. Since that time, the City has only exercised the equipment at the intake and considers the Garrison Lake intake to be an emergency back-up.

Garrison Lake continues to be impacted by salinity levels from wave run-up and overtopping of the Agate Beach berm. During wet winters Garrison Lake flooding has been a recurring problem resulting in frequent breaching of the lake outlet. Lake breaking in turn, depleted a section of the beach, potentially allowing more overtopping and increased lake salinity levels. Breaching of the



SHN Consulting Engineers & Geologists, Inc.

City of Port Orford
North Fork Hubbard Creek Impoundment
Port Orford, Oregon

Existing Water System

SHN 608040.070

April 2010 | 608040-Ex-W-Distrib-Sys-ALL-Fig_3 | Figure 3

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sand barrier may no longer be a problem since Oregon State Parks constructed the Garrison Lake outlet in 2008. This outlet has been effective at regulating the lake level since its construction.

The South Coast Watershed Association has studied the water quality of Garrison Lake during the period of lake breaching and frequent overtopping events. In the spring of 2005 a report from the Watershed Association concluded that the lake can be best characterized as a lagoon due to high salinity levels. In 2009, it was reported that salinity levels had declined significantly. Water in the deep pools of the lake may be more brackish than salty. In the future, the water quality may continue to improve which could allow the City to consider Garrison Lake a more viable back-up water supply.

3.1.3 Gold Run Creek

Prior to construction of the impoundment on Hubbard Creek, residents of Port Orford obtained their water from a small impoundment on Gold Run Creek. Currently the old impoundment is completely silted in and no longer used as a raw water source. Use of Gold Run as a city water source no longer seems practical; however, the impoundment area may have use for the City as a mitigation area, potentially through a lease of its water right to the State or reservation of the area for wetlands and natural forest habitat.

3.2 Water Rights

All water in Oregon is publicly owned. Because of this public ownership, a water right is generally required for anyone to use water from a source, whether surface or underground.

Oregon's water laws are based on the principal of prior application. That is, if a person obtains a water right on a particular source before someone else, the person would have a "senior" water right that would permit them first use of the water during times of lower flows or droughts. A "junior" water right is one that is, by comparison, obtained after other water rights for a particular source have been assigned. A water right may be both senior to some and junior to other water rights.

During periods of low water availability, a water right holder may use as much water as their water right allows as long as the use is truly beneficial and all senior water rights are satisfied. This method of resource appropriation governs all water used until the water is exhausted.

The water rights currently held by the City are "perfected" that is the City holds a certificate for each of the water rights. Typically, a permit allows an entity to remove water for a limited amount of time. When the time limit has expired, an extension must be filed for the water right. If the permit is converted to a certificate, the water right is said to be "perfected" and extensions are no longer required.

The City has water rights for 1.25 cfs (0.81 MGD) from its primary water source on North Fork Hubbard Creek and a water right for an impoundment of up to 3.2-Acre ft (1.04 MG). Additional water rights include those on Garrison Lake (1.0 cfs) and Gold Run Creek (1.0) cfs.

Table 3.1 Water Rights Summary*				
Location	**Permit #**	**Cert. #**	**Magnitude**	**Priority**
North Fork Hubbard Cr.(R)		69194	3.2 Acre Ft.	01/01/1993
North Fork Hubbard Cr	S32982	42379	0.50 cfs	03/20/1968
North Fork Hubbard Cr	S47688	65322	0.75 cfs	07/15/1983
Garrison Lake	S42566	65199	1.00 cfs	04/25/1977
Gold Run Creek	S12266	11810	1.00 cfs	08/20/1936
1. Source: Oregon Water Resources Department Water Right Inventory System (WRIS)				

3.3 Raw Water Transmission

A schematic of the City of Port Orford's water systems, showing the raw water supplies, pump stations and transmission lines is provided in Figure 3. The City has a raw water pump station, intake, and raw water transmission piping for appropriating water from each of its current sources: Hubbard Creek and Garrison Lake.

3.3.1 North Fork Hubbard Creek

The intake at the impoundment on the North Fork Hubbard Creek consists of a pump station which houses two horizontally mounted centrifugal pumps. The building is a wood framed shed with a concrete slab that has an exterior suction well cast into it. The suction well extends into the impoundment. The pump suction pipes exit the building and draw water approximately 4-feet below the normal water surface. The existing pumps are 60 HP and were installed in 1983. One of the pumps has a trimmed impeller and a capacity of 350 gpm; the other pump, recently rebuilt, has a capacity of 500 gpm.

The raw water transmission line that delivers North Fork Hubbard Creek water was constructed in two segments. Prior to construction of the water treatment plant, water from the impoundment was simply chlorinated and pumped to the reservoir on Deady Street. In 1983 the raw water piping was extended from the reservoir to the treatment plant with 4,880 feet of 8-inch PVC pipe. The raw water line from the impoundment to Deady Street was also replaced with 8-inch PVC pipe.

3.3.2 Garrison Lake Intake

The existing intake pipe on Garrison Lake is located near the Pinehurst Dock. The original intake consisted of a suction strainer placed within a perforated manhole submerged along the shoreline near the Pinehurst dock. In the fall of 1993 the City was forced to use the Garrison Lake source because of siltation problems in the Hubbard Creek impoundment. At that time, the intake was extended to the end of the dock. The City utilized the lake water through the winter but discontinued withdrawal from this source in late spring when they began to receive taste and odor complaints due to increased algae growth. As previously discussed, modifications to the intake were implemented to deepen the point of withdrawal, however these modifications failed to improve water quality. The City has not used Garrison Lake as a primary water source since 1993.

Pumping facilities on the shore of Garrison Lake consist of two vertically mounted close, coupled centrifugal pumps, with vacuum assisted priming and electrical controls. Pumps are 15 HP and are

rated at 560 gpm each @71 feet of TDH. The City periodically exercises this facility as part of the public works routine maintenance program. New facilities may be required if increased reliance on Garrison Lake is ever considered practical.

3.4 Water Treatment

The City's water treatment facilities were first constructed in 1978 to provide for chemical coagulation, flocculation, sedimentation, filtration, and disinfection of water from the Hubbard Creek impoundment or Garrison Lake. The plant was designed with a capacity of 555 gpm (0.8 MGD) though the plant has been rated by the Oregon Health Division at 450 gpm (equivalent to a filtration rate of 3 gpm/ft^2). Sedimentation and filtration process occur in a pre-designed aluminum equipment basin "package" supplied by Keystone Engineering & Products Co., Inc.

Upgrades to the treatment plant were performed in 1996 and included: filter to waste piping, effluent turbidity meters and chlorine analyzers, larger chemical storage tanks, improved backwash valving and electrical and control modifications. In 2009 a new on-site chlorine generation and feed system was installed, allowing the City to avoid the continued use of chlorine gas.

After treatment, finished water is pumped into the Port Orford distribution system by two 25 HP centrifugal pumps. One pump operates at 290 gpm at 210 feet of dynamic head while the second pump operates between 370 gpm to 415 gpm. Both pumps normally operate independent but can operate in parallel to provide up to 550 gpm to the distribution system. Pump start/stop is controlled by the manual operation of the treatment plant. The 2 pump discharges are equipped with standard spring loaded, non-slam check valves, a flow rate control valve, and manual wafer butterfly valves for isolation. A single 2" combination air/vacuum release valve is located on the combined pump discharge pipe. Piping from the treated water pumps is also configured so that the pumps can be used for filter backwashing in an emergency. A new influent flow meter has been suggested to improve monitoring of the city's raw water supply line.

3.5 Treated Water Storage

Port Orford currently has two treated-water storage reservoirs. The main reservoir (Coast Guard Hill Reservoir) is a pre-stressed concrete 1.0 million gallon reservoir constructed in 1976. It is located midway up the Coast Guard Hill Road (See Figure 3) and has a high water level of 212 feet.

The City's other reservoir is a 0.2 million gallon rectangular concrete reservoir (Deady Street Reservoir), which was built in the early 1950's. Originally the City pumped water from its Gold Run source and later the North Fork Hubbard Creek source directly to this reservoir. This reservoir has an estimated high water level of 217 feet. Due to the age and condition of the tank roof, the Deady Street Reservoir has not been used in recent years. The City's Water System Master Plan recommends converting the tank into an emergency raw water storage/pretreatment facility.

A third reservoir is located on Hensley Hill. This reservoir is a 50,000-gallon rectangular concrete tank which supplies water to a residential development north of the City within the UGB. The City maintains the water system in this development, including the concrete tank.

3.5.1 Existing Finished Water Reservoir Capacity

Data for each tank is summarized in Table 3.2 below.

Table 3.2
Water Reservoir Data

	Coast Guard Hill Reservoir	Deady Street Reservoir	Hensley Hill Reservoir
Capacity	1 MG	0.2 MG	0.05 MG
Pressure Zone	Low level	Low level	High level
Floor Elevation	183	205	296
Max. Water Surface Elevation	212	214	306
Inside Diameter/Dimension	76.5	50′ X 50′	30′ X 20′

3.5.2 Existing Reservoir Condition

In 1996, level transmitters were installed in both reservoirs to help the City manage its finished water reserves. At the same time the reservoirs were cleaned and inspected and interior crack sealing applied. The exteriors of the tanks were both painted and/or treated. The Coast Guard Hill Reservoir is in good condition, however, the exterior should be treated and cleaned on a routine basis and a new level indicator should be installed. The roofing, fascia, and sheeting on the Deady Street Reservoir exhibit significant signs of dry rot and deterioration, and the concrete needs to be inspected. Consequently, the Deady Street tank is no longer in service. As discussed in the City's water master plan, the Deady Street tank should be converted to an online or offline raw water supply tank.

3.6 Distribution System

The distribution system extends from the storage reservoirs throughout the City in a classic grid iron configuration. In general the system is well looped although some areas of the piping system are undersized. See the Master Plan for more description of the distribution system.

3.6.1 Pipe Inventory

The City's distribution system is a mixture of pipe materials and sizes comprised of approximately 78,000 feet of piping (excluding individual services). More than 70% of the system is older asbestos cement pipe installed in the 1970s. The remaining portions of the piping system are PVC pipe. Much of this PVC pipe is older "Class" PVC pipe, a version of plastic pipe that is inferior to modern C-900 or C-905 PVC pipe. An inventory of the piping system is shown in Table 3.6.1.

Of the two materials, the AC pipe appears to be more susceptible to cracking and leaking than the PVC pipe, particularly in areas where ground movement occurs from seismic or unstable soil conditions. The condition of the AC pipe is believed to be contributing to high water losses in the system. Portions of the pipe have become soft or "spongy", a condition where the outside of the pipe becomes malleable which can allow joints and service connections to weaken and separate.

Unfortunately, portions of the PVC pipe in the system are also prone to sudden failures from ruptures. Problems with this pipe material have been associated with the older pressure class PVC pipe. Pressure class PVC is thinner than C-900 PVC pipe (commonly used today) and can become more brittle over time due to UV degradation and early formulations of polyvinyl chloride pipe.

Table 3.3
Pipe Inventory

	PVC Ft.	Asbestos Ft.	Total Ft.
2-inch	1,294	--	1,294
4-inch	2,244	13,970	16,214
6-inch	4,924	36,991	41,915
8-inch	10,417	--	10,417
10-inch	2,199	882	3,081
Totals	**19,784**	**53,137**	**72,921**

3.6.2 Pressure Zones

In accordance with OAR 333 residual pressures at service connections in a distribution system must never drop below 20 psi, which is approximately equivalent to a 46-foot tall column of water. Customers must be located more than 46-feet below the minimum water level in a storage tank (or effective elevation of a pressure reducing valve) to have a sufficient pressure without a booster pump. Storage tanks and pressure reducing valves are generally located to provide a pressure of less than 100 psi at the lowest service elevations in a pressure zone.

Most of the City's service area is in the main lower pressure zone, which extends from an elevation of 0 ft. (MSL) to approximate elevation 146 ft. (MSL). Users located above 146-feet elevation need to have their water pumped in order for the pressure to remain at acceptable levels during normal reservoir operating levels. There are four high pressure zones in the City's water system, each served by a booster pump station.

3.6.3 Booster Pump Stations

The water distribution system includes four booster pump stations. The largest of these, Coast Guard Hill serves approximately 50 residences. This pump station consists of three pumps and electrical controls all contained within a small building just below the Coast Guard Hill Reservoir. The pumps are part of a constant pressure package booster system provided by PACO Pump Company. With this system, pump controls sense the system pressure and, when the pressure drops due to water use, the pumps operate automatically to boost the pressure back-up to the desired set point. With all three pumps operating, this station should provide 500 gpm at a total dynamic head of 215 feet. Pump motors consist of one 15 HP jockey pump and two identical 25 HP pumps. Piping within this station is in poor condition. After many years of exposure to the marine environment, fittings are corroded and leaking and the control system is antiquated. Actual fire flow from the station is less than the rated 500 gpm due to the control system causing cycling of the pumps and surging of flows. Replacement of the package pumping system is desired by the City, unfortunately the building was not constructed to allow removal of the equipment without significant alterations to the building. A major upgrade is required for Coast Guard Hill.

There are currently three other small service areas where small pump stations boost the water pressure to serve 2 to 10 homes. These areas are as follows: the upper ends of Deady Street (north of 9th Street above Battle Rock Park), on the north end of the system up Vista Drive east of Jefferson Street serving the Hensley Hill tank, and on the south end of the City near the intersection of Highway 101 and Cemetery Loop Road. Of these stations, the Deady Street station is in poor condition and requires replacement. Portions of this high pressure zone are noncompliant with Health Division requirements because there are residential customers with less than the required 20- psi minimum operating pressure.

3.6.4 Unaccounted Water

The difference between the quantity of treated water pumped from raw water source to the distribution system and the quantity of water measured at customer meters is referred to as unaccounted water. The difference can be attributed to system leaks, inaccuracies in customer meters, unmetered services, and other unmetered use such as fire flows and system flushing. Since the City does not currently meter raw water at the source, losses in the raw water transmission line are unknown.

Table 3.6.4 illustrates the recent history of unaccounted water in the City's water distributions system. Current water losses exceed 50%-percent [one gallon out of every two gallons of treated water produced is lost to leakage]. The City is considerably over the limit of what is considered acceptable loss even without taking into account losses in the raw water transmission line. According to the Oregon Water Resources Department and Oregon Administrative Rules 690-86, all water systems should work to reduce unaccounted water levels to less than 15-percent with a ten percent water loss value being the goal for each water system. Within the last decade, the City has never had water losses below 20 percent even though they continually search for leaks, repair major line breaks, and continually replace their customer metering equipment.

Table 3.4
Annual Water Production and Consumption

Water Record Year	Raw Water Demand[1]	Treatment Plant Production[2]	Metered Consumption	System Losses
	MG	MG	MG	%
1999	50.64	45.07	35.58	21%
2000	64.39	39.16	39.16	28%
2001	75.38	66.96	39.00	42%
2002	75.57	68.70	41.55	40%
2003	67.13	61.02	37.63	38%
2004	65.61	59.65	38.42	41%
2005	74.87	38.06	36.01	47%
2006	85.03	77.30	35.43	54%
2007	83.77	69.42	33.52	52%
2008	72.18	59.74	30.04	50%
2009	77.36	62.35	29.32	53%
Average Unaccounted Water				**47%**

1. Metered at the water treatment plant.
2. Treated water plus 10% for backwash.

3.7 Financial Status

3.7.1 Current Rate Schedule

Water meters are read and billed every month and customers are billed for both water and sewer on the same bill. Effective July 1, 2010 rates will be as follows:

Base rate:

$25.64 (which includes first 2000 gallons)

Consumption rate:

First 0 - 2,000 gallons included in base rate
2,001 - 5,000 gallons billed at $6.48 per thousand gallons
5,001 - 10,000 gallons billed at $8.49 per thousand gallons
10,001 - 20,000 gallons billed at $9.49 per thousand gallons
Anything over 20,000 gallons billed at $10.49 per thousand gallons

Based upon water use records for the past three years, the average single family residence in Port Orford uses approximately 3,700 gallons per month. This equates to an average monthly water use bill of $37.00.

3.7.2 Annual Operations and Maintenance (O&M) Cost

The following excerpts from the City of Port Orford 2009-2010 annual operating budget represent the O&M costs associated with the City's water fund.

Table 3.5
2009-2010 Operating Budget
City of Port Orford

Account Name	Current Adopted Budget 2009 - 10	Account Name	Current Adopted Budget 2009 - 10
Personal Services		**Materials and Services**	
Overtime	5,000	Supplies	12,000
City Administrator	12,225	Software Maintenance	10,975
Finance Director	12,105	Uniforms	250
Office Clerk	11,770	Fuel (Equipment and Vehicles)	3,500
Accountant Assistant	3,432	Auditing / Accounting	6,469
Recorder / Clerk	860	Engineering	4,800
P/W Superintendent	31,140	Contract Services	500
WWTP / Operator	4,476	Legal Services	500
Utility Worker	10,685	Telephone	2,000
Seasonal Worker	5,480	Postage	1,700
Maintenance Worker #2	9,555	Travel & Training	2,800

Table 3.5, continued			
Account Name	**Current Adopted Budget 2009 - 10**	**Account Name**	**Current Adopted Budget 2009 - 10**
Personal Services		**Materials and Services**	
Public Works Tech	12,390	Insurance & Bonds	6,000
Social Security	9,110	Electricity	28,600
PERS Retirement	16,000	Repairs / Maint. - WTP	10,000
Workers Comp / Disability	3,500	Repairs / Maint. - Pump Station	5,000
Health, Dental, Life	36,720	Repairs / Maint. - Water Lines	8,000
Unemployment Insurance	3,000	Small Tools and Minor Equipment	1,264
Total Personal Services	187,448	Vehicle & Equipment Maint.	3,000
		Meter Repairs	2,500
		Dues & OR Statutes	400
		Permits	1,500
		Miscellaneous	200
		Equipment Testing	500
		Testing	3,500
		Total Materials and Services	115,958
Total Operations and Maintenance		**$**	**303,406**

3.7.3 Capital Improvement Plan

The Water Master Plan Update presents a capital improvement plan for the planned upgrade of the system components identified in need of improvements. That CIP included improvements to the City's source, treatment, storage, and distribution system with all proposed projects totaling approximately $8.4 million in 2005. Water production and use records have indicated that unaccounted water has increased dramatically since the Master Plan was completed. It has become more apparent that more thorough and immediate attention must be given to the distribution system along with insuring a source of supply for the next fifty years.

3.7.4 Users by Category

The City of Port Orford currently sells finished water through approximately 680 serviced connections associated with the following breakdown of user classification:

Table 3.6 EDUs by Customer Class		
Res & Small Com. EDUs	EDU	591
MF Res User EDUs	EDU	35
RV Park EDUs	EDU	57
No. Res, MF, RV & Sm Com Type EDUs	EDU	692
Exclusive Res Users w/o Sm Com	EDU	598
No. EDUs Large Commercial	EDU	129
No EDUs Small Commercial	EDU	94
Total EDUs	**EDU**	**821**

3.7.5 Existing Debts

The City of Port Orford's water funds currently support three separate long term debts.

1. During the 2008-09 fiscal year, The City Council approved an internal loan agreement between the Water SDC Fund to the Water Enterprise Fund. The principal amount was $63,850 with an interest rate of 2.5% per annum. This loan is to be repaid over a period of four years. The current annual payment including principal and interest for FY 09-10 is $16,972.

2. A Water Revenue Bond in the amount of $462,181 was issued in October, 1997 for improvements to the City's municipal water system. Annual payments including 5.01% interest are due December each year. The current annual payment including principal and interest for FY 09-10 is $43,023.

3. In 2005 the City entered into a loan agreement with Sterling Savings Bank in association with its purchase of a Vac-Con truck. Payments of $16,454 are made annually, including 5.50% interest, through October 2010.

4.0 Need for Project

The City's Water System Master Plan identified that raw water availability during dry weather months was insufficient during drought periods to meet the demands of the City. In addition, the high level of water losses (at that time 30 percent) would have a significant impact on the ability of the water supply system to meet future demand. It is not uncommon during the late summer months for the City to appropriate all of the flow in the North Fork of Hubbard Creek, causing the reach of stream below the dam to go dry. Additional water supply was recommended to address this issue and prevent the City from running out of water in the near future.

A more focused evaluation of the City's raw water supply needs and production within the North Fork Hubbard Creek watershed is presented in this section. A hydrological model of the existing watershed is also presented and used to analyze alternatives to address the City's supply shortages.

The City's projected demands for water were evaluated based upon current water usage, estimated growth rate, and significant reductions of unaccounted for water. Since completion of the Water Master Plan in 2005, water production and use records indicate that unaccounted water has increased dramatically. In 2000, unaccounted water was approximately 28% of the treated water production. As represented in Table 3.6.4, unaccounted water has now reached a level of 52% of production based upon records for the past three years.

4.1 Hydrological Model (North Hubbard Creek)

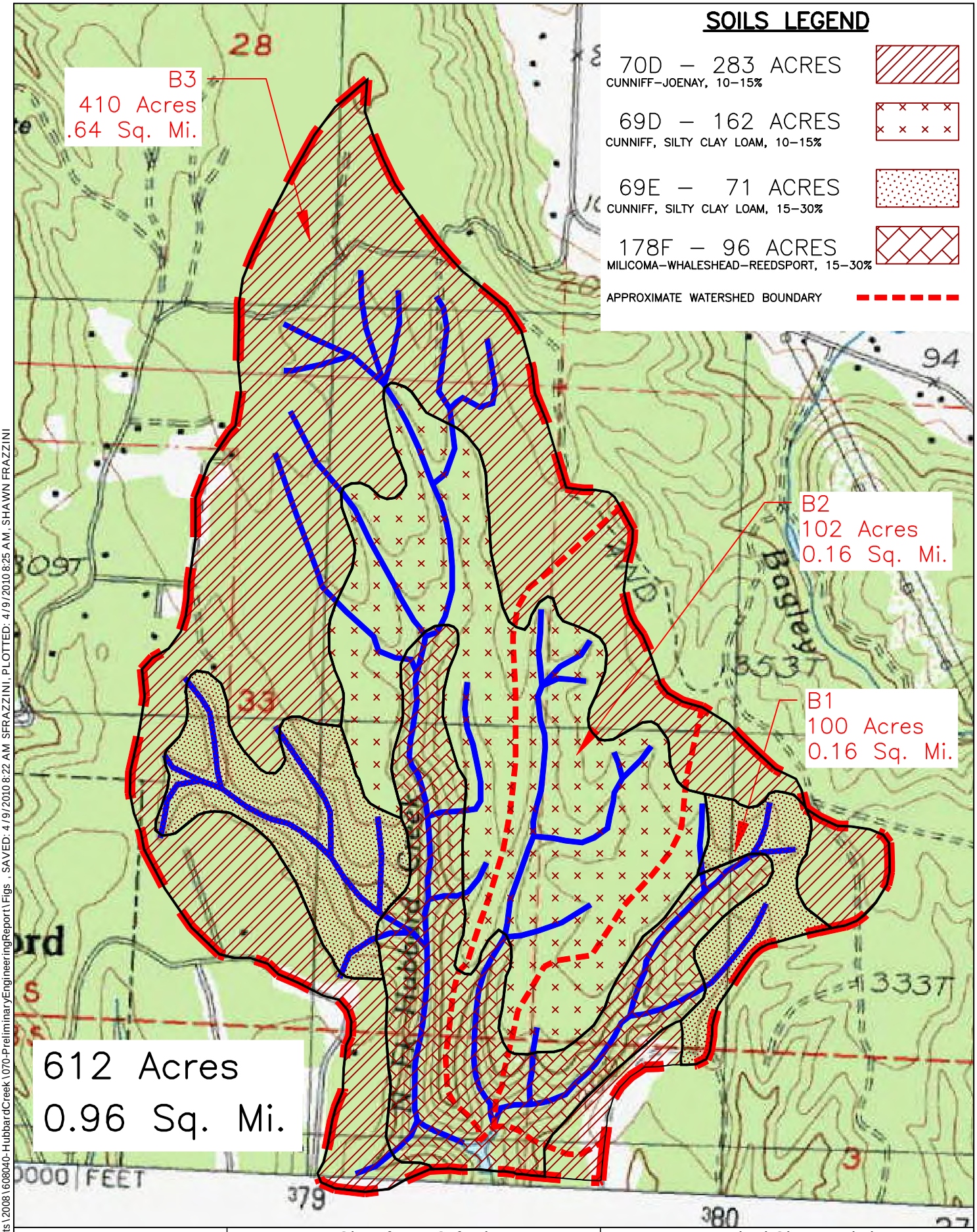
The North Fork Hubbard Creek watershed is an un-gauged tributary of Hubbard Creek which drains to the Pacific Ocean just south of the city. The lack of gauging data for this system required modeling expected run-off based on basin characteristics and precipitation. The Hydrological Modeling System HEC-HMS provided by the Army Corps of Engineers Hydrologic Engineering Center was employed to model the watershed and proposed impoundments. HEC-HMS is designed to simulate the precipitation-runoff process of dendritic watersheds (watersheds with tributary streams formed by erosion of sediments and taking on the pattern of the veins of leaves). The hydrographs produced by the model are used directly for determination of water availability, forecast flows, and reservoir spillway design.

4.1.1 HEC-HMS Methodology

The HEC-HMS modeling system allows the user to select from multiple methods for characterizing the watershed, defining meteorological events, and calculating infiltration and run-off. To develop a model for the Hubbard Creek watershed the Soil Conservation Service (SCS) Curve number methodology was employed to define run-off and infiltration characteristics. The SCS unit hydrograph was employed to calculate the predicted surface run-off volumes.

Basin Descriptions

The Hubbard Creek watershed has an estimated area of 0.94 square miles (612 acres) and can be divided into three primary sub-basins. These sub-basins are delineated in Figure 4.1.



\\Coosbay\projects\2008\608040-HubbardCreek\070-PreliminaryEngineeringReport\Figs , SAVED: 4/9/2010 8:22 AM SFRAZZINI, PLOTTED: 4/9/2010 8:25 AM, SHAWN FRAZZINI

SHN Consulting Engineers & Geologists, Inc.	City of Port Orford Hubbard Creek	Watershed Characteristics SHN 608040
April 2010	608040-Watershed Character-Fig_4-1	Figure 4.1

Curve Number

To use the SCS method, soil and ground covers are classified by curve numbers that have been derived empirically to give accurate estimates of the run-off and infiltration. Typical CNs are based on soil type and land use. The majority of the Hubbard Creek watershed is dense woods in good hydrological conditions. Soils types range from silty clay loam to the more permeable gravelly loam of the soils adjacent to the creek channels. The derivation of composite curve numbers for each of the three sub-basins is presented in Table 4.1.

Table 4.1
Hubbard Creek Watershed
SCS Curve Numbers

Soil		**70D Cunniff Joeney**		**69D Cunniff silty clay loam**		**60E Cunniff silty clay loam**		**178F Millacoma-Whaleshead-Reedsport**		
Slope		0-15%		0-15%		15-30%		30-40%		
Capacity in/hr		0.06-0.2		0.06-0.2		0.06-0.2		0.2-0.6		
Soil Type		C		D		D		B		
Land-use/Condition		Woods /Good		Woods /Good		Woods/Good		Woods/Fair		
Curve Number		70		77		77		60		
Basin	**Acres**	**Acres**	**%**	**Acres**	**%**	**Acres**	**%**	**Acres**	**%**	**Ave. CN**
B1	100	25	0.25	21	0.21	23	0.23	31	0.31	70
B2	102	20	0.20	62	0.61		0.00	20	0.20	72
B3	410	238	0.58	79	0.19	48	0.12	45	0.11	71

SCS Unit Hydrograph

A hydrograph is a continuous plot of instantaneous discharge versus time. The hydrograph results from a combination of physiographic and meteorological conditions in a watershed and represents the integrated effect of precipitation, hydrologic losses, surface runoff, interflow, and groundwater flow.

The concept of the SCS unit hydrograph was originally developed from observed data in small agricultural watersheds. Today the SCS method is in wide use as an accepted method for estimating runoff from a watershed. A unit hydrograph is a reflection of an actual hydrograph scaled by a time lag factor specific for each subbasin. By combining subbasin hydrographs according to the time lag produces a unit hydrograph suitable for modeling run-off in an entire basin. The time lag is defined as the length of time between the center of the precipitation and the peak flow of the resulting discharge hydrograph.

Studies by SCS have shown the time lag is approximately 60 percent of the time of concentration T_c, or travel time through the basin. The T_c, is the sum of overland or sheet flow, shallow concentrated flow (small rivulets) and channel flow. Each of these components were estimated for the basins in the North Fork Hubbard Creek watershed based upon a velocity calculated from the over-land

watercourse, channeled slopes, and an estimate of surface roughness. Calculated values for the time of concentration and lag time are summarized in Table 4.2

Table 4.2
Hubbard Creek Watershed Run-off Calculations

				B1		B2		B3	
			Vel.	Len.	T_c		T_c		Tc
	n	Slope	fps	Ft.	Hrs.	Ft	Hrs	Ft	Hrs
Sheet Flow	0.4	0.02	NA	300	0.68	300	0.68	300	0.68
Shallow Concentrated Flow	NA	0.02	2.2	500	0.06	500	0.06	500	0.06
Channel Flow	0.05	0.03	2.0	3500	0.48	4200	0.58	9400	1.30
Total Time of Concentration (T_c)					1.22		1.32		2.04
Lag Time(L_g)					0.73		0.79		1.23

Meteorological Descriptions

Several methods were utilized to examine the effects of precipitation on the predicted run-off. These includes

- Input of hyetographs: time series precipitation data from neighboring gauged weather stations,
- Input of hyetograph developed from statistical precipitation data,
- Design storms for varying return intervals based on intensity duration, and
- Design storms for varying return intervals using the SCS Type 1A storm.

Time-Series Precipitation Data

In the first method precipitation data from a gauged weather station in Brookings, Oregon (Weather Station 351055) was input directly. Precipitation in the Hubbard Creek watershed is comparable to that in Brookings as evidenced by the fact that the City is on an equivalent isopluvial contours for typical design storms. Precipitation data was downloaded for the period from January 1975 through December 2002 and used to model the discharge over a period of one year. The time series analysis provided the following:

- Daily averages providing the basis for the Annual Average Flow scenario;
- Daily precipitation in 1976 providing the basis for the Minimum Annual Flow scenario; and
- Daily precipitation in 1996 providing the basis for the Maximum Annual Flow scenario.

Statistical Precipitation Data

Precipitation data based on statistical summaries of monthly rainfall data reported by National Oceanographic and Atmospheric Administration (NOAA) was used to compare model generated stream flow to flow estimates generated by WARS program for 50 percent and 80 percent exceedance levels. Exceedence is defined as the probability, or percentage of time, that a streamflow will be greater than or equal to a defined stream-flow.

Design Storms

Design storms with return intervals from 12-years to 100 years were modeled based on isopluvials of 6 -hr and 24-hr storms published by the NOAA. All storm simulations were assumed to happen in January following a period of average rainfall in December, with ground conditions saturated and the stream flowing full at an average of 10 cfs.

The frequency storm method is designed to produce a design storm using statistical precipitation data for the intensity and duration of precipitation. Meteorological models for the design storms were created based upon the data available from isopluvials for the 6-hr and 24-hr storms as summarized in Table 4.3.

Table 4.3
Intensity Duration Data
Design Storms

Return Interval	6 Hrs[1]	12 Hrs[2]	24 Hrs[1]
Years	inches	inches	inches
2	2.2	3.0	5.0
5	2.4	3.5	5.5
10	2.8	4.1	6.5
25	3.4	6.0	8.0
50	3.6	6.2	8.5
100	4.0	6.5	9.0
500	4.8	7.0	11.0

1. Data for 6-hr , and 24-hr duration from NOAA isopluvials
2. Data for 12-hr duration extrapolated from the return period diagram

Design storms were also modeled using the SCS Type 1A 24-hr synthetic rainfall distributions developed by SCS for storms representing the Pacific maritime climate.

4.1.2 Model Results

Outflow from Hubbard Creek Reservoir was modeled using simulations employing a range of meteorological events. A summary of model simulations is included in Appendix B, Stream flow Analyses. Results from these simulations of run-off and stream discharge in the North Fork Hubbard Creek Watershed will be used for two purposes:

1. Estimating the minimum stream flow to determine additional reservoir storage capacity , and
2. Simulating the maximum flow for design of hydraulic control structures for the dam.

To verify reliability of the model, a minimum of one of three methodologies were employed to validate model results. The verification process included:

1. Statistical analysis of monthly flows from two small gauged watersheds with similar basin soil characteristics, area, and precipitation;

2. Peak discharge estimates from prediction equations provided by Oregon Water Resources Department (WRD), and;
3. Estimated flows at the 50% and 80% exceedence level as calculated by the WRD water availability program, (WARS).

Average Monthly Run-off

Simulation results based on recorded minimum, average, and maximum annual precipitation are summarized in Table 4.4. Results from the model are compared to minimum, average and maximum monthly average flows from the comparable gauged watersheds in the Alsea River Basin.

The average run-off was obtained by averaging the daily precipitation at the Brookings weather station for the period of record, 1975 - 2000. The minimum run-off was modeled using precipitation data from Brookings for 1976, the year with the lowest annual precipitation during the 25 year period. The maximum recorded annual precipitation occurred in 1996 so precipitation data from Jan 1996 – through December 1996 was entered to simulate run-off during a wet year.

Table 4.4
Hubbard Creek Flows - Simulated

	Minimum		Average		Maximum	
	HEC(1976)	Gauged WS[1]	HEC(AVE)[2]	Gauged WS[1]	HEC(1996)	Gauged WS[1]
	cfs	cfs	cfs	cfs	cfs	cfs
Jan	4.3	3.9	6.5	14.0	11.7	23.0
Feb	9.1	2.5	9.1	10.4	11.8	23.0
Mar.	4.6	3.1	7.2	8.7	3.6	17.0
April	3.5	2.3	5.2	5.1	5.9	9.7
May	0.9	1.1	2.9	3.0	4.6	6.2
Jun.	0.5	0.6	1.6	1.5	0.3	4.2
Jul,	1.1	0.4	0.4	0.7	0.4	1.6
Aug.	3.1	0.2	0.9	0.4	0.0	1.1
Sep.	0.4	0.2	1.6	0.6	1.0	2.1
Oct.	1.9	0.2	4.8	1.5	5.3	4.1
Nov.	3.4	1.8	9.2	7.4	14.9	13.5
Dec	1.7	5.2	10.4	11.8	14.7	25.0

1. Gauged water sheds US Geological Survey Alsea River Basin, Flynn Creek (1430689) and Deer Creek (14306810).
2. HEC Hydrological Modeling System

Statistical Return Intervals

The results of modeling based meterological data (monthly rainfall statistics) are summarized in Table 4.5 and compared to results obtained from WARS and from the comparative gauged sub-basins of the Alsea River Basin. The simulation assumes that stream flow will be proportional to

rainfall and does not take base flow conditions into account. Validation of the model predictions following periods of zero rainfall appear to under-predict stream flows during dry months.

Estimates of North Fork Hubbard Creek flows are based on a percentage of the flows predicted to occur at the mouth of Hubbard Creek. For simplicity, the percentage of flow was based on an equal percentage of the basin area.

Table 4.5
Statistical Estimates of North Fork Hubbard Creek Flow

	50% Exceedence				80% Exceedence				95% Exceedence[1]	
	WARS[2]		Gauged	HEC[3]	WARS		Gauged	HEC	Gauged	
	Hubbard	North F.	WS[4]		Hubbard	North F.	WS[3]		WS[3]	HEC
	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs
Oct	4.37	0.64	0.62	3.09	2.82	0.41	0.25	1.56	0.18	0.63
Nov	23.60	3.43	3.73	6.4	8.23	1.20	1.29	3.57	0.76	1.85
Dec	55.60	8.09	7.88	7.9	19.70	2.87	4.00	4.80	1.87	2.75
Jan	56.10	8.16	9.35	7.56	22.70	3.30	4.55	4.36	2.97	2.2
Feb	57.00	8.29	6.97	6.21	27.90	4.06	4.25	4.00	2.48	2.01
March	48.80	7.10	5.85	5.5	25.40	3.69	3.43	3.70	2.19	2.15
April	24.20	3.52	3.67	3.12	14.90	2.17	2.34	1.65	1.68	0.74
May	10.10	1.47	2.13	0.76	7.70	1.12	1.44	0.98	0.98	0.39
June	11.00	1.60	1.03	1.96	5.19	0.75	0.74	0.39	0.53	0.09
July	8.01	1.17	0.53	0.20	4.37	0.64	0.39	0.29	0.29	0
Aug	5.27	0.77	0.35	0.23	3.66	0.53	0.25	0.12	0.14	0
Sept	3.54	0.51	0.29	1.3	3.30	0.48	0.18	0.56	0.14	0.11

1. Values used for 95% exceedence flows are shown in bold.
2. WARS: Water Availability Reporting System, Water Resources Department.
3. HEC-HMS Modeling System, simulations based on precipitation statistics NOAA Atlas 20.
4. Gauged water sheds from the US Geological Survey for Alsea River Basin, Flynn Creek (1430689) and Deer Creek (14306810).

Storm Flow Scenarios

Simulated discharge from the North Fork Hubbard Creek watershed following storm events was modeled using two different methods of creating synthetic rainfall distributions. Both methodologies were based on ispopleths of 24-hr precipitation. In the first method, described previously as the frequency storm, the intensity of precipitation during the first six hours was based on isopluvials of 6-hr storm events. The second method, the SCS Type 1A storm, uses an empirically derived rainfall distribution to describe the 24-hr storm event

Simulation results for run-off from design storms are presented in Table 4.6. The rainfall distribution for a Type 1 A storm assumes that approximately 2/3 of the 24-hour precipitation occurs in the first 12 hours. The result is a high instantaneous run-off. The peak instantaneous run-off for the Type 1 A storm is sustained for approximately 1 hour, but the storm with a more evenly

distributed precipitation event has a sustained peak of approximately 4 hours. The average daily discharge flow is similar for both methodologies.

Table 4.6
North Fork Hubbard Creek Watershed
Discharge Resulting from Design Storms

Return Interval	Discharge							
	"Frequency Storm"		Type 1A Storm			Peak Discharge Estimates[2]		
	Peak (3-4 hrs.)	Peak Day	Peak (1 hr)	Peak (3-4 hrs.)[1]	Peak Day	Peak Flow	Lower Limit	Upper Limit
Years	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs
2	198	122	389	200	107	88.6	50.6	155
5	236	134	429	225	119	130	76.3	222
10	284	144	508	300	144	159	92.4	272
25	349	197	627	325	181	195	111	344
50	369	210	667	350	193	223	124	402
100	410	220	706	375	206	251	135	466
500	487	268	869	400	256	315	158	629

1. The four hour sustained peak estimated from graphical methods is included in Appendix B.
2. Prediction Equations from the WRD

Design Criteria

Model simulations of average monthly stream discharge in the North Fork of Hubbard Creek correlated well with the WRD predictions (in WARS), but during low flow periods following extended dry weather the model may underestimate stream flows. Since there is close approximation between the flows estimated based on the gauged watersheds tributary to the Alsea River and the flow reported in WARS, it is considered appropriate to use the gauged watersheds as a basis of predicting minimum flows in the North Fork of Hubbard Creek.

The discharge resulting from the design storms will be used to design appropriate discharge control structures for the dam. Preliminary modeling of the reservoir assumed a 40 foot weir length to control the high water level. At this length the one–hour instantaneous flow for the five-hundred year storm predicted for the Type 1A Storm raised the pool depth 3.8 ft. Based on this analysis, the following assumptions will be applied for determining the dam height and hydraulic control structure design.

- No dam overflow will occur during the five hundred year storm event
- A five foot freeboard will be provided over the top of the spillway to account for increased water level during extreme flow events and to provide for adequate free-board to prevent wind generated overtopping

4.2 Water Demands

Water requirements were updated from the Water Master Plan projections based on an analyses performed on the past three years of water usage records provided by the City. The monthly average water requirement for the City is summarized in Table 4.7.

Table 4.7
Monthly Average Water Demand[1]
City of Port Orford

	Raw Water[2] (gallons)	Treated Water (gallons)	metered usage (gallons)
Jan	6,996,000	5,899,667	2,430,833
Feb	6,572,000	5,084,667	2,125,567
March	7,332,000	5,327,667	2,241,384
Apr	6,097,500	4,760,333	2,186,463
May	6,673,000	5,545,333	2,746,387
June	6,773,000	5,744,333	2,714,547
July	7,518,333	6,278,667	3,386,797
Aug	6,642,000	5,603,667	3,282,450
Sept	5,795,000	5,036,000	3,006,120
Oct	5,724,667	4,500,667	2,342,443
Nov	5,709,333	4,643,667	2,148,893
Dec	6,343,000	5,410,667	2,605,637

1. Monthly data from consumption use records from 2007-2209.
2. Raw water usage estimated from treated water records plus 10 percent for WTP plant usage.

A community's water source must be capable of meeting the City's demand for water over a period of many years. Water rights that allow the appropriation of water are becoming more critical to small communities such as Port Orford because as the State's population and water demand increases, the number of viable water sources remains constant. Therefore, the water source(s) considered for Port Orford were evaluated to ensure enough water is available to meet the City's water requirements 50-years into the future.

Monthly raw water requirements for the City were projected 50 years into the future based upon anticipated population growth rates (1% per year). Projections were made considering; 1) no significant leak reduction in the system (52% losses), and; 2) with unaccounted water losses of 10%, pursuant to the goals of the Oregon Water Resources Department (discussed in Section 3). Water usage projections are summarized in Table 4.8.

Table 4.8 Raw Water Demand				
	Current		Projected	
	52%	10%	52%	10%
	MGD	MGD	MGD	MGD
Jan	0.226	0.121	0.282	0.151
Feb	0.235	0.126	0.288	0.155
March	0.237	0.127	0.303.	0.163
Apr	0.203	0.109	0.335	0.180
May	0.215	0.116	0.370	0.199
June	0.226	0.121	0.416	0.223
July	0.243	0.130	0.459	0.247
Aug	0.214	0.115	0.444	0.239
Sept	0.193	0.104	0.407	0.219
Oct	0.185	0.099	0.317	0.171
Nov	0.190	0.102	0.291	0.156
Dec	0.205	0.110	0.282	0.151

4.3 Storage Requirements

Storage requirements were evaluated through the use of a water balance comparing water supply with the water demand of the City including unaccounted water. As discussed in Section 4.1, monthly water production from the North Fork of Hubbard Creek during the storage period of the 95 percent exceedence year are based upon statistical data from the two Alsea River tributaries with flow gauging stations as summarized in Table 4.5.

4.3.1 Water Balance Scenarios

Water balance scenarios were created which included the following component:

- Supply based on North Fork Hubbard Creek at 95% exceedence level
- Projected 50-year demand including unaccounted for water at 50 percent and 10 percent
- Losses due to evaporation
- Maintenance of minimum stream flow of 0.14 cfs
- Implementation of a conservation plan resulting in 15 percent reduction in water usage

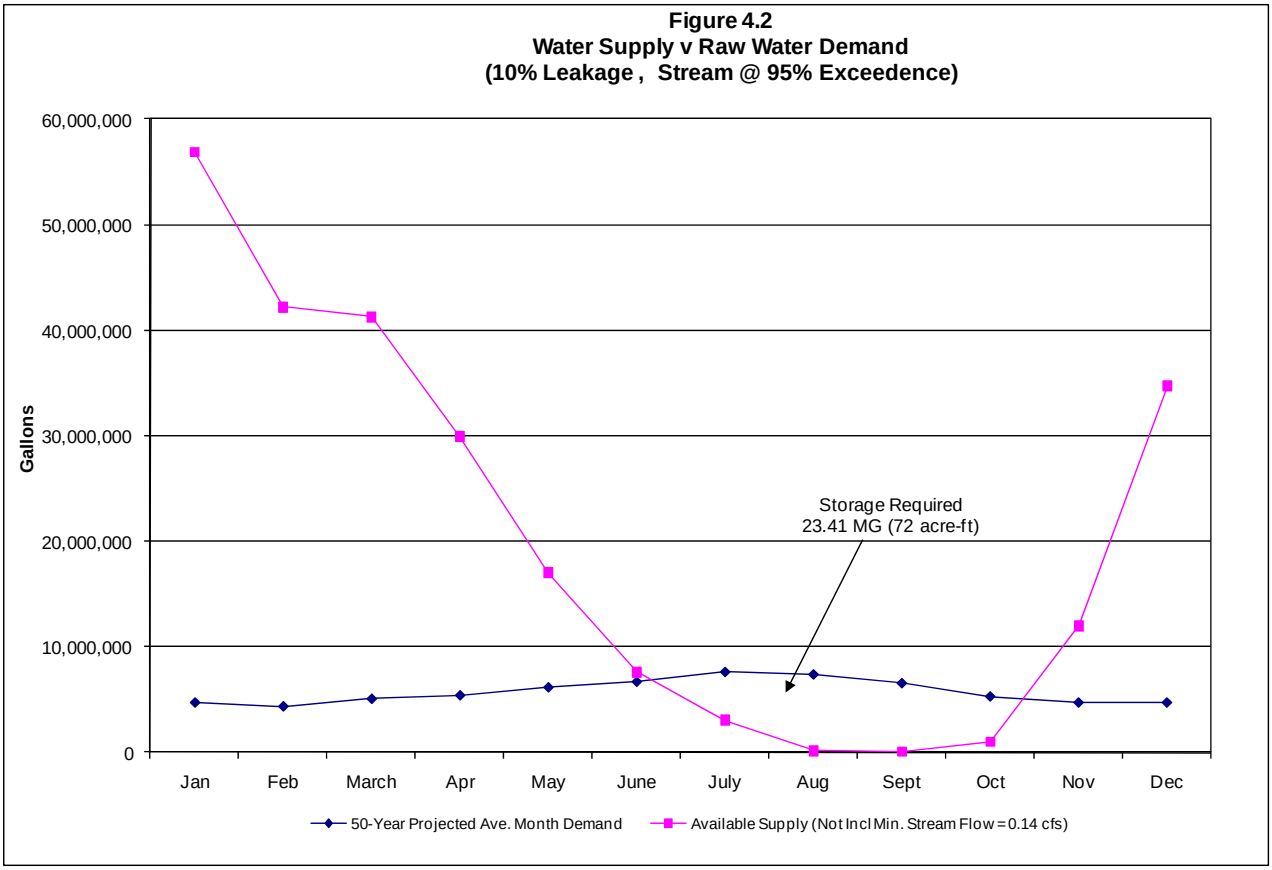
Scenario 1 – 50 percent water losses

Considering existing water usage, implementation of conservation during drought periods, 50 percent water loss rates, and release of minimum stream flows during the summer, it was estimated that a storage impoundment of 158 acre-feet would be required. An impoundment of this size would require a dam height of 50 feet or greater. Increasing the height of the existing dam to create an impoundment large enough to accommodate the City's water demand with a 50 percent or greater water losses is not good practice. In order to economically address increasing the source of

supply for the City, the distribution system losses need to be reduced to the WRD recommended rate of 10 percent.

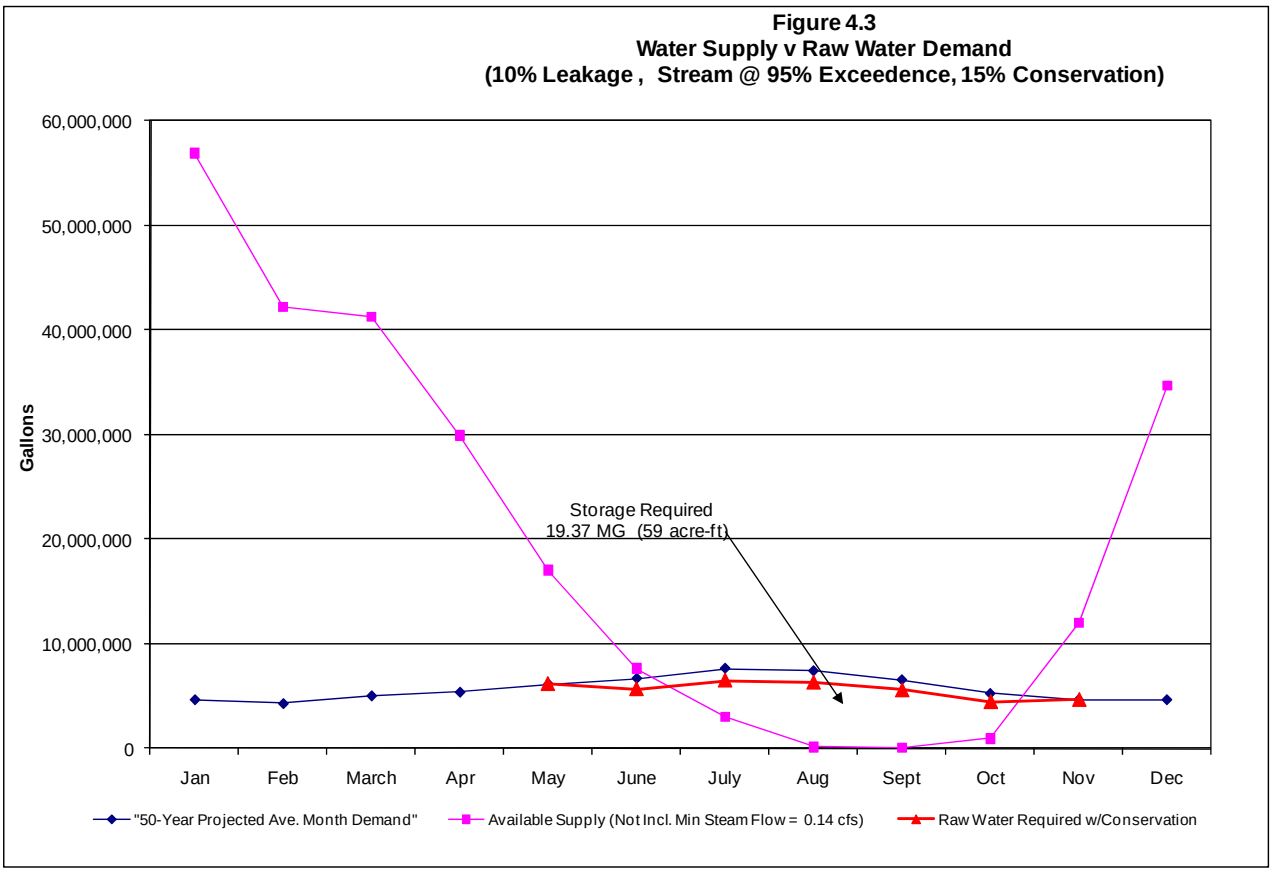
Scenario 2 – 10 percent water loss.

Scenario 2 considers the size of the impoundment required for the 95 percent exceedence year (1 in 20 year drought) assuming leakage rates in the distribution system are reduced to 10 percent or less and conservation is not employed to curtail water usage. If water demand during the drought is not curtailed by conservation, and the available water supply is reduced by 0.14 cfs to provide minimum stream flows; it is estimated that storage of 72 acre feet would be required. This analysis is presented graphically in Figure 4.2.



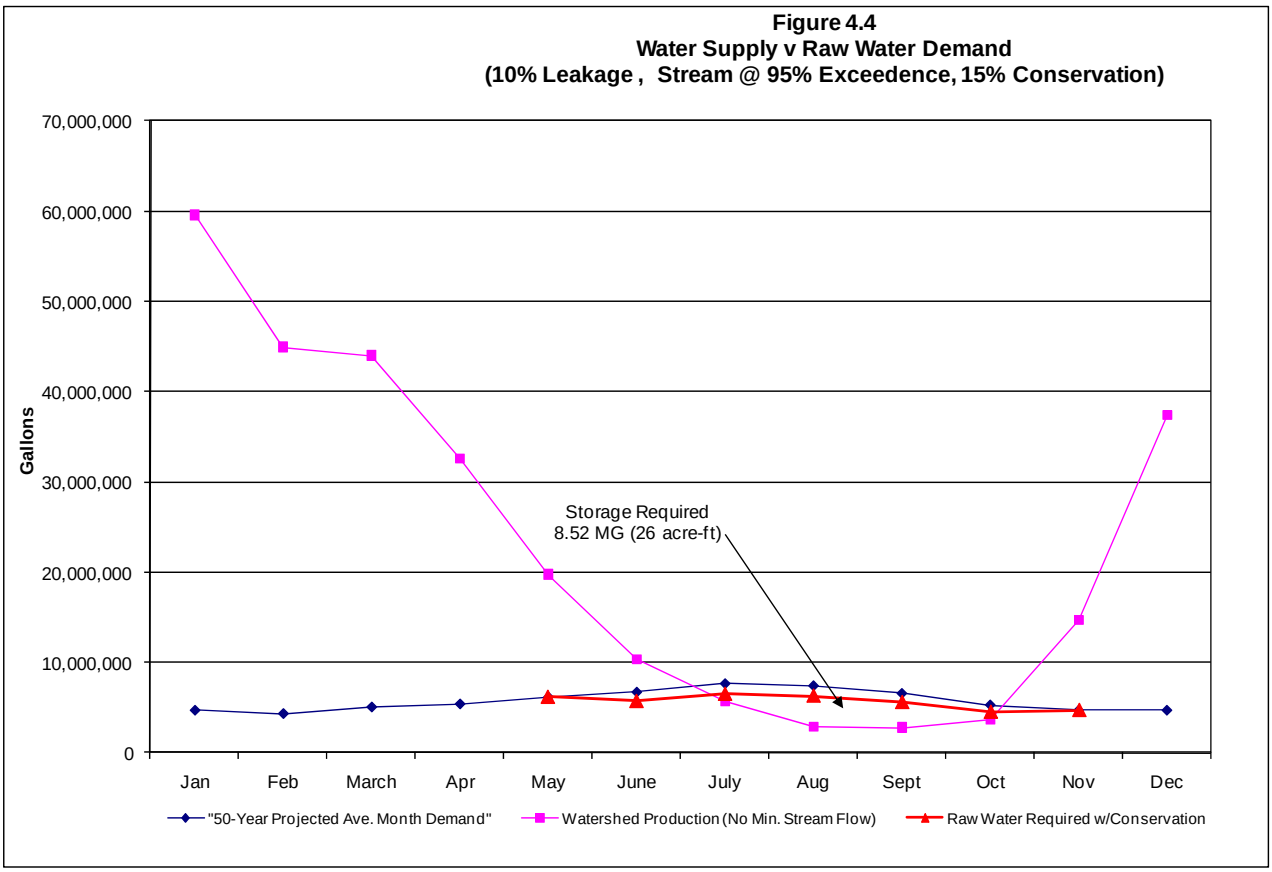
Scenario 3 - Conservation

As currently practiced by the City, it is assumed that a conservation plan will be implemented during drought years to reduce water use during the period of minimum stream flows. With this scenario, the required storage is reduced by approximately 13 acre-ft (4 MG) resulting in a more economical dam height of 38 feet and a reservoir sized to accommodate 59 acre feet of storage. Figure 4.3 represents the water balance assuming summer time water usage is reduced by 15 percent due to conservation measures, and water losses are reduced to 10 percent or less. The water balance analyses also include maintaining a minimum stream flow of 0.14 cfs.



Scenario 4 – No Stream Flow

If all of the water produced by the North Fork Hubbard Creek watershed is stored because it is not considered necessary to maintain a minimum flow in the creek downstream of the reservoir, the size of the required impoundment could be reduced to 26 acre ft. (assuming conservation practices are implemented). This analysis is shown in Figure 4.4. In this scenario, water would not be released during the late summer, consequently, impacts to down stream users may need to be mitigated.

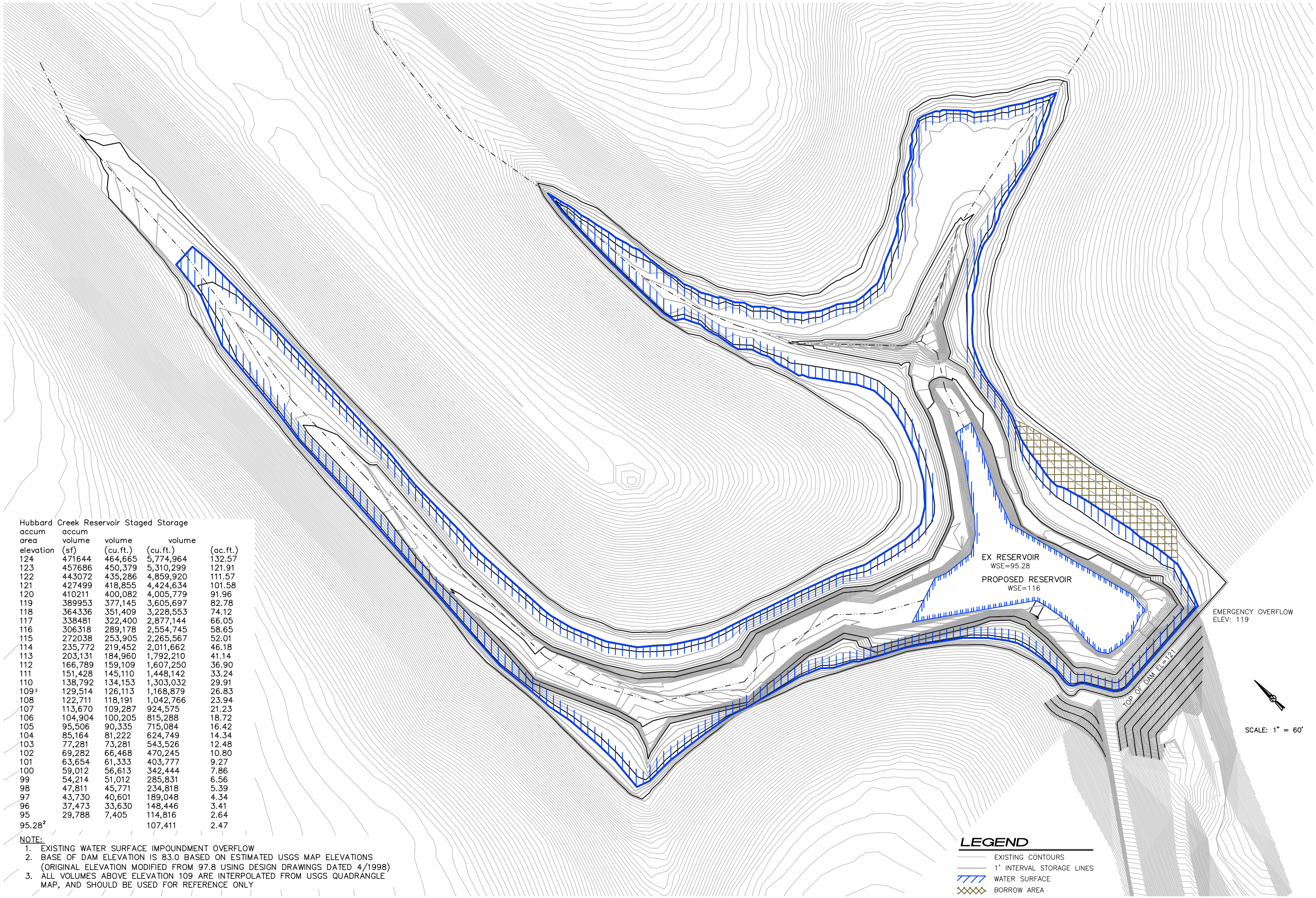


Summary of Storage Options

Storage requirements for an impoundment on the North Fork Hubbard Creek will need to provide storage for average monthly flows projected for the year 2060 during a drought equal to the 95 percent exceedence (1 in 20 year event). The amount of storage considered will be affected by distribution system leakage rates, conservations practices and whether it is necessary to maintain minimum stream flows downstream of the impoundment. Resulting storage requirements for four scenarios are summarized in Table 4.10. Impoundment water surface elevations are estimated from the stage storage relationship presented in Figure 4.5.

Table 4.10
Reservoir Requirements
50-year Projected Demand
City Of Port Orford

Leakage	Consv.	Min. Flow	Required Storage		Water Surface	Reservoir Height
Percent		cfs	Acre -ft	MG	ft	Ft.
51	Yes	0.14	158	51.61	45	50
10	No	0.14	72	23.41	35	40
10	Yes	0.14	59	19.37	33	38
10	Yes	0	26	8.52	26	31
10	No	0	39	12.55	30	35



Hubbard Creek Reservoir Staged Storage

accum area elevation	accum volume (sf)	volume (cu.ft.)	volume (cu.ft.)	(ac.ft.)
124	471644	464,665	5,774,964	132.57
123	457686	450,379	5,310,299	121.91
122	443072	435,286	4,859,920	111.57
121	427499	418,855	4,424,634	101.58
120	410211	400,082	4,005,779	91.96
119	389953	377,145	3,605,697	82.78
118	364336	351,409	3,228,553	74.12
117	338481	322,400	2,877,144	66.05
116	306318	289,178	2,554,745	58.65
115	272038	253,905	2,265,567	52.01
114	235,772	219,452	2,011,662	46.18
113	203,131	184,960	1,792,210	41.14
112	166,789	159,109	1,607,250	36.90
111	151,428	145,110	1,448,142	33.24
110	138,792	134,153	1,303,032	29.91
109[3]	129,514	126,113	1,168,879	26.83
108	122,711	118,191	1,042,766	23.94
107	113,670	109,287	924,575	21.23
106	104,904	100,205	815,288	18.72
105	95,506	90,335	715,084	16.42
104	85,164	81,222	624,749	14.34
103	77,281	73,281	543,526	12.48
102	69,282	66,468	470,245	10.80
101	63,654	61,333	403,777	9.27
100	59,012	56,613	342,444	7.86
99	54,214	51,012	285,831	6.56
98	47,811	45,771	234,818	5.39
97	43,730	40,601	189,048	4.34
96	37,473	33,630	148,446	3.41
95	29,788	7,405	114,816	2.64
95.28[2]			107,411	2.47

NOTE:
1. EXISTING WATER SURFACE IMPOUNDMENT OVERFLOW
2. BASE OF DAM ELEVATION IS 83.0 BASED ON ESTIMATED USGS MAP ELEVATIONS (ORIGINAL ELEVATION MODIFIED FROM 97.8 USING DESIGN DRAWINGS DATED 4/1998)
3. ALL VOLUMES ABOVE ELEVATION 109 ARE INTERPOLATED FROM USGS QUADRANGLE MAP, AND SHOULD BE USED FOR REFERENCE ONLY

VERIFY SCALES
BAR IS ONE INCH ON ORIGINAL DRAWING
0 1"
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

SHN CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501 FAX (707)441-8855 (707)441-8877

NO.	DATE	REVISION	BY

DSGN –
DR JTG
CHK CRJ
APVD

HUBBARD CREEK RESERVOIR
T.33S., R.15W., SEC. 04
PORT ORFORD, OR

STAGE STORAGE EXHIBIT

SHEET FIG 4.5
DATE 4/1/10
PROJ. NO. 608040.070

A new reservoir should be constructed to keep the City of Port Orford from running out of water in the North Fork Hubbard Creek Impoundment during a moderate or worse drought event. Before the reservoir is constructed, the City must make repairs to the water distribution system to achieve a leakage rate of 10 percent. Conservation measures should be considered especially during drought years. The design should also consider maintaining a minimum stream flow of 0.14 cfs. Based on these criteria, an impoundment providing 59 acre feet is recommended. Preliminary design criteria for the recommended reservoir are summarized in Table 4.11.

Table 4.11
Hubbard Creek Impoundment
Preliminary Design Criteria Year 2060

Reservoir	
Storage	59 Acre-ft (19.37 MG)
Water Surface	33 feet
Height[1]	38 feet
Spillway and Outfall	
Capacity	500-Year storm Event
Peak Day Flow	270 cfs (174 MGD)
Sustained Peak	490 cfs (317 MGD)
Peak Hour	870 cfs (572 MGD)
Weir Length	40 feet

1. Includes 5 feet of freeboard

5.0 Alternatives Evaluation

The Water Master Plan Update presented three feasible alternatives for providing a long term water supply for the City of Port Orford. The three alternatives, plus the alternative of relying on the existing supply (the do nothing alternative) have been considered as part of this Preliminary Engineering Report. All alternatives are based on the assumption that major improvements to the water distribution system are required to bring the leakage rate down to 10 percent or less.

Alternative 1	Do Nothing Alternative- No new water supply, improve distribution system and in-line storage including Deady Street Reservoir.
Alternative 2	Expand Existing Impoundment – Increase storage capacity to 59 acre feet.
Alternative 3	New Impoundment – Increase storage capacity to 59 acre feet with new impoundment.
Alternative 4	Garrison Lake Source – Construct treatment facilities to allow Garrison Lake to be used as the primary source.

5.1 Do Nothing

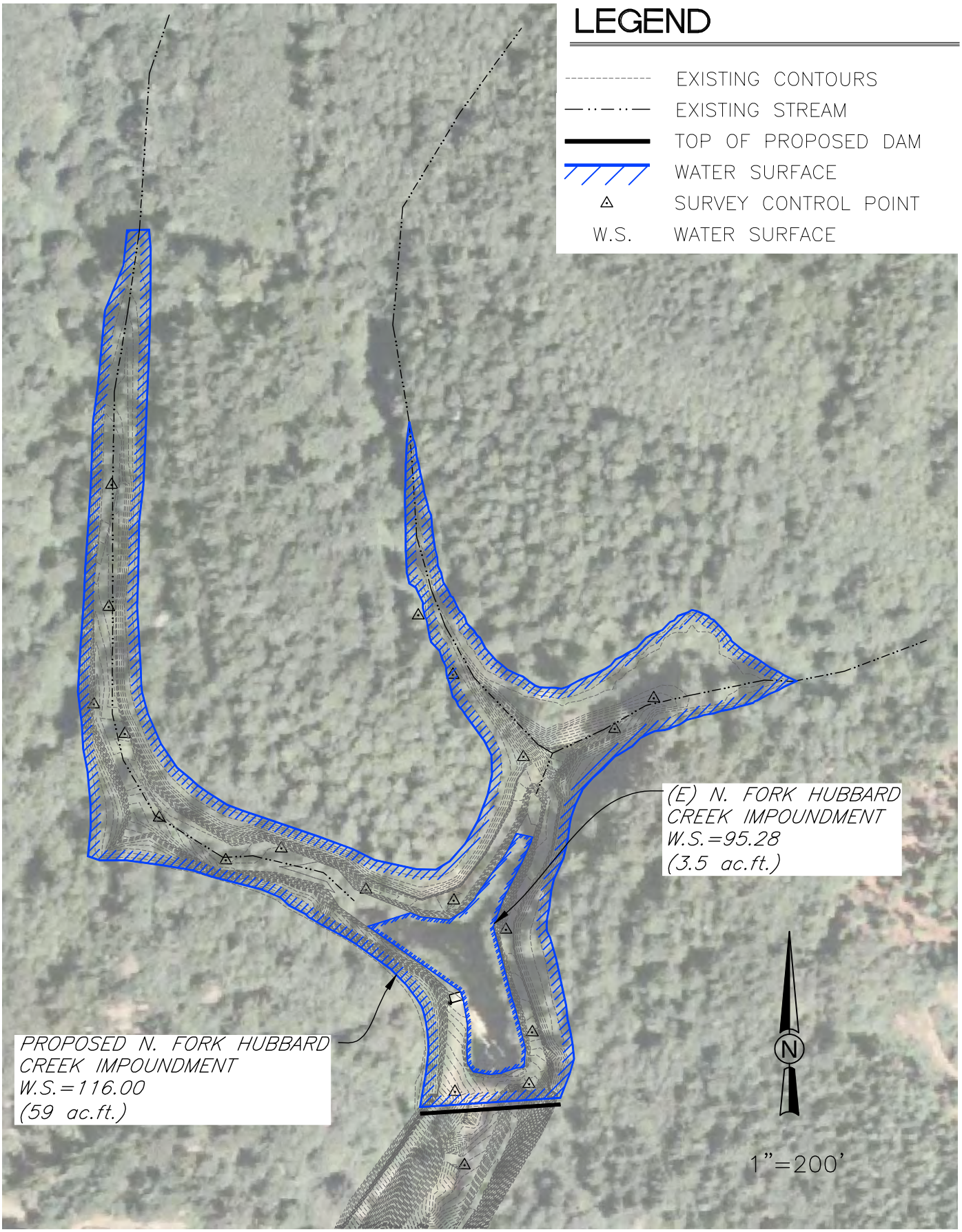
The do-nothing alternative is not acceptable as the leakage of unaccounted water in the system is excessive and promulgating unnecessarily high O&M Costs along with stressing the current water supply. In addition to tightening up the distribution system, the current water source is not adequate without adding storage to allow some stream discharge during the dry months and for assuring the City a reliable source for the next 50 years. In order for Hubbard Creek to continue to serve as the primary source of water for Port Orford system leakage needs to be addressed, additional storage would need to be provided by expanding the existing impoundment, by constructing a new impoundment, or by implementing both alternatives.

5.2 Expand Existing Impoundment

Based on the analysis presented in Section 4, an impoundment of approximately 59 acre feet will be necessary to provide the storage required to assure adequate water supply during dry months while accommodating long-term growth. Estimates of staged storage based on recent survey information indicate that the water surface of Hubbard Creek Impoundment would need to be raised by approximately 26 feet to a new height of 38 feet to provide the required storage (see Figure 5.1).

Raising the existing dam to the height required will require construction of a new earthen dam behind the existing dam. Allowing for 3 feet of free-board between the maximum water surface for the emergency spillway and an additional 2 feet to prevent over flow, the top of the new dam will need to be raised to 38 feet. The new elevation will be 121 feet measured from the base of dam at elevation 83 feet.[1]

[1] The base elevation is based on Port Orford Water System Improvements, Sheet B1 which shows the bottom of impoundment 12 ft below overflow. (April 1998, HGE Architects and Engineers)



SHN Consulting Engineers & Geologists, Inc.	City of Port Orford North Fork Hubbard Creek Impoundment Port Orford, Oregon	Proposed Reservoir SHN 608040.070
	April 2010	608040-Proposed Reservoir-Fig_5-1	Figure 5.1

\\Coosbay\projects\2008\608040-HubbardCreek\070-PreliminaryEngineeringReport\Figs. , SAVED: 4/2/2010 3:30 PM SFRAZZINI, PLOTTED: 4/5/2010 3:45 PM, SHAWN FRAZZINI

5.3 New Impoundment

Based on the topography of the North Fork Hubbard Creek basin, there is a potential new reservoir site located approximately 1/3 mile upstream from the existing impoundment (Figure 5.2). Above this location, the Canyon widens considerably prohibiting the economics of dam construction. Appropriation of water at this site will require that the City apply for a new permit to store water and a secondary right to release the water to the lower impoundment where it could be appropriated under the existing water right. It may also be feasible to increase the lower impoundment water right and thereby increase the City's total Hubbard Creek water right. While the option appears feasible at this preliminary stage, there will be extensive permitting, geotechnical, and environmental issues that must be addressed before the City could actually implement this improvement.

Based on existing topographical information a staged storage relationship was developed for the upstream site. To provide the required storage of approximately 59 acre feet, a water depth of approximately 33 feet would be required. It is estimated that the height of the required dam would be 38 feet which is comparable to the height of the dam required to expand the existing impoundment. Both dams would have a span of approximately 90 feet.

When compared to expanding the existing impoundment by construction a new dam, locating the dam upstream would have some major disadvantages, which will increase construction costs relative to the cost of expanding the existing impoundment.

- Lack of access will require additional construction cost for a new road access,
- siting of new pump station,
- power supply, and
- the new dam will require additional transmission line.

5.4 Garrison Lake Source

The City has an intake on Garrison Lake that has not been used since 1993 due to poor water quality. The use of Garrison Lake to supplement the Hubbard Creek source during dry weather periods would require additional measures to improve water quality.

Salinity in Garrison Lake has impacted the existing Garrison Lake source resulting in poor treated water quality and taste concerns. In recent years, conditions in the lake have deteriorated and the existing intake may not provide water that can be made potable without requiring significant and complicated treatment technologies. The Master Plan suggested relocation of the intake to the upper arm of the lake above Arizona Street and increased treatment such as ultra-filtration or reverse osmosis to remove salinity.

5.4.1 Intake Relocation

SHN visited the lake's intake site to evaluate how to relocate the intake to an area protected from salt-water intrusion. Options appear to be limited to relocating the intake to the deepest pool east of Arizona Street. In this location, salinity levels should be less due to the location being up gradient of the lower lake and isolated by the bridge and the Arizona Street roadway prism. (The

Lake Rehabilitation Study addresses other alternatives for preventing saltwater intrusion in the lake that are beyond the scope of this PER).

Relocation of the intake to a new location will involve transfer of water rights; installation of a new raw water transmission line either on-land or along the lake bottom; improvements to the raw water pump station or construction of a new raw water pumping facility; and construction of a new multi level intake.

5.4.2 Intake Design

In the past algae growth in the lake has posed problems with taste and odor. The ability to withdraw water from various lake depths may eliminate this problem. Water quality sampling should be conducted at the proposed site to determine the effects of salinity and algae on water quality. The sampling and analysis should also include determination of whether there are any occurrences of blue-green algae. Access to allow isolating various intake levels will need to be considered. It may be more desirable to install several pipelines from the shore to each intake screen, allowing the City to select an intake level by modulating valves from shore.

5.4.3 Treatment

Because of the high levels of organics and salinity in Garrison Lake it will be necessary to use membrane filtration to treat this source. Membrane filtration units provide clarification, softening disinfection, and organic removal, in compact automated modular units and they provide a positive physical barrier between contaminants in the feed water and the purified product water. To remove salinity Reverse Osmosis (RO) high pressure membranes will be required

Since the lake can be classified as brackish due to salt-water intrusion, the removal of salt and other metal ions will require RO membrane systems. Depending on the degree of salinity it may be possible to purify a portion of the raw water source and blend it with water that has been treated using ultra-filtration membranes. Ultra-filtration will remove: viruses, bacteria, particulates, and high molecular weight compound greater than 0.01 microns but will not remove divalent ions such as salt. Micro-filtration or ultra-filtration would also be required as a pre-treatment step preceding the RO membranes. These treatment systems are feasible, but require significant operator training and skill. It is likely that a person trained for this type of plant would be paid more to work for a large city where wages would be more commiserate with their skills.

5.4.4 Transmission Line

Two options for construction of the transmission line involve a submerged line running along the lake bottom or an on-land alignment constructed from Arizona Street to the 18th Street boat ramp. Both options would connect to the existing raw water intake line.

Routing of the pipeline across the lake bottom is the preferred option but would require addressing several permitting and engineering issues. Another important engineering issue is how to place the line under the bridge since in water work will be difficult to construct and may not be permitted with heavy equipment. It is assumed, therefore that the work under the bridge will involve hand excavating a trench and encasing the pipe in a concrete backfill for protection from vandalism and scour. If the water line can be installed across the lake, then the existing raw water pumping facility

will need to be upgraded to serve the new intake. The upgrade would need to address selection of pumping equipment that provided the necessary suction lift required for the extended pipeline. If the on-land route is selected, then a new raw water pump station will be required.

5.4.5 Source Protection

To use Garrison Lake as a primary source of water would require that the City implement policies to protect Garrison Lake water quality and reverse problems of eutrification and water quality deterioration. These are difficult changes for the City to implement without County and State support since the northern half of Garrison Lake is under Curry County's jurisdiction and Highway 101 crosses over the lake. As future development occurs along the lake, this issue will become more important, since invariably, lakefront property will be highly desirable. Policy issues addressing storm water discharges and potential prohibition against septic systems along the lakeshore will need to be considered. Because most of the residential development along the northern portion of the lake is located outside the City's jurisdiction, Curry County participation would be required.

5.5 Evaluation of Alternatives

5.5.1 Cost Comparison

Costs for the three supply alternatives were compared based on preliminary estimates of presented in the Water Master Plan Update. A construction cost for installing a new intake, treatment system, and transmission line for water from Garrison Lake was comparable to the estimated cost for the new dam constructed on the North Fork of Hubbard Creek. The new treatment system will be technically complex and increased costs for operation and maintenance should be anticipated.

Table 5.1
Water Supply Alternatives
Preliminary Estimate of Cost[1]

	Garrison Lake	New Dam Existing Site	New Dam Upstream Site
Construction Cost	$4,492,350	$3,829,275	$4,480,252
Power costs[1]	$5,062	$10,124	$10,124
Additional O&M	$100,000	-	
Present Value 0&M[2]	$714,662	$206,352	$206,352
	$5,312,074	$4,045,751	$4,696,728

1. Power costs include estimated pumping cost for raw water and additional cost for RO pressure system
2. Present value over thirty year time period assuming federal real discount rate of 2.7%

5.5.2 Evaluation Matrix

The estimated present value cost of the water supply alternatives are close enough that other project criteria may result in the preferred project having a higher cost factor than the lowest cost option . Based on the evaluation matrix presented in Table 5.2 constructing a new dam to expand the existing Hubbard Creek Impoundment is the preferred alternative, not only because it is the lowest cost project, but also because of the other values provided by the project.

Table 5.2 Evaluation Matrix			
	Garrison Lake Source	**Expand Existing Impoundment**	**Upstream Dam**
Land Acquisition	Required	Some Required	Not Required
(1-3)	2	2	3
Water Rights	Move Intake	Move intake, Increased storage	New Intake, New storage
(1-3)	2	2	1
Operational Complexity	Membrane water treatment	Similar to existing	Release and storage required
(1-3)	1	3	2
Quality / Reliability	Potential for Taste and Odor and Salinity	High Quality	High Quality / Less Area
(1-3)	1	3	2
Environmental Considerations	Construction of intake	Impacts: downstream flow, riparian areas, fisheries	
(1-3)	2	1	1
Present Worth Cost			
(2-6)	4	6	4
Total Score	**12**	**17**	**13**

6.0 Proposed Project

The proposed project consists of two phases. The first phase of the project addresses the high water loss associated with the City's distribution system. The second phase addresses the City's need to expand its raw water storage facility. While either phase can be accomplished independent of the other, the water losses in the distribution system must be reduced before the City runs out of water during a drought year. Furthermore, it would appear to be poor planning if the City constructed a new impoundment when over half of the water stored in the facility would be lost during distribution. Based on the watershed production analyses performed for the North Fork of Hubbard Creek, the City will not have sufficient water during a year with rainfall less than the average year.

6.1 Distribution System Improvements

The distribution system improvements include replacing aged and deteriorated pipe, installing new fire hydrants, valves, appurtenances, service laterals and pump stations throughout 90 percent of the existing distribution system. The project is proposed to be performed in stages to minimize disruption of service and the inconvenience of construction activities to small impact areas within the City. When one phase has been completed, the next phase would be initiated, moving construction impacts to another area of town. Phasing areas and sequencing will be determined during the design period of the project.

Figure 6.1 depicts the various elements of the water system replacement project. A detailed cost estimate of for each phase of the distribution system project is provided in Appendix C

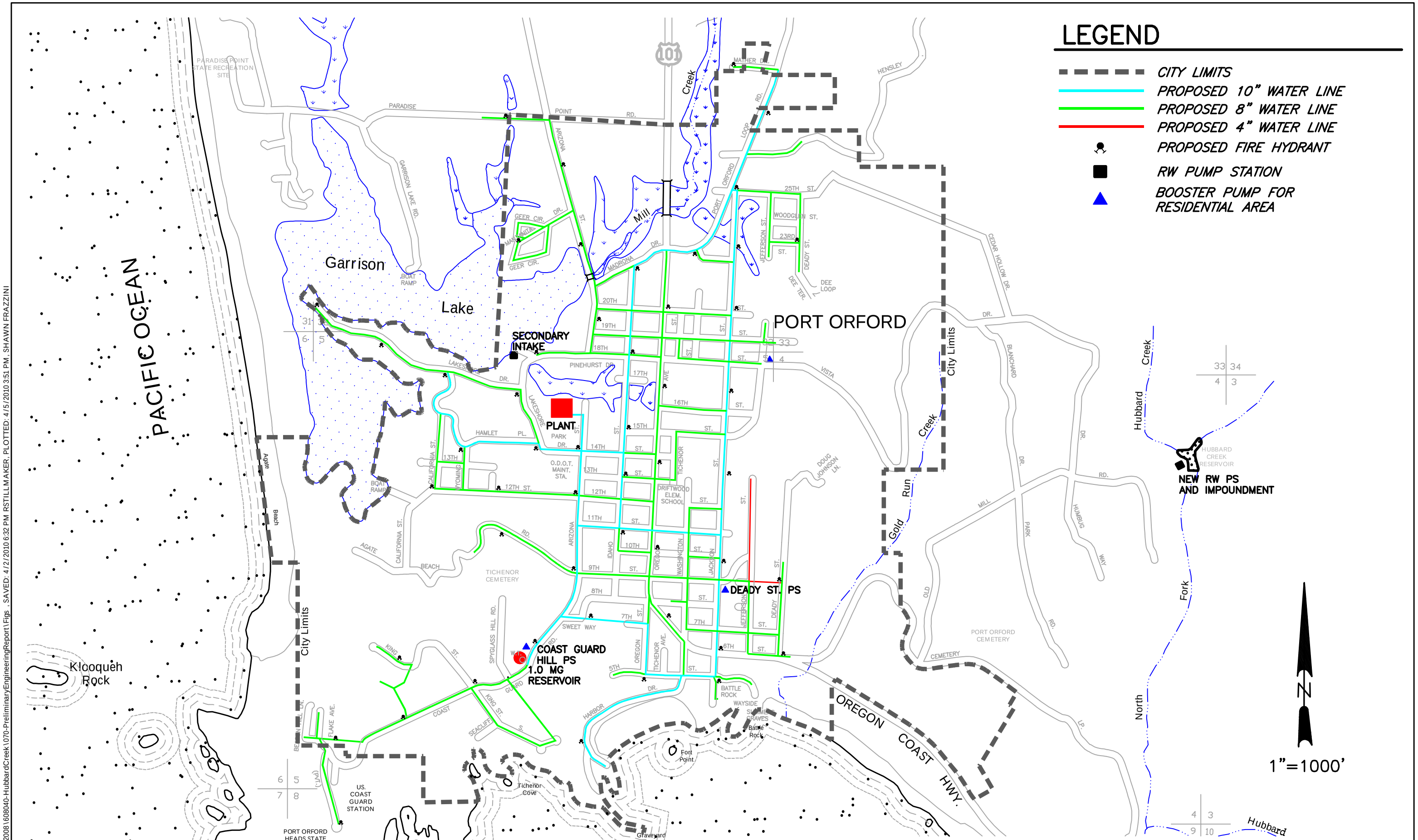
6.2 Reservoir Expansion

The second phase of the project involves securing an adequate water source for the City through expanding the dam and impoundment. The new earthen dam would have a height of 38 feet from the bottom of the existing stream channel to the crest. The impoundment would expand from the existing 3.2 acre-feet storage capacity to a maximum of 59 acre-ft. This phase of the project would also involve installation of new raw water intake structure, principal spillway, bottom drain, stilling basin and a new raw water pumping and pretreatment facility. Figures 6.2 through 6.4 provide a conceptual design of the proposed new earthen dam structure and associated reservoir improvements.

6.2.1 Earthen Dam Construction

Earthen dams consist primarily of the dam structure and associated hydraulic elements. The type of dam structure is primarily dictated by the foundation materials (pervious vs. semi pervious) and the type of materials available in the near vicinity for creating the embankment of the main dam structure.

Published geologic literature and surficial geologic investigations indicate the proposed reservoir and dam site area is underlain by bedrock consisting of silicified sandstone, siltstone, and shale characteristic of the early crotaceous age Rocky Point Formation. Cut bank exposures and shallow test pits encountered highly to moderately weathered, moderately to highly fractured sandstone



SHN Consulting Engineers & Geologists, Inc.

City of Port Orford
North Fork Hubbard Creek
Impoundment Porject

Water Distribution System Improvements
608040

March 2010 | 608040-W-Distrib-Improvements-Fig_6-1 | Figure 6.1

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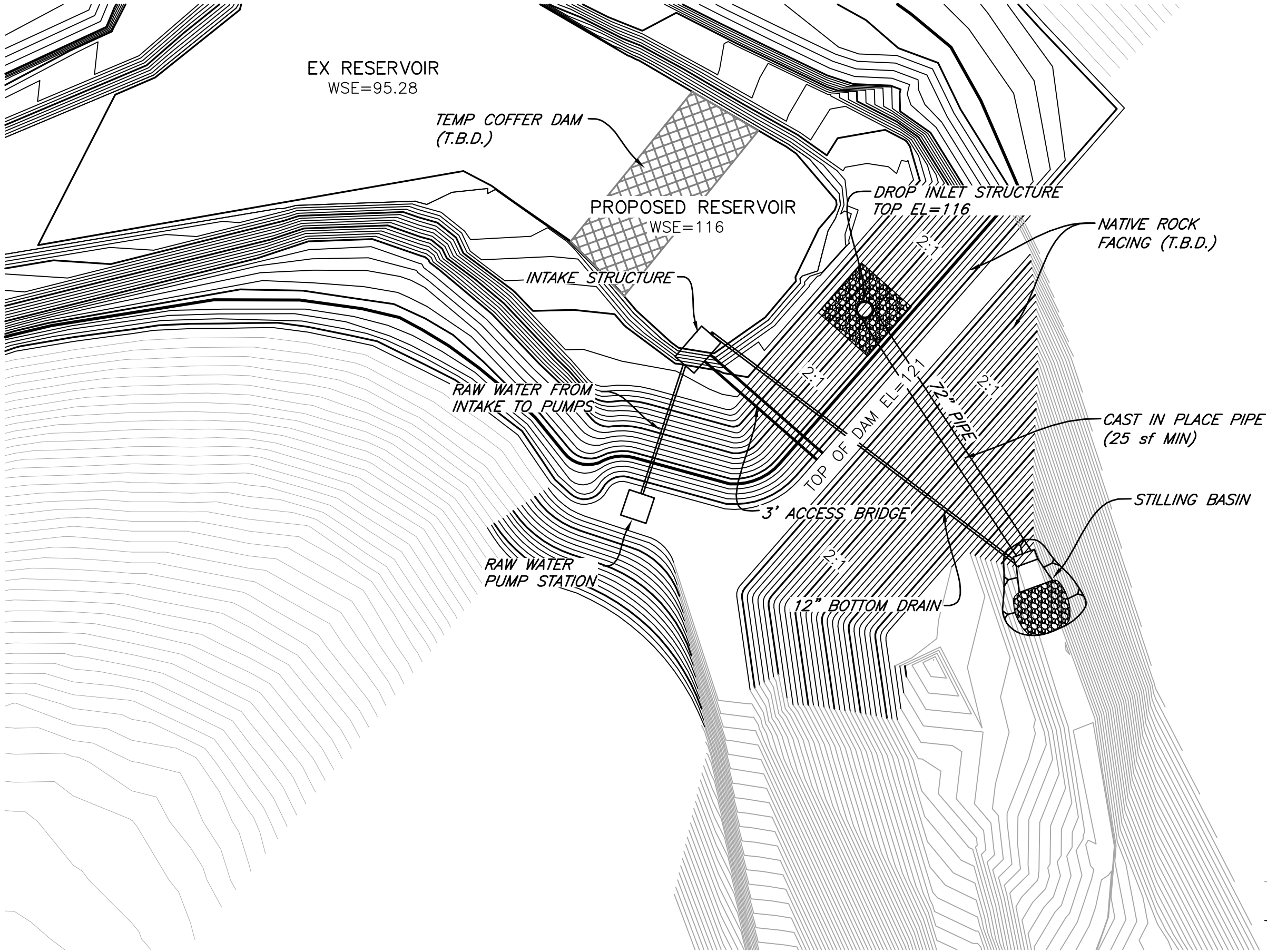

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LEGEND

EXISTING CONTOURS

1' INTERVAL STORAGE LINES

SHN Consulting Engineers & Geologists, Inc.	City of Port Orford Hubbard Creek Reservoir City of Port Orford, Oregon	Plan View of Proposed Earthen Dam SHN 608040.070
	April 2010 / 608040-Dam-Intake-Overflow_figs_6	Figure 6.2

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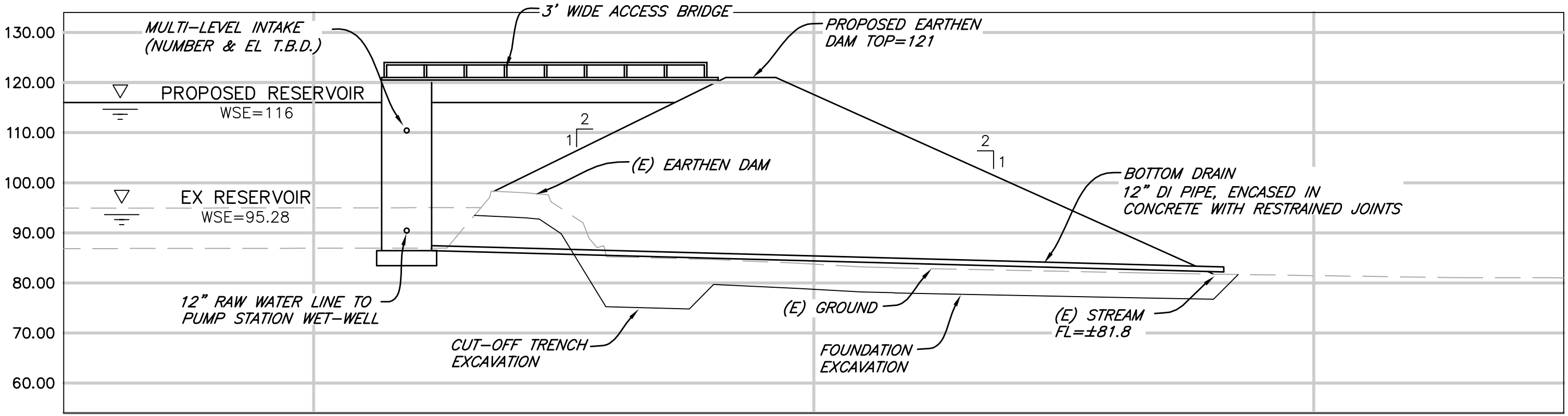

SHN Consulting Engineers & Geologists, Inc.	City of Port Orford Hubbard Creek Reservoir City of Port Orford, Oregon	Sectional View of Proposed Dam and Intake SHN 608040.070
	April — 608040-Dam-Intake-Overflow_figs_6	Figure 6.3

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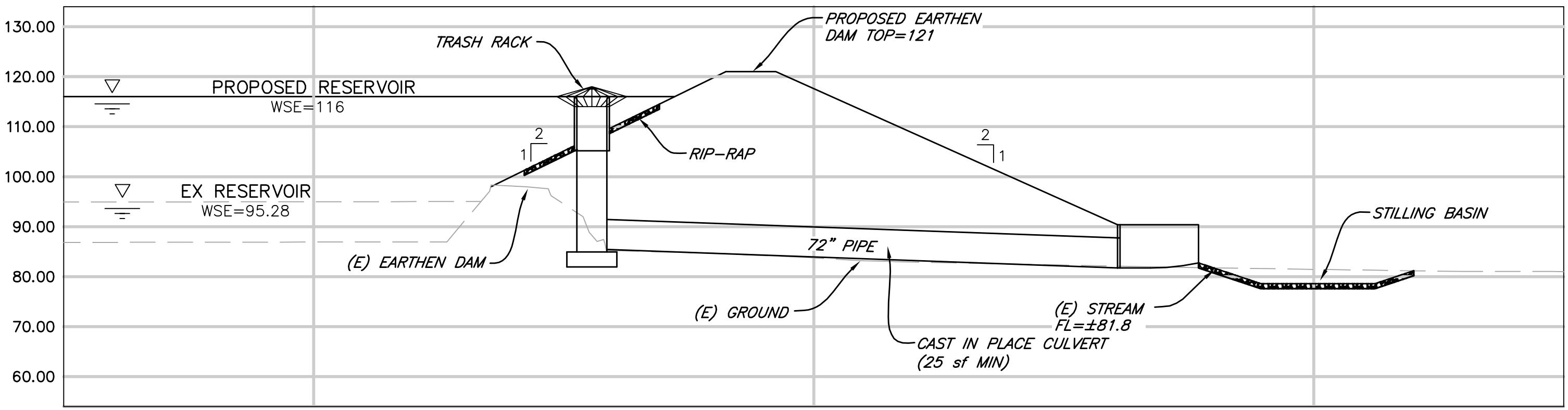

SHN Consulting Engineers & Geologists, Inc.	City of Port Orford Hubbard Creek Reservoir City of Port Orford, Oregon	Sectional View of Proposed Dam and Primary Spillway SHN 608040.070
	April 2010 / 608040-Dam-Intake-Overflow_figs_6	Figure 6.4

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with minor sandy siltstone strata. Natural slopes were typically mantled with 2-feet to over 8-feet of loose rocky colluvial soils. Moderately hard, slightly to moderately weathered sandstone would be anticipated at relatively shallow depths beneath the abutments and foundation of the proposed dam and in the proposed embankment borrow area. The bedrock materials in the abutments and beneath the embankment cut off trench are expected to be relatively "impervious and capable of supporting the embankment loads". On site borrow materials, weathered sandstone and colluvial soils consisting of a well graded mix of textures, are expected to be suitable for homogeneous fill embankment that incorporates internal filter and drainage structures.

The foundation of the dam will be extended through the upper alluvial and loose colluvial soil layers and keyed into the weathered sandstone comprising the walls and floors of the North Fork Hubbard Creek canyon. Site specific geologic investigations that include deep borings and permeability testing will need to be performed to determine the subsurface characteristics and geotechnical construction requirements at the site. Based on preliminary investigations, there appears to be satisfactory borrow sites for the embankment fill materials located, just northeast of the proposed dam expansion (See Figure 4.6). The borrow site is in close vicinity of the project which will be beneficial to overall economics of construction.

Literature research and surficial reconnaissance at and adjacent to the proposed reservoir site indicates that earthquake fault and slope failure hazards are minimal to negligible.

6.2.2 Spillway Design

The primary spillway is proposed to be a drop inlet. Alternatives evaluated included the following types of spillways: Chute, gated tower, labyrinth, and side channel. Other options should be investigated further, however, site constraints may restrict the feasibility of alternatives and for purposes of this report, a drop inlet is the preferred approach.

A preliminary hydraulic analysis of the drop inlet indicates that a 60" diameter pipe (minimum 25 square feet of flow area) would be required to pass the design flows. If approved by WRD, the drop inlet would serve as a spillway and also as an emergency overflow. If approved as an emergency structure, the inlet weir would also need to provide 40 feet of weir length. As proposed, the drop inlet would be able to convey approximately 700 CFS (500 year storm peak runoff), providing capacity for the emergency event. Depending upon final design considerations, the spillway will discharge in the vicinity of the existing creek channel on the downstream side of the new dam structure. The discharge area will include a stilling basin to address energy dissipation and potential scour.

6.2.3 Reservoir Drain

The new impoundment should be equipped with a drain. A minimum 12-inch diameter ductile iron pipe with restrained joints and encased in concrete will be used for a bottom drain of the reservoir. The upstream inlet will be located adjacent to the raw water inlet structure and controlled with a sluice gate. The inlet should be equipped with a vent to atmosphere to allow inlet control and free drainage of the pipe when the gate is closed. The downstream end of the drain should discharge to the spillway outlet structure which could also have a gate valve installed for double isolation.

6.2.4 Raw Water Intake Structure

The proposed raw water intake system will consist of an intake level control tower which will collect water from the reservoir and deliver it through a gravity supplied pipe to the raw water pump stations wet well. One of the elements associated with design of the intake structure is the ability to draw raw water from varying depths in the reservoir. Raw water supplies associated with other reservoirs on the Oregon Coast have experienced the occurrence of taste and odor problems along with high levels of manganese during certain times of the year. Those occurrences are typically associated with algae growth, solubility of manganese, and thermocline formation within the reservoir. Raw water with high manganese levels can usually be avoided by adjusting the depth in which the raw water is taken from, however, too high of an intake level could create taste problems as more algae is introduced into the supply. Alternatively, an adjustable intake structure was evaluated including retractable suction and adjustable floating devices. The recommended intake structure is mechanically the simplest to operate of all the alternatives considered. The intake structure would consist of a stationary tower with bridge access and manually actuated sluice gates to open intake ports to the desired lake level. Adjustment every 5 feet would provide the City with access to the full vertical stratum of the impoundment. The structure would also be configured to allow maintenance access to the inlet of the bottom drain.

6.2.5 Raw Water Pump Station

From the intake structure, raw water will be carried by a 12-inch diameter pipe to a pumping facility adjacent to the crest of the dam at the south west corner of the reservoir. The pumping equipment will consist of two 500 gpm vertical turbine pumps (duplex system). Raw water will be withdrawn from a wet well and discharged into the existing raw water transmission line. The new pumps will replace the existing 500 gpm pump and the existing 350 gpm pump. Each pump will be equipped with a hydraulically actuated well control valve to prevent surging and water hammer in the transmission main.

In addition to raw water pumping equipment, the facility will be equipped with a new SCADA system, turbidity monitoring equipment, flow metering equipment, video surveillance, prechlorination equipment, and back-up power supply. The SCADA system will communicate equipment status, emergency conditions, real time raw water turbidity, flow data, and allow improved monitoring with the City's existing surveillance system. Prechlorination equipment will allow the City to improve contact time at the water treatment facility, particularly during color events which seasonally impact raw water quality. The prechlorination equipment proposed would match the on-site generation equipment currently installed at the City's water treatment facility.

6.2.6 Fish Passage Structure

Considering the existing impoundment and local wildlife habitat, it is not anticipated that a fish passage structure will be installed with the new project. However, a final determination of the need for fish passage will occur through future review of the project with National Marine Fisheries and the Oregon Department of Fish and Wildlife.

6.2.7 Water Rights

Water rights were researched and evaluated by interpreting Oregon State Water Law and discussions with the local water master and the state dam safety coordinator for OWRD. Water rights issues associated with moving the intake of the reservoir are as follows:

- Transfer of the water right if the intake is moved more than 10 feet will be required (per water master).
- The priority of senior water right will not be affected by the transfer under Oregon Water Law 540.510(1).
- No injury to junior rights will exist unless the point of diversion is moved upstream or downstream of another right.
- The supplemental water right should be transferred as well, as the Water Resources Department may cancel the supplemental right if not transferred.

Considerations involved with applying for a supplemental water right for increase impoundment for the reservoir:

- Application for a supplemental water right will be required. The supplemental water right will be an addition to the existing right and will not take the place of the original right.
- The application will trigger a design review by the state dam safety coordinator to verify that the design will handle the applied volume of storage.
- A complete design review of the actual dam (full hard copy design drawings) will occur as a requirement of the application review or after approval of the new right.

If the Point of Diversion is proposed to be moved, an application will need to be submitted to the Water Resources Department (WRD). If the WRD issues a permit for the transfer, a Certified Water Rights Examiner will need to prepare a certification to be approved by the WRD

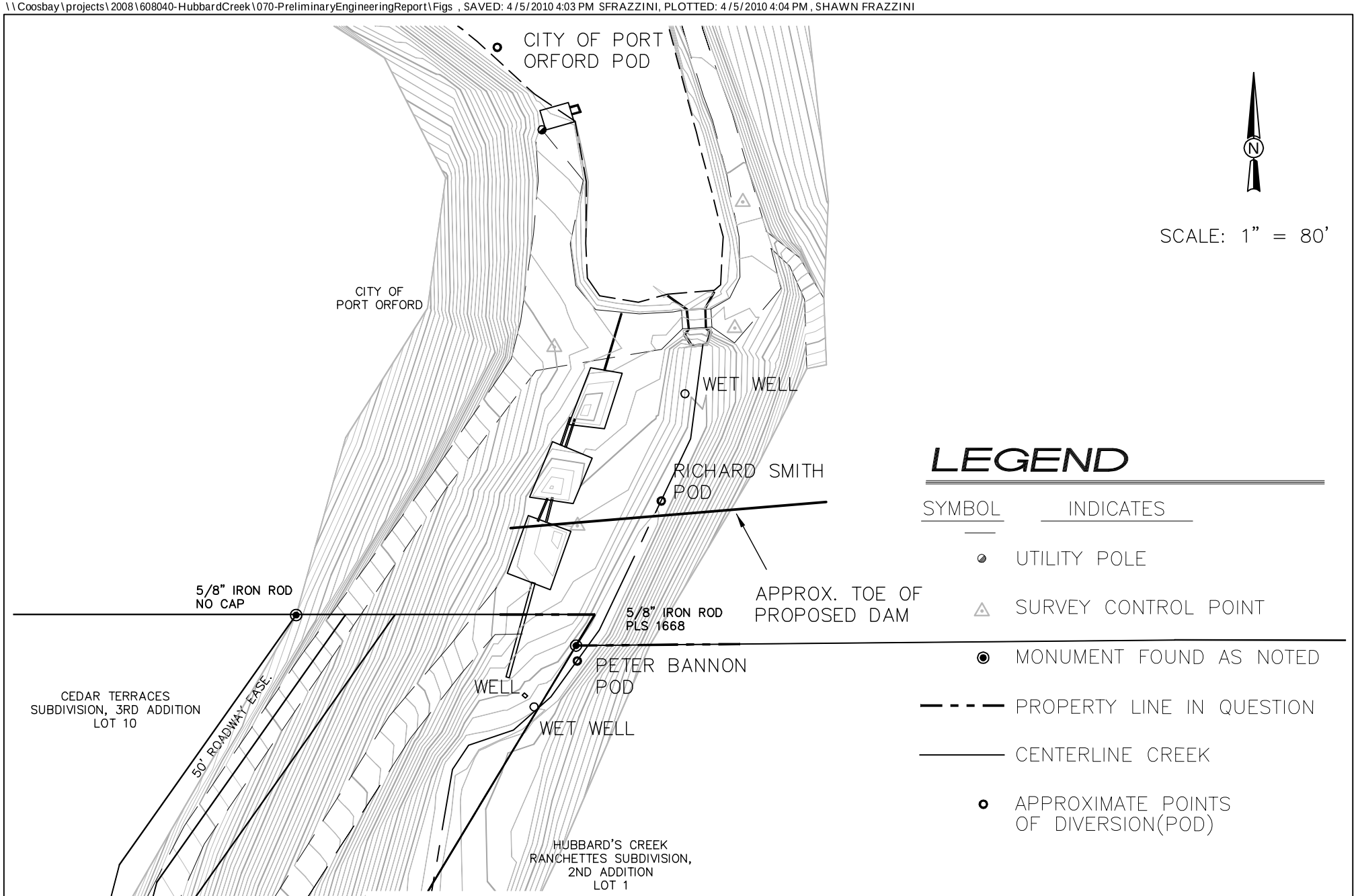
Two private, certified Points of Diversion (POD) exist along North Fork Hubbard Creek, as shown in Figure 6.5

The Richard Smith POD is approximately 70 feet north of the southerly boundary line of the City of Port Orford. This POD could potentially be impacted by the downstream embankment of the proposed dam.

The Peter Bannon POD is on private property just south of the southerly boundary line of the City of Port Orford.

6.2.8 Property Acquisition

As discussed in Section 6.2.7, the proposed dam site and inundation area for the expansion of the North Fork Hubbard Creek Impoundment may require property acquisitions or permanent easements, and relocation of neighboring water supply cisterns.



SHN Consulting Engineers & Geologists, Inc.	City of Port Orford Hubbard Creek Reservoir City of Port Orford, Oregon	Water Rights SHN 608040.070
	April 2010	608040-Water-Rights-Fig_6-5	Figure 6.5

The proposed lower reservoir is located on City of Port Orford property, Tax Lot 1300 in T. 32 S., R. 15 W., Section 33D and Tax Lot 100 in T. 33 S., R. 15 W., Section 4A. The inundation area of the most easterly tributary appears to encroach on adjacent Tax Lot 300 in T. 33 S., R. 15 W., Section 3B. An agreement to release a portion of the interest in that property should be negotiated before moving forward with the design phase of the project. Also, the area proposed for construction of the dam will potientially require that the Point of Diversion certified to Richard Smith be transferred to a new location. Potential impacts to property at points of diversion are shown in Figure 6.5.

7.0 Costs and Implementation

7.1 Basis for Cost Estimates

The cost estimates presented in this Plan will typically include four components: construction cost, engineering cost, contingency, and legal and administrative costs. Each of the cost components is discussed in this section. The estimates presented herein are preliminary and are based on the level and detail of planning presented in this Study. As projects proceed and as site-specific information becomes available, the estimates may require updating.

7.1.1 Construction Costs

The estimated construction costs in this Plan are based on actual construction bidding results from similar work, published cost guides, discussions with local contractors and other construction cost experience. Reference was made to the as-built drawings, and system maps of the existing facilities to determine construction quantities, elevations of the reservoirs and major components, and locations of distribution lines. Where required, estimates will be based on preliminary layouts of the proposed improvements. The cost estimates provided within this Plan assume that all projects are constructed under public contract.

Future changes in the cost of labor, equipment, and materials may justify comparable changes in the cost estimates presented herein. For this reason, common engineering practices usually tie the cost estimates to a particular index that varies in proportion to long-term changes in the national economy. The Engineering News Record (ENR) construction cost index is most commonly used.

It is anticipated that construction of any necessary projects will start by the summer of 2011. Cost estimates presented in this Plan for construction performed in later years should be projected with an increase of three percent per year or future yearly ENR indices can be used to calculate the cost of projects for their construction year based on the annual growth in the ENR index.

It is also recommended that in the event other public works projects are being performed in the same location, (i.e., sewer, street, storm, etc.), planning priority be given to combining the water projects with the projects at hand. The City can save money by eliminating repetitive mobilization, demolition, and road patching in the same locations.

7.1.2 Contingencies

A contingency factor equal to approximately 10 percent of the estimated construction cost has been added. In recognition that the cost estimates presented are based on conceptual planning, allowances must be made for variations in final quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigation and studies, and other difficulties which cannot be foreseen at this time but may tend to increase final costs.

7.1.3 Engineering

The cost of engineering services for major projects typically includes special investigations, a predesign report, surveying, foundation exploration, preparation of contract drawings and

specifications, bidding services, construction management, inspection, construction staking, start-up services, and the preparation of operation and maintenance manuals. Depending on the size and type of project, engineering costs may range from 15 to 25 percent of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small, complicated projects. The engineering costs for waterline and reservoir design and construction projects will average 17 percent and 20 percent of the construction cost, respectively.

Additional engineering services may be required for specialized project elements. This could include geotechnical evaluations, structural evaluations, and other specialized consulting activities. Where possible, these projects have been included in the estimates.

7.1.4 Legal and Administrative

An allowance of four percent (4%) of construction cost has been added for legal and administrative services. This allowance is intended to include internal project planning and budgeting, grant administration, liaison, and interest on term loan financing, legal services, review fees, legal advertising, and other related expenses associated with the project.

7.1.5 Land Acquisition

Some projects may require acquisition of additional right-of-way or property for construction of specific improvement. The need and cost for such expenditures is difficult to predict and must be reviewed as a project is developed. It should be noted that the cost of land is subjective and depends on the seller, current land use practices, the size of the plot to be acquired, options available to the City, and other issues outside the common practice of engineering. Some land acquisition may be required for relocation of private property Points of Diversion and for private property east of the existing dam which will be inundated by the new reservoir.

7.2 Distribution System Improvements Costs

The opinion of probable costs for the water distribution system improvements are summarized in Table 7.1 below. All phases of the project are presented in Figure 3 except where the primary loop is identified as Idaho and Jackson Street projects. Appendix C contains detailed cost breakdowns of each of the phases.

Table 7.1 North Fork Hubbard Creek Impoundment Project Opinion of Probable Costs Water Distribution System Improvements					
Breakdown by Project Phase					
Idaho Street				$	602,105
Jackson Street				$	911,291
Phase 1				$	1,330,718
Phase 2				$	1,217,270
Phase 3				$	1,766,519
Phase 4				$	1,781,694
Phase 5				$	1,932,835
Phase 6				$	1,064,308
Phase 7				$	696,784
Deady Street Pump Station				$	173,575
Cost Guard Hill Pump Station				$	169,645
Total Project Cost				**$**	**11,753,142**
Breakdown by Quantity & Unit price					
	Unit	**Quantity**	**Unit Price**		**Cost**
Mobilization	LS	All	varies	$	958,000
4" PVC Waterline	LF	1,557	$ 30	$	46,710
8" PVC Waterline	LF	49,836	$ 50	$	2,491,800
10" PVC Waterline	LF	23,874	$ 65	$	1,551,810
Service Laterals (short)	LF	4,890	$ 15	$	73,350
Service Laterals (long)	LF	22,820	$ 30	$	684,600
Connection to Existing	EA	75	$ 2,500	$	187,500
Fire Hydrants	EA	51	$ 3,000	$	153,000
AC Pavement	LF	52,686.9	$ 25	$	1,317,173
Gravel Surfacing	LF	22,580.1	$ 15	$	338,702
Highway 101 Crossings	EA	13	$ 25,000	$	325,000
Misc Fittings	LS	All	varies	$	501,000
Deady Street Pump Station				$	173,575
Coast Guard Hill Pump Station				$	169,645
Construction Cost				**$**	**8,897,186**
Contingency (10%)				$	897,186
Engineering (17%)				$	1,525,217
Administration (4%)				$	358,875
Total Project Cost				**$**	**11,753,142**

7.3 North Fork Hubbard Creek Dam/Impoundment Expansion Costs

The opinion of probable costs for the Dam/Impoundment expansion project is presented in Table 7.2 on the following page.

Table 7.2 City of Port Orford Water System CIP North Fork Hubbard Cr. Dam/Impoundment Expansion Project				
	Unit	**Quantity**	**Unit Price**	**Cost**
Mobilization	LS	100%	$ 306,179	$ 306,179
Clearing and Grubbing	AC	0.5	$ 40,000	$ 20,000
Haul Route Construction	LF	400	$ 120	$ 48,000
Foundation Excavation	CY	4,500	$ 10.40	$ 46,800
Embankment	CY	170,000	$ 4.85	$ 824,500
72" Primary Spill Way	LF	130	$ 535	$ 69,550
72" Inlet Structure	LS	100%	$ 85,000	$ 85,000
Outlet Structure/Stilling Basin	LS	100%	$ 145,000	$ 145,000
12" CIP Underdrain	LF	150	$ 265	$ 39,750
Raw Water Inlet Structure	LS	100%	$ 175,000	$ 175,000
Inlet Strucure Access Bridge	LS	100%	$ 55,000	$ 55,000
12" Raw Water Line	LF	80	$ 140	$ 11,200
Raw Water Pump Station	LS	100%	$ 425,000	$ 425,000
Toe Filter/Drains	CY	1,611	$ 44.50	$ 71,690
Cofferdam/Temporary Source	LS	100%	$ 250,000	$ 250,000
Misc Fittings/Appurtenances	LS	100%	$ 225,000	$ 225,000
Construction				$ 2,857,668
Contingency (10%)				$ 285,767
Engineering (20%)				$ 571,534
Administration (4%)				$ 114,307
Total Cost				**$ 3,829,275**

7.4 Operation and Maintenance Costs

It is anticipated that operation and maintenance (O&M) costs currently experienced by the City, will be reduced as a direct result of the proposed projects. Current annual O&M costs are presented in Section 3 of this study. Personal services costs should remain the same since personnel changes are not required. However, annual electrical costs should be reduced by about 1/3, because of the reduction in system losses (less finished water will have to be produced through the treatment plant and pumped throughout the system). The electrical savings also reflect cost savings associated with the energy efficient pumping facilities being proposed. Additional savings should be realized due to the decrease in maintenance resulting from a replacement of the older, worn out distribution system with new.

Operating and maintaining the reservoir and raw water pump station should be similar in costs as the current system. The City should not have expenses related to dredging but they would have a little more land area for landscape maintenance (trimming, mowing, etc.).

The City's current annual budget for O&M related to their water fund is $303,406. Upon reviewing line item expenditure in the O&M budget, savings, in today's dollars, are estimated at approximately $16,500.

7.4.1 Short -lived Assets and Reserve

Additional reserve amounts are needed to provide for timely replacement of short-lived assets. The following is an estimated schedule of short-lived asset annual expenses which would be associated with the upgraded system and reservoir:

<u>Impoundment and raw water intake</u>	
Raw water pump replacement/overhaul (2- raw water pumps)	$1,000
Flow Meter replacement/overhaul	$1,500
<u>Water Treatment</u>	
Pump replacement/overhaul (2- filter pumps, 2- service pumps)	$2,000
Flow Meter replacement/overhaul	$3,000
Valve replacement	$ 500
VFD replacement	$ 750
Overhaul MCC	$1,000
Chlorine generation cell replacement	$ 800
<u>Storage</u>	
Water level gauge replacement	$ 100
Valve replacement	$ 1,000
Tank Clean/Paint	$ 5,000
<u>Distribution System</u>	
Pump replacement/overhaul (4- pump stations, 2 pumps ea.)	$2,000
Flow meter replacement/overhaul	$1,500
<u>General equipment replacement</u>	
Vehicle Replacement	$20,000
Replace/overhaul pump and vacuum equipment on Vac-truck	$ 1,000
Lab equipment	$ 500
Total annual short-lived asset reserve	$41,650

Short-lived assets considered include those items not typically included in ongoing O&M and does not include long-lived assets such as pump station or treatment facility replacement that should be funded with long-term financing.

7.5 Schedule

The projected schedule for implementation of the proposed projects is:

Distribution System Improvements

TASK	DATE
Authorization and funding – Design	May 2010
Preparation of plans/Specifications	June 2010-August 2010
Authorization and funding secured for construction	September 2010
Advertise for bids	November 2010
Receive Bids	December 2010
Award Contract	February 2010
Begin Construction first phase of improvements	May 2011
Completion of final phase of improvements	September 2012
Project Close-out	November 2012

Dam/Impoundment Expansion

TASK	DATE
Authorization and funding – Design/Permitting	May 2010
Preparation of plans/Specifications	June 2010 – August 2011
Project Permitting (WRD, DSL, COE, DEQ, NMF, ODF&W)	July 2010 – July 2011
Authorization and funding secured for construction	September 2011
Advertise for bids	November 2011
Receive Bids	January 2012
Award Contract	March 2012
Begin Construction	May 2012
Completion of improvements	September 2014
Project Close-out	December 2014

Appendix A

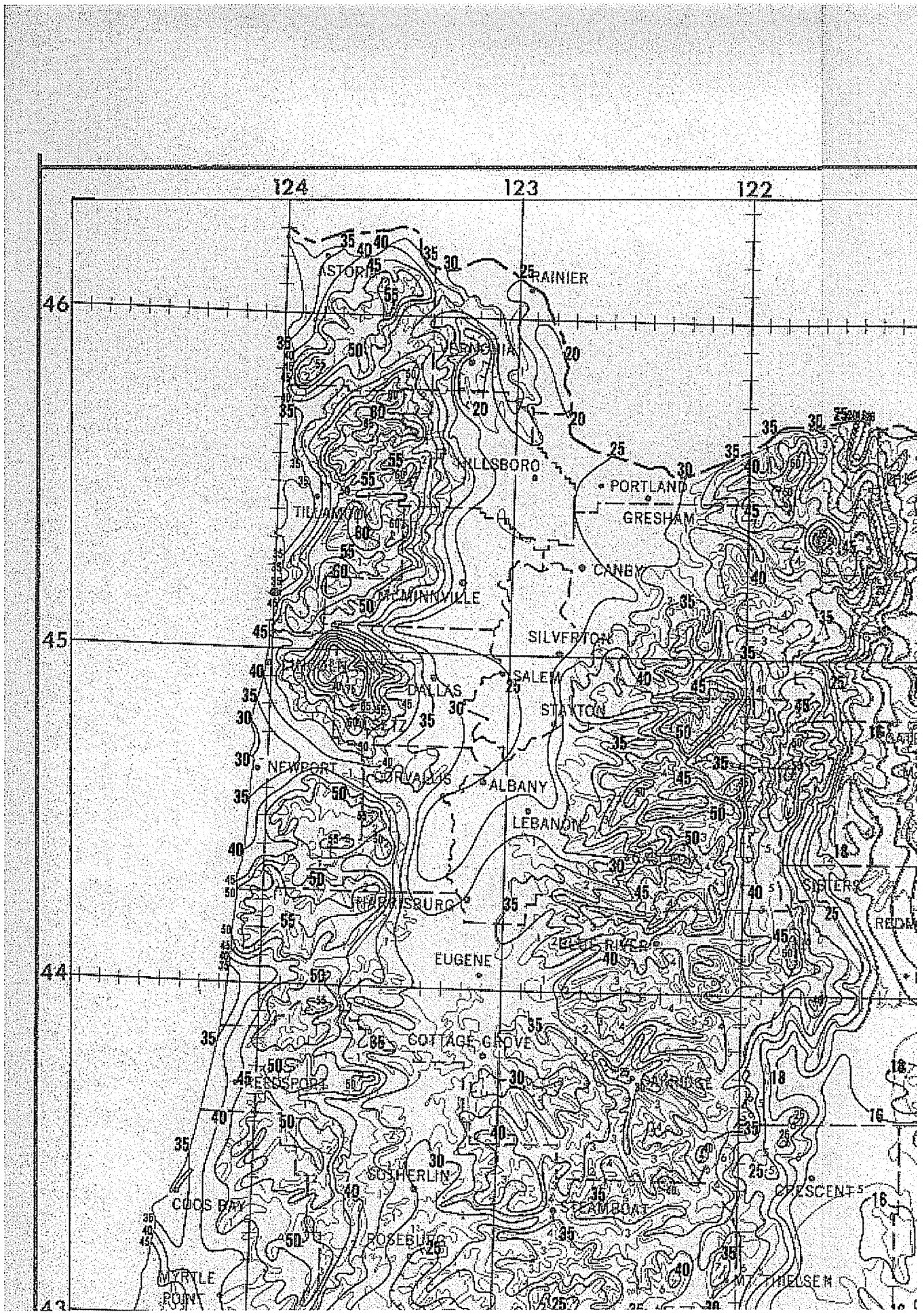
Summary Capital Improvements List

Summary CIP List					
Priority	Project	Estimated Cost		Capacity	
				Exist	Imp.
				gpm	gpm
	Pump Stations				
5 year	Coast Guard Hill PS	$ 173,000		500	1,000
Immediate	Deady Hill PS & WL Extension	$ 258,000		50	1,000
	Subtotal	$ 431,000			
	Distribution System				
Immediate	Metering Plan	$ 196,000		N/A	N/A
5 year	Driftwood School	$ 266,000		2,000	3,500
10 year	Commercial District (Option1&2)	$ 890,000	$ 1,060,000	850	2,000
15 year	Coast Guard Hill	$ 409,000		500	1,000
5 year	Harbor Drive	$ 305,000		300	1,700
20 year	Lakeshore Drive	$ 348,000		324	1,000
20 year	25th	$ 227,000		620	1,300
15 year	Mather Hensley Hill	$ 251,000		440	1,800
Immediate	Hensley Hill Reservoir Site	$ 100,000		440	1,800
15 year	Hensley Hill Reservoir	$ 500,000		440	1,800
20 year	Paradise Pt & Manzanita Dr.	$ 167,000		430	1,300
	Subtotal	$3,659,000	$3,087,000		
	Treatment Plant				
Immediate	Improvements to Existing	$ 235,000		325	450
10 Year	Upgrade with Absorption Clarifier	$ 879,000		450	550
	Subtotal	$1,114,000			
	Water Source				
Immediate	Deady Street Reservoir conversion	$ 63,000		1MG	1.2MG
5 year	Hubbard Creek Impoundment (Existing)	$1,273,000		1.2 MG	8.4 MG
20 year	Hubbard Creek Impoundment (New)	$2,350,000		8.4 MG	15.4 MG
Future	Garrison Lake Intake with Treatment	$1,660,000		1.25 cfs	2.25 cfs
Future	Garrison Lake Treatment w/o Intake	$1,700,000		1.25 cfs	2.25 cfs
	Subtotal Existing Hubbard Creek & Deady St.	$1,336,000			
Phasing of Projects					
2	Immediate Needs Projects 2005/2006				$ 852,000
5	Priority 1 Distribution System				$ 744,000
5	5 Year Source Upgrade				$ 1,273,000
10	Treatment Upgrade 2015				$ 879,000
10	Priority 2 Distribution System				$ 890,000
15	Priority 3 Distribution System				$ 1,160,000
20	Priority 4 Distribution System				$ 742,000
20	20 Year Source/Treatment Improvements				$ 2,350,000
	Total 20-Yr Need				**$ 8,890,000**
Outside City Limits Improvements					
NA	**Total Need as Developed**				**$8,428,000**

Appendix B

Stream Flow Analyses

Precipitation Data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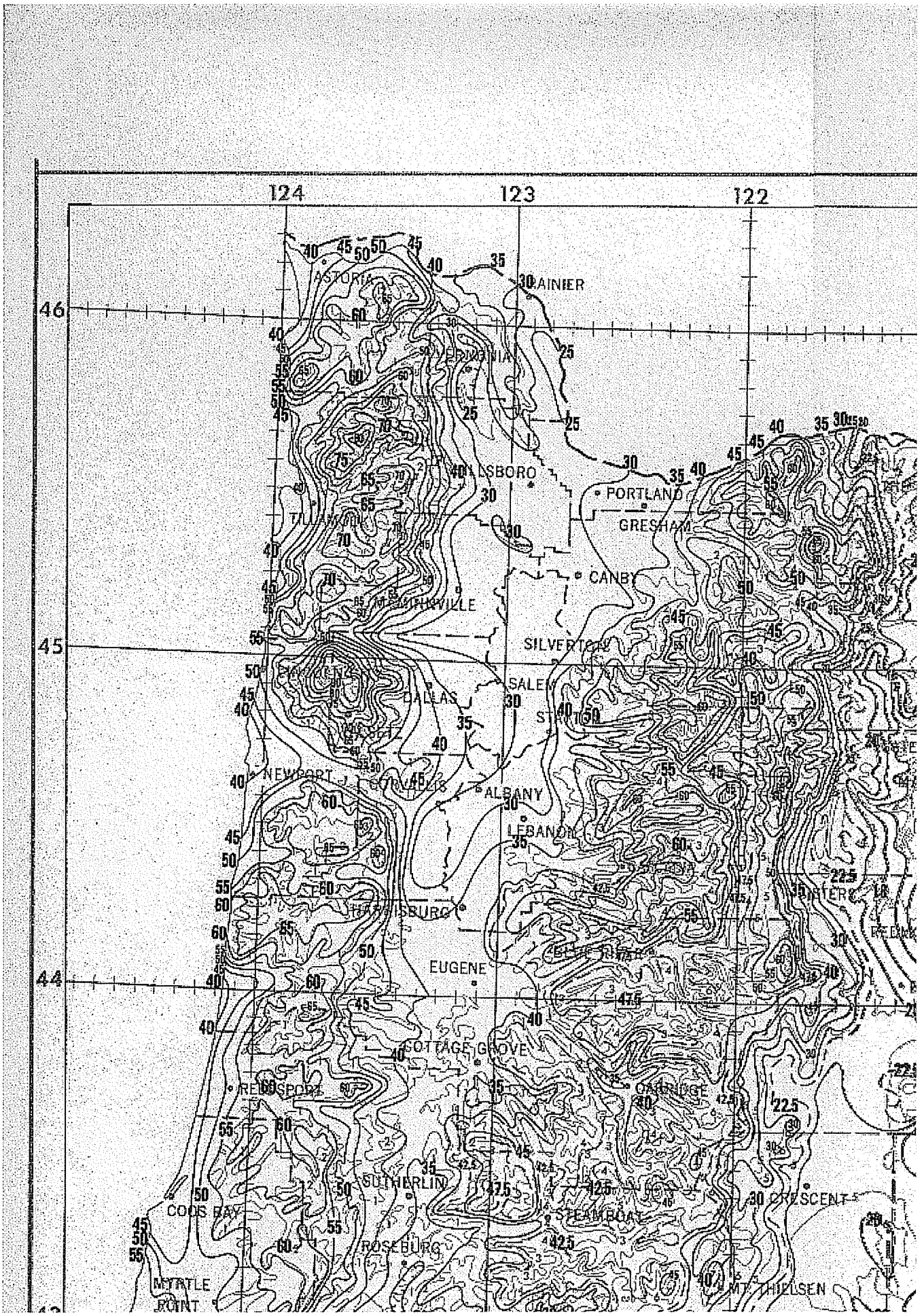
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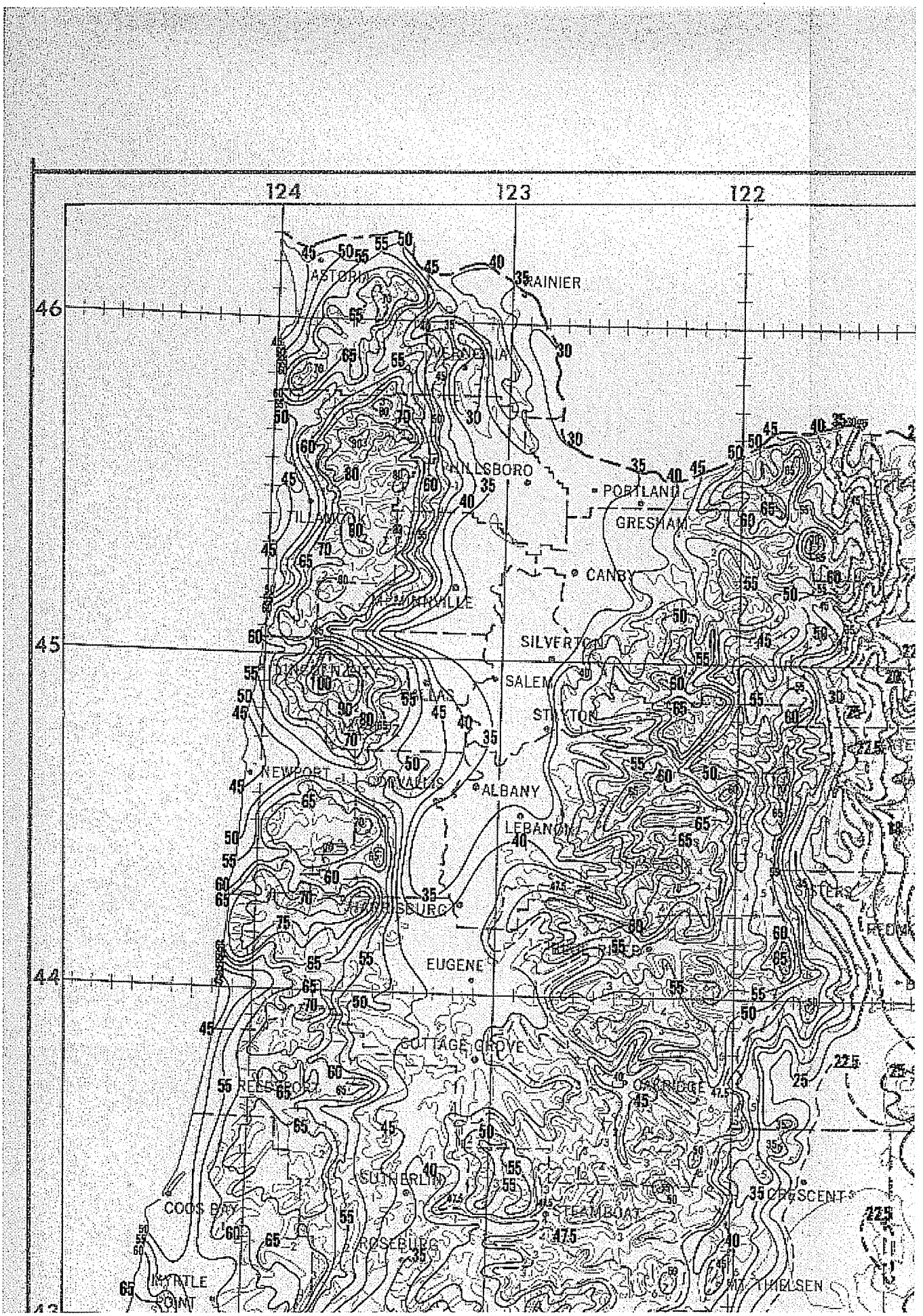
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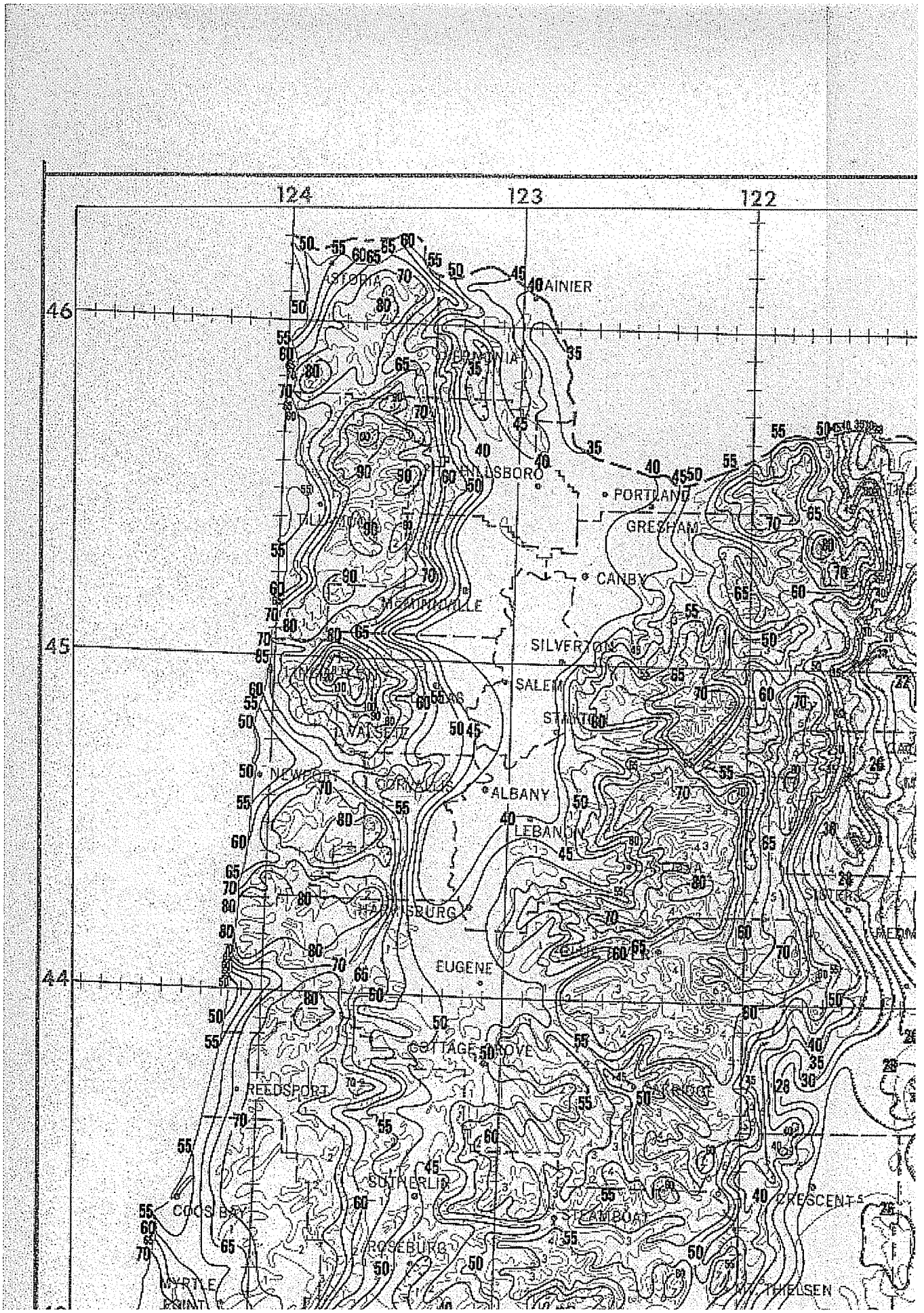
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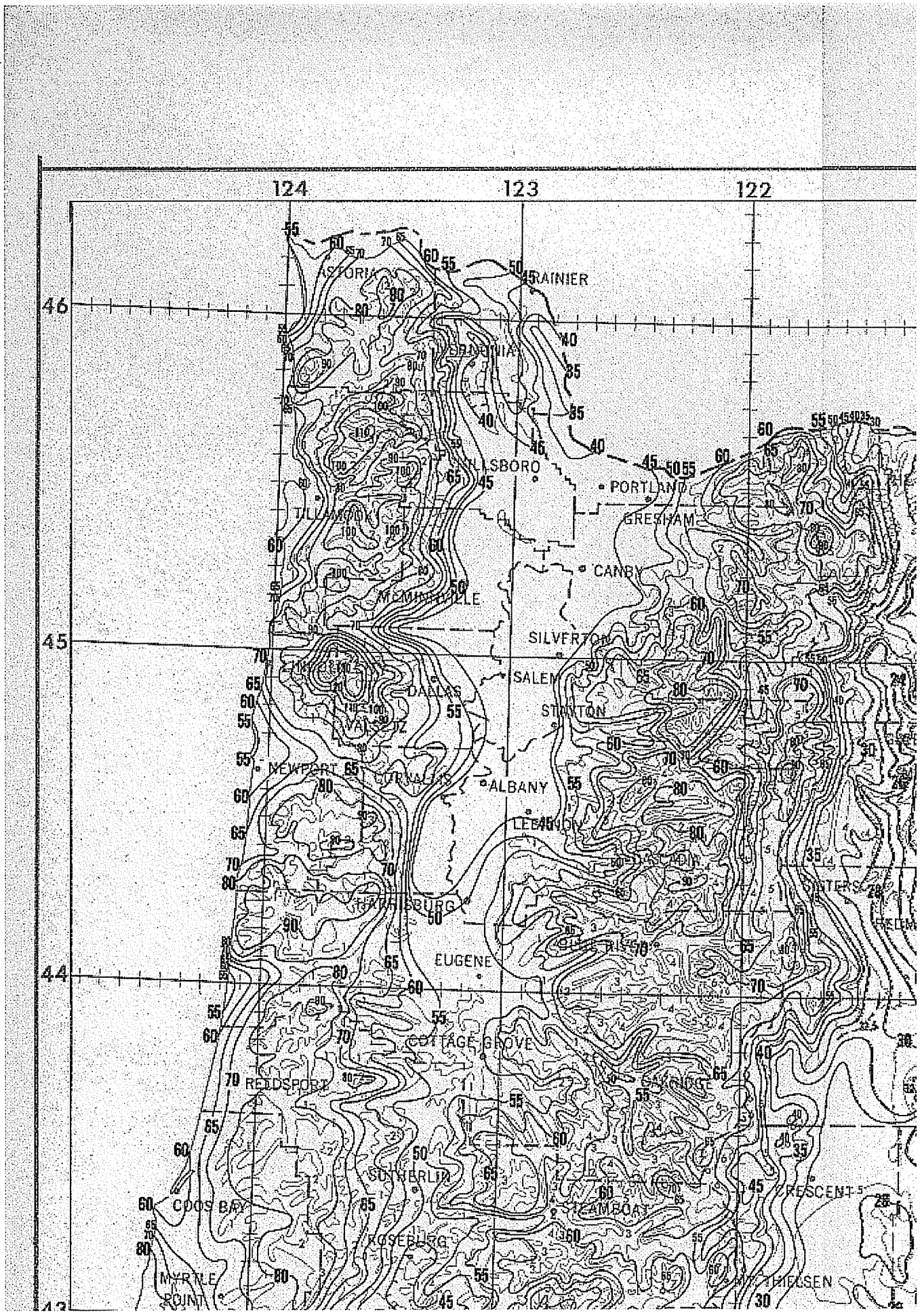
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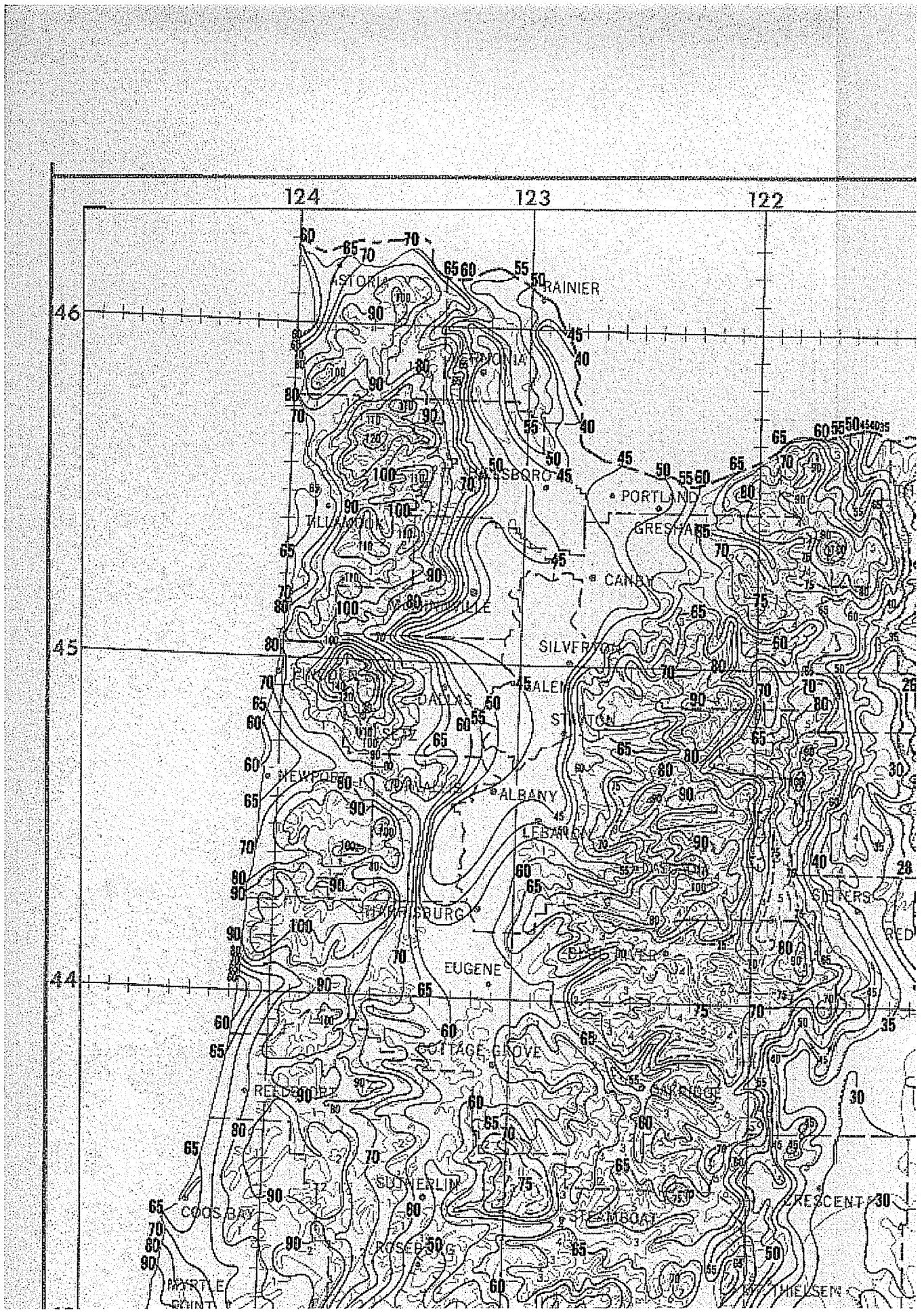
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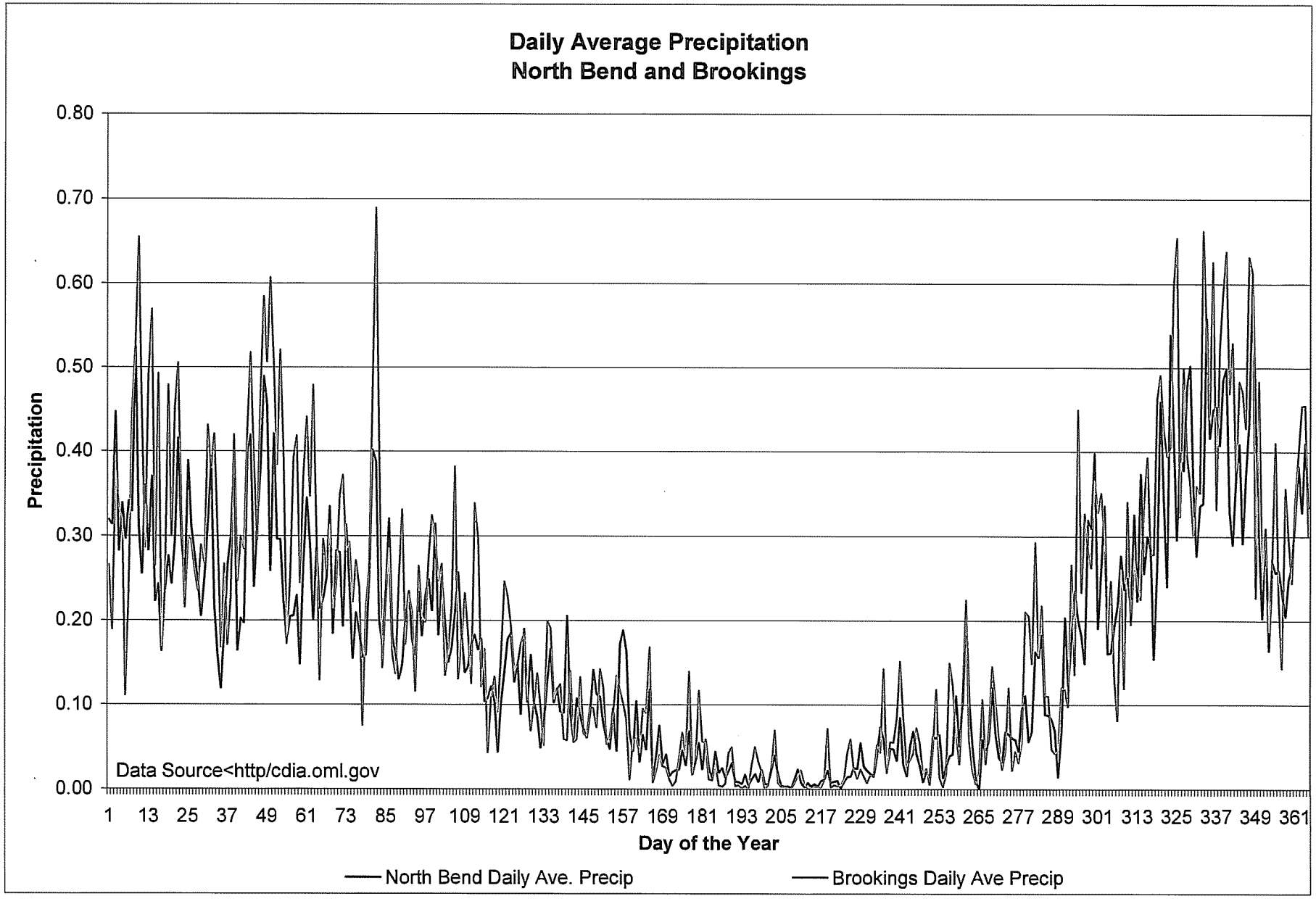
Daily Average Precipitation
North Bend and Brookings
Precipitation
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0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
Data Source<http/cdia.oml.gov
1 13 25 37 49 61 73 85 97 109 121 133 145 157 169 181 193 205 217 229 241 253 265 277 289 301 313 325 337 349 361
Day of the Year
North Bend Daily Ave. Precip
Brookings Daily Ave Precip

CLIMATOGRAPHY OF THE UNITED STATES NO. 20

BANDON, OR

PERIOD: 1951-80
ELEVATION: 80 FT

CLIMATOLOGICAL SUMMARY

	TEMPERATURE (F)															PRECIPITATION TOTALS (INCHES)											
	MEANS			EXTREMES						MEAN NUMBER OF DAYS				DEGREE DAYS		*	*					SNOW			MEAN NUMBER OF DAYS		
										MAX		MIN															
	* DAILY MAXIMUM	* DAILY MINIMUM	* MONTHLY	RECORD HIGHEST	YEAR	DAY	RECORD LOWEST	YEAR	DAY	90 AND ABOVE	32 AND BELOW	32 AND BELOW	0 AND BELOW	* HEATING BASE 65	* COOLING BASE 65	MEAN	GREATEST MONTHLY	YEAR	GREATEST DAILY	YEAR	DAY	MEAN	MAXIMUM MONTHLY	YEAR	.10 OR MORE	.50 OR MORE	1.00 OR MORE
JAN	52.5	38.2	45.4	75+	76	30	16+	80	29	0	0	7	0	608	0	10.63	20.14	56	4.00	53	18	.8	18.0	69	15	7	3
FEB	54.5	39.6	47.1	79+	68	27	24+	79	2	0	0	4	0	501	0	7.47	15.96	61	4.99	61	10	.0	.0		13	5	2
MAR	54.4	39.2	46.9	78+	69	24	26+	74	8	0	0	4	0	561	0	7.47	12.48	61	3.87	72	02	.0	.0		14	5	1
APR	56.3	40.6	48.5	84+	51	10	29+	75	20	0	0	2	0	495	0	4.42	11.35	78	3.30	71	09	.0	.0		9	3	1
MAY	59.6	44.0	51.8	90+	70	31	30+	72	24	0	0	0	0	409	0	2.80	7.31	53	2.56	63	06	.0	.0		6	2	0
JUN	62.9	48.0	55.4	85+	70	1	35	76	4	0	0	0	0	288	0	1.24	3.19	80	1.80	71	25	.0	.0		3	0	0
JUL	64.6	50.0	57.3	85	78	21	38+	73	1	0	0	0	0	239	0	.35	1.21	74	.99	74	08	.0	.0		1	0	0
AUG	65.8	50.0	57.9	90	68	30	35	51	23	0	0	0	0	223	0	.90	4.39	68	3.40	77	24	.0	.0		2	0	0
SEP	65.9	48.0	56.9	90+	75	4	32+	72	27	0	0	0	0	246	0	1.70	4.14	77	1.80	57	26	.0	.0		3	1	0
OCT	62.6	44.8	53.7	91+	78	13	29+	72	30	0	0	1	0	350	0	4.42	12.05	56	3.03	55	09	.0	.0		7	3	1
NOV	57.3	41.9	49.6	79	66	01	21+	78	13	0	0	3	0	462	0	8.51	23.35	73	4.45	61	23	.0	.0		12	6	2
DEC	53.6	39.5	46.6	77+	80	15	12+	72	8	0	0	5	0	570	0	10.35	25.56	55	4.17	80	03	.1	2.0	72	15	7	3
YEAR	59.2	43.7	51.4	91	OCT 78	13	12	DEC 72	8	0	0	26	0	4952	0	60.26	25.56	DEC 55	4.99	FEB 61	10	.9	18.0	JAN 69	100	39	13

*FROM 1951-80 NORMALS

ESTIMATED VALUE BASED ON DATA FROM SURROUNDING STATIONS

+ ALSO ON EARLIER DATES.

DEGREE DAYS TO SELECTED BASE TEMPERATURES (F)

BASE	HEATING DEGREE DAYS												
BELOW	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
65	608	501	561	495	409	288	239	223	246	350	462	570	4952
60	453	361	406	345	254	145	98	91	116	197	312	415	3193
57	360	283	313	255	164	76	43	41	63	113	227	322	2260
55	302	232	254	197	108	44	18	18	36	69	173	265	1716
50	165	124	118	73	19	0	0	0	5	8	70	134	716

BASE	COOLING DEGREE DAYS												
ABOVE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
55	0	11	0	0	9	56	89	108	93	28	11	0	405
57	0	6	0	0	0	28	52	69	60	10	0	0	225
60	0	0	0	0	0	7	15	26	23	0	0	0	71
65	0	0	0	0	0	0	0	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0	0	0	0	0	0

DERIVED FROM THE 1951-80 MONTHLY NORMALS

PROBABILITY THAT THE MONTHLY PRECIPITATION WILL BE EQUAL TO OR LESS THAN THE INDICATED PRECIPITATION AMOUNT

MONTHLY PRECIPITATION (INCHES)

PROBABILITY LEVELS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
.05	3.52	2.96	3.01	.92	.52	.15	.00	.00	.11	.76	2.28	3.44
.10	4.53	3.65	3.69	1.32	.77	.25	.00	.00	.27	1.14	3.08	4.43
.20	6.01	4.63	4.67	1.97	1.18	.43	.04	.03	.54	1.79	4.31	5.86
.30	7.26	5.44	5.47	2.56	1.56	.60	.09	.11	.79	2.39	5.39	7.08
.40	8.47	6.20	6.22	3.16	1.95	.78	.15	.24	1.05	3.01	6.45	8.26
.50	9.72	6.98	6.99	3.79	2.36	.98	.22	.42	1.34	3.68	7.57	9.47
.60	11.09	7.82	7.82	4.51	2.84	1.22	.31	.66	1.68	4.45	8.80	10.79
.70	12.69	8.79	8.78	5.37	3.41	1.51	.42	1.01	2.10	5.39	10.27	12.34
.80	14.74	10.03	10.00	6.51	4.17	1.91	.58	1.53	2.66	6.63	12.18	14.33
.90	17.92	11.92	11.86	8.32	5.39	2.56	.85	2.47	3.58	8.65	15.17	17.41
.95	20.86	13.64	13.55	10.05	6.56	3.19	1.13	3.45	4.48	10.57	17.97	20.25

THESE VALUES WERE DETERMINED FROM THE INCOMPLETE GAMMA DISTRIBUTION.

CLIMATOGRAPHY OF THE UNITED STATES NO. 20

BROOKINGS, OR

CLIMATOLOGICAL SUMMARY

PERIOD: 1951-80
ELEVATION: 80 FT

	TEMPERATURE (F)															PRECIPITATION TOTALS (INCHES)											
	MEANS			EXTREMES						MEAN NUMBER OF DAYS				DEGREE DAYS		*	*					SNOW			MEAN NUMBER OF DAYS		
										MAX		MIN															
	* DAILY MAXIMUM	* DAILY MINIMUM	* MONTHLY	RECORD HIGHEST	YEAR	DAY	RECORD LOWEST	YEAR	DAY	90 AND ABOVE	32 AND BELOW	32 AND BELOW	0 AND BELOW	* HEATING BASE 65	* COOLING BASE 65	MEAN	GREATEST MONTHLY	YEAR	GREATEST DAILY	YEAR	DAY	MEAN	MAXIMUM MONTHLY	YEAR	.10 OR MORE	.50 OR MORE	1.00 OR MORE
JAN	53.8	40.5	47.2	72+	68	22	21+	62	21	0	0	3	0	552	0	13.06	24.62	53	6.19	53	17	.2	6.0	72	15	9	4
FEB	55.7	41.5	48.6	78+	67	6	26+	76	5	0	0	1	0	459	0	9.71	24.76	58	4.50	58	24	.0	.0		13	7	3
MAR	56.2	41.1	48.7	80+	69	24	30+	74	8	0	0	1	0	505	0	9.42	17.91	61	5.00	72	02	.1	2.0	76	14	7	3
APR	58.8	42.2	50.5	92+	51	10	31+	76	1	0	0	0	0	435	0	5.25	17.43	63	3.05	65	18	.0	.0		9	4	1
MAY	62.5	45.6	54.1	94+	70	31	32+	65	6	0	0	0	0	338	0	3.77	14.15	63	5.97	63	06	.0	.0		6	3	1
JUN	65.8	48.8	57.3	101	77	28	39+	62	3	0	0	0	0	231	0	1.37	4.73	71	2.92	71	25	.0	.0		3	1	0
JUL	66.4	50.6	58.5	96	73	26	39+	63	13	0	0	0	0	202	0	.46	2.62	64	1.08	76	08	.0	.0		1	0	0
AUG	66.8	51.2	59.0	96+	68	29	42+	64	31	0	0	0	0	186	0	1.22	6.97	68	2.81	78	24	.0	.0		2	1	0
SEP	68.1	50.8	59.5	103	73	09	39+	65	21	1	0	0	0	176	11	2.31	7.38	77	3.27	51	30	.0	.0		3	1	1
OCT	64.1	47.7	55.9	96+	58	1	34+	71	28	0	0	0	0	282	0	5.72	13.84	56	3.75	55	09	.0	.0		8	5	2
NOV	58.5	44.4	51.5	81	69	11	29+	77	19	0	0	1	0	405	0	11.36	26.01	73	5.53	53	22	.0	.0		13	7	4
DEC	54.9	41.4	48.2	79+	80	15	18+	72	8	0	0	2	0	521	0	12.57	23.65	55	6.88	54	30	.0	1.0	72	14	9	4
YEAR	61.0	45.5	53.3	103	SEP 73	09	18	DEC 72	8	1	0	8	0	4292	11	76.22	26.01	NOV 73	6.88	DEC 54	30	.3	6.0	JAN 72	101	54	23

*FROM 1951-80 NORMALS

ESTIMATED VALUE BASED ON DATA FROM SURROUNDING STATIONS

+ ALSO ON EARLIER DATES.

DEGREE DAYS TO SELECTED BASE TEMPERATURES (F)

BASE	HEATING DEGREE DAYS												
BELOW	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
65	552	459	505	435	338	231	202	186	176	282	405	521	4292
60	397	319	350	285	185	94	64	58	73	136	255	366	2582
57	304	240	257	200	102	39	18	16	32	69	170	276	1723
55	246	189	199	147	61	16	0	0	16	37	117	217	1245
50	115	87	78	53	7	0	0	0	0	0	30	96	466

BASE	COOLING DEGREE DAYS												
ABOVE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
55	0	10	0	12	33	85	113	129	151	65	12	6	616
57	0	0	0	5	13	48	64	78	107	35	0	0	350
60	0	0	0	0	0	13	18	27	58	8	0	0	124
65	0	0	0	0	0	0	0	0	11	0	0	0	11
70	0	0	0	0	0	0	0	0	0	0	0	0	0

DERIVED FROM THE 1951-80 MONTHLY NORMALS

PROBABILITY THAT THE MONTHLY PRECIPITATION WILL BE EQUAL TO OR LESS THAN THE INDICATED PRECIPITATION AMOUNT

MONTHLY PRECIPITATION (INCHES)

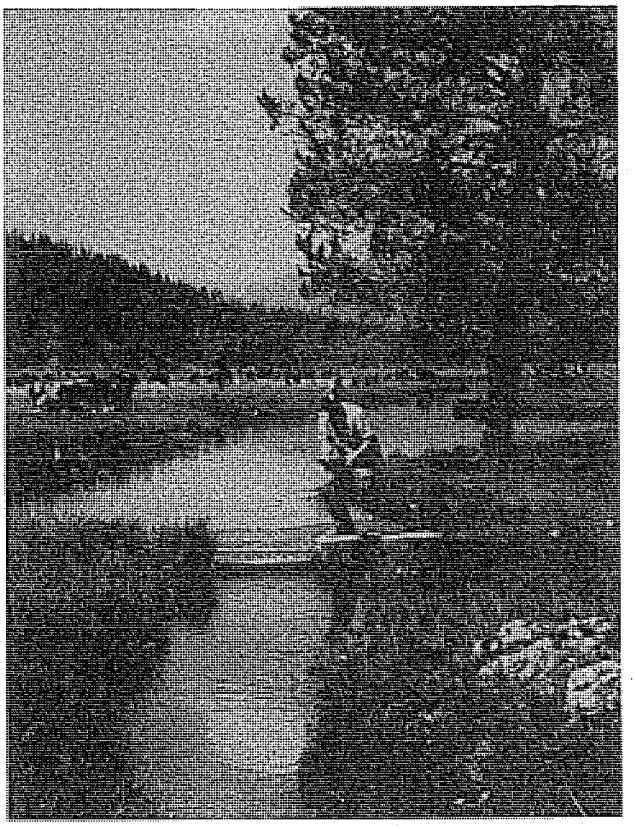
PROBABILITY LEVELS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
.05	4.41	3.63	3.47	.81	.37	.11	.00	.00	.06	.91	3.10	4.39
.10	5.65	4.54	4.35	1.26	.64	.20	.00	.00	.14	1.40	4.17	5.57
.20	7.46	5.84	5.62	2.02	1.15	.38	.02	.03	.36	2.23	5.81	7.30
.30	8.99	6.93	6.68	2.75	1.68	.57	.07	.12	.64	3.02	7.25	8.75
.40	10.46	7.96	7.69	3.50	2.25	.78	.13	.26	.99	3.83	8.65	10.14
.50	11.97	9.01	8.73	4.32	2.89	1.02	.22	.49	1.43	4.72	10.12	11.57
.60	13.63	10.16	9.85	5.26	3.65	1.31	.34	.81	1.98	5.74	11.76	13.13
.70	15.56	11.48	11.15	6.42	4.60	1.67	.51	1.28	2.72	6.98	13.69	14.95
.80	18.04	13.17	12.81	7.96	5.91	2.18	.77	2.02	3.79	8.65	16.20	17.27
.90	21.88	15.77	15.37	10.47	8.08	3.02	1.23	3.42	5.67	11.35	20.14	20.87
.95	25.42	18.15	17.72	12.88	10.20	3.86	1.71	4.92	7.59	13.93	23.83	24.18

THESE VALUES WERE DETERMINED FROM THE INCOMPLETE GAMMA DISTRIBUTION.

Stream Flow Data
USGS - WRD

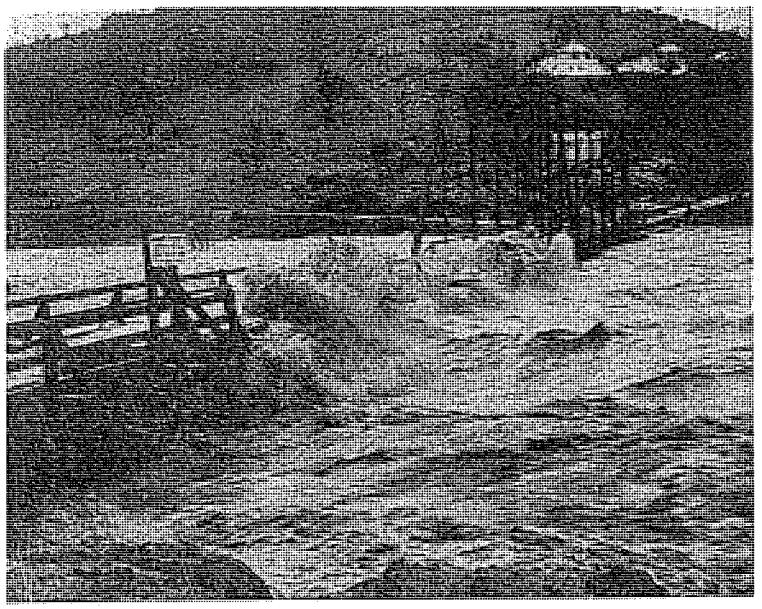
U.S. GEOLOGICAL SURVEY
Open-File Report 90-118

Prepared in cooperation with the
OREGON WATER RESOURCES DEPARTMENT



STATISTICAL SUMMARIES OF STREAMFLOW DATA IN OREGON:

VOLUME 1--MONTHLY AND ANNUAL STREAMFLOW, AND FLOW-DURATION VALUES



14306810 DEER CREEK NEAR SALADO, OR

LOCATION.--Lat 44°32'05", long 123°52'35", in SW 1/4 sec.11, T.12 S., R.10 W., Lincoln County, Hydrologic Unit 17100205, Siuslaw National Forest, on right bank 1,000 ft upstream from mouth, 4.6 mi west of Salado, and 6.5 mi southeast of Toledo.

DRAINAGE AREA.--1.17 mi^2, computed as 749.5 acres on basis of field survey by Oregon State University.

PERIOD OF RECORD.--September 1958 to September 1973.

GAGE.--Water-stage recorder and concrete control. Elevation of gage is 600 ft, from topographic map.

REMARKS.--No regulation or diversion upstream from station.

AVERAGE DISCHARGE.--15 years (water years 1959-73), 6.49 ft^3/s, 75.33 in/yr, 4,700 acre-ft/yr.

EXTREMES FOR PERIOD OF RECORD.--Maximum discharge, 201 ft^3/s Jan. 28, 1965, gage height, 4.21 ft; maximum gage height, 4.39 ft Jan. 20, 1972 (backwater from log); minimum discharge, 0.15 ft^3/s Sept. 2, 14-16, 1972.

STATISTICAL SUMMARIES FOR THE PERIOD 1959-1973

[n = number of values used to compute statistics; months are abbreviated; Ann = annual]

Monthly and annual statistics based on mean daily discharge, in cubic feet per second

Month	n	Minimum	(year)	Maximum	(year)	Mean	Standard deviation	Percent of annual runoff
Oct	15	0.3	1973	5.0	1960	1.9	1.5	2.5
Nov	15	2.5	1973	16	1961	9.0	4.7	11.4
Dec	15	5.9	1961	30	1965	14	6.7	18.3
Jan	15	4.5	1963	28	1965	17	8.3	22.6
Feb	15	2.9	1973	27	1961	12	5.9	14.5
Mar	15	3.5	1965	20	1961	10	5.2	13.6
Apr	15	2.7	1969	11	1963	6.0	2.8	7.6
May	15	1.2	1966	7.0	1960	3.5	1.7	4.5
Jun	15	0.7	1966	5.1	1968	1.8	1.1	2.2
Jul	15	0.5	1970	1.9	1969	0.8	0.4	1.1
Aug	15	0.3	1966	1.2	1968	0.5	0.2	0.7
Sep	16	0.3	1967	2.7	1959	0.7	0.6	0.9
Ann		4.2	1973	9.1	1972	6.5	1.2	100.0

Flow duration statistics based on mean daily discharge

Discharge, in cubic feet per second, which was equaled or exceeded for indicated percent of time

Month	95	90	85	80	75	70	60	50	40	30	25	20	15	10	5	(n)
Oct	0.3	0.3	0.3	0.3	0.4	0.4	0.5	0.9	1.2	1.8	2.4	3.1	3.5	4.5	6.8	15
Nov	1.1	1.3	1.5	1.8	2.1	2.4	3.3	4.6	6.8	9.4	12	14	18	22	31	15
Dec	2.6	3.3	4.3	4.8	5.5	5.5	7.0	9.6	12	15	17	19	23	33	44	15
Jan	3.5	3.9	5.0	5.7	5.7	6.5	8.9	12	14	18	21	24	32	40	54	15
Feb	2.9	3.7	4.6	5.1	5.7	6.4	7.1	8.0	11	13	15	17	20	26	38	15
Mar	2.5	3.2	3.5	4.0	4.4	5.0	5.5	6.9	9.3	12	13	15	18	21	29	15
Apr	2.0	2.2	2.4	2.7	2.9	3.2	3.6	4.3	5.2	6.4	7.0	7.7	9.3	13	15	15
May	1.2	1.4	1.6	1.7	1.9	2.1	2.3	2.5	2.8	3.4	4.1	4.5	5.5	6.0	8.0	15
Jun	0.7	0.8	0.8	0.9	1.0	1.0	1.2	1.3	1.5	1.6	1.8	2.0	2.3	2.8	3.2	15
Jul	0.4	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	1.0	1.1	1.3	1.6	15
Aug	0.2	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.9	15
Sep	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.6	0.7	0.9	1.0	1.4	2.0	16
Ann	0.3	0.3	0.5	0.6	0.7	1.1	1.6	2.9	4.5	6.4	7.3	8.9	12	16	25	

14306800 FLYNN CREEK NEAR SALADO, OR

LOCATION.--Lat 44°32'20", long 123°51'05", in SW 1/4 sec.12, T.12 S., R.10 W., Lincoln County, Hydrologic Unit 17100205, on right bank 1,000 ft upstream from mouth, 3.4 mi west of Salado, and 6.9 mi southeast of Toledo.

DRAINAGE AREA.--0.78 mi^2, computed as 501.96 acres on basis of field survey by Oregon State University.

PERIOD OF RECORD.--September 1958 to September 1973.

GAGE.--Water-stage recorder and concrete control. Elevation of gage is 685 ft, from topographic map.

REMARKS.--No regulation or diversion upstream from station.

AVERAGE DISCHARGE.--15 years (water years 1959-73), 4.37 ft^3/s, 76.08 in/yr, 3,170 acre-ft/yr.

EXTREMES FOR PERIOD OF RECORD.--Maximum discharge, 139 ft^3/s Jan. 11, 1972, gage height, 4.73 ft; minimum, 0.07 ft^3/s Sept. 27, 1967.

STATISTICAL SUMMARIES FOR THE PERIOD 1959-1973

[n = number of values used to compute statistics; months are abbreviated; Ann = annual]

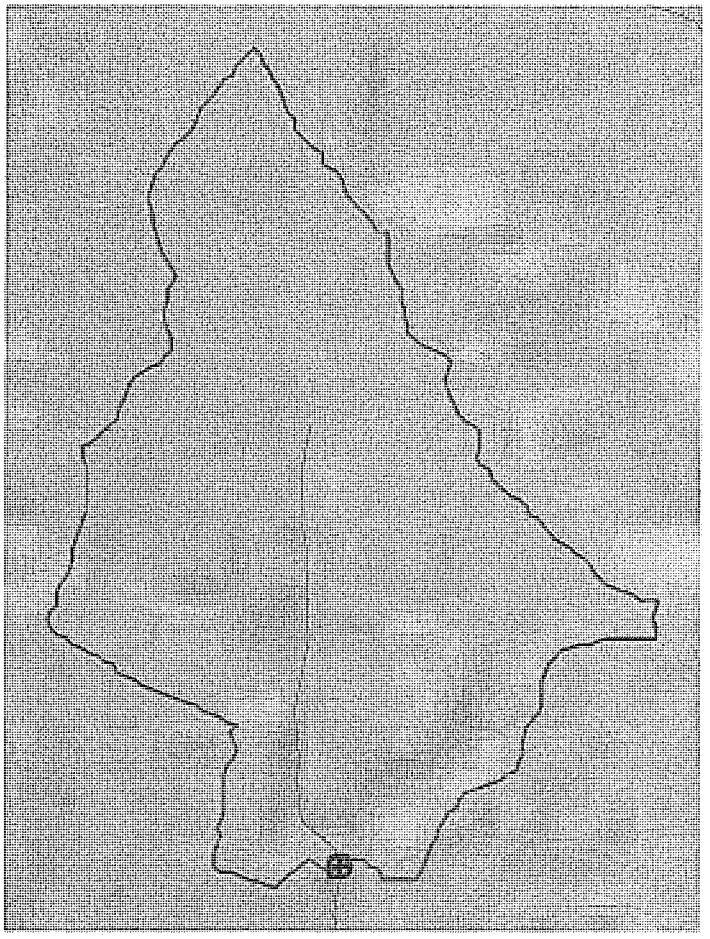
Monthly and annual statistics based on mean daily discharge, in cubic feet per second

Month	n	Minimum	(year)	Maximum	(year)	Mean	Standard deviation	Percent of annual runoff
Oct	15	0.2	1973	3.2	1960	1.0	0.9	1.9
Nov	15	1.0	1973	11	1961	5.7	3.3	10.6
Dec	15	4.4	1961	20	1965	9.6	4.7	18.5
Jan	15	3.2	1963	18	1965	11	5.5	23.0
Feb	15	2.0	1973	19	1961	8.7	4.1	15.3
Mar	15	2.7	1965	14	1961	7.3	3.6	14.0
Apr	15	1.8	1969	8.3	1963	4.2	2.0	7.9
May	15	0.9	1966	5.3	1960	2.4	1.3	4.7
Jun	15	0.5	1966	3.2	1968	1.1	0.7	2.1
Jul	15	0.3	1970	1.2	1969	0.5	0.2	1.0
Aug	15	0.1	1966	1.0	1958	0.3	0.1	0.5
Sep	16	0.1	1965	1.4	1959	0.4	0.3	0.7
Ann		2.7	1973	6.3	1972	4.4	0.8	100.0

Flow duration statistics based on mean daily discharge

Discharge, in cubic feet per second, which was equaled or exceeded for indicated percent of time

Month	95	90	85	80	75	70	60	50	40	30	25	20	15	10	5
Oct	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.4	0.4	0.9	1.3	1.5	2.0	2.3	3.5
Nov	0.5	0.6	0.8	0.9	1.1	1.3	1.8	3.0	3.6	6.2	7.4	8.8	12	15	21
Dec	1.3	2.5	2.9	3.3	3.8	3.8	4.9	6.4	8.2	10	12	14	17	23	30
Jan	2.5	2.9	3.2	3.6	4.5	4.5	5.7	7.2	10	13	15	17	22	27	36
Feb	2.1	2.8	3.2	3.5	3.9	4.4	5.4	6.0	7.5	9.3	11	13	15	18	25
Mar	1.9	2.4	2.6	2.9	3.2	3.6	4.4	4.9	6.1	7.5	8.4	11	13	15	20
Apr	1.4	1.5	1.8	2.0	2.2	2.4	2.6	3.1	3.7	4.4	4.8	5.7	6.7	8.0	12
May	0.8	1.0	1.1	1.2	1.3	1.4	1.5	1.8	2.2	2.6	2.9	3.2	3.4	4.1	5.9
Jun	0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	1.0	1.1	1.1	1.3	1.4	1.7	2.2
Jul	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.6	0.6	0.6	0.7	0.8	1.0
Aug	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.5
Sep	0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.4	0.5	0.6	0.9
Ann	0.1	0.2	0.3	0.4	0.5	0.6	1.0	1.9	2.9	4.3	5.3	6.7	8.4	11	17

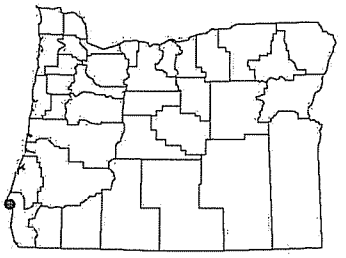


0 1 mile

Pour point

Watershed boundary

N



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Report and output shapefile is avilable for 7 days at:

http://www1.wrd.state.or.us/files/wars/091221.121903.zip

PEAK DISCHARGES FOR SELECTED FREQUENCIES

Report prepared for: autodelin
Time: 12:19 Date: 12/21/2009

Watershed Name: N FK HUBBARD CR

PEAK DISCHARGE CALCULATION BY PREDICTION EQUATION

Peak discharges for the ungaged watershed have been determined from a set of hydrologic prediction equations derived using generalized least squares. The models relate peak discharges to physical watershed characteristics such as area and precipitation. The equations take this form:

Q(T)=(10.0^C0(T))*(CHR1^C1(T))* . . . (CHRn^Cn(T))

Q(T) = Peak Discharge for Return Period T
Cx(T) = Coefficient x for Return Period T
CHR1 = The First Watershed Characteristic
CHRn = The nth Watershed Characteristic

Note: * = multiplication, ^ = exponentiation

For this ungaged watershed, peak discharges were estimated using prediction equations for this flood region:

COASTAL WATERSHEDS

Prediction Equation for Coastal Watersheds

Q(T)=(10.0^C0(T))*(X1^C1(T))*(X2^C2(T)*(X3^C3(T))*(X4^C4(T))*(X5^C5(T))

Q(T)	=	Peak Discharge for Return Period T	
Cx(T)	=	Coefficient x for Return Period T	
X1	=	Drainage area	(square miles)
X2	=	2-year 24-hour precipitation intensity	(inches)
X3	=	Soil permeability	(inches/hour)
X4	=	Mean maximum January temperature	(degrees F)
X5	=	Soil storage capacity	(inches)

Note: * = multiplication, ^ = exponentiation

Prediction Equation Coefficients

Return Period T	Coefficients C0(T)	C1(T)	C2(T)	C3(T)	C4(T)	C5(T)
2	-1.296E+00	9.489E-01	1.360E+00	-1.576E-01	1.280E+00	-4.421E-01
5	-1.881E+00	9.385E-01	1.272E+00	-2.234E-01	1.738E+00	-5.026E-01
10	-2.095E+00	9.324E-01	1.226E+00	-2.552E-01	1.926E+00	-5.267E-01
20	-2.248E+00	9.273E-01	1.190E+00	-2.812E-01	2.069E+00	-5.438E-01
25	-2.291E+00	9.258E-01	1.179E+00	-2.888E-01	2.109E+00	-5.484E-01
50	-2.410E+00	9.215E-01	1.151E+00	-3.111E-01	2.223E+00	-5.605E-01
100	-2.516E+00	9.176E-01	1.126E+00	-3.319E-01	2.325E+00	-5.701E-01
500	-2.723E+00	9.099E-01	1.078E+00	-3.770E-01	2.527E+00	-5.855E-01

Required Watershed Characteristics

Drainage area	(square miles)	0.970
2-year 24-hour precipitation intensity	(inches)	3.660

Soil permeability	(inches/hour)	4.780
Mean maximum January temperature	(degrees F)	54.200
Soil storage capacity	(inches)	0.140

WARNING: WATERSHED CHARACTERISTICS ARE OUT OF BOUNDS

One or more of the required watershed characteristics is an extrapolation from the set used to develop the regression equations.

PEAK DISCHARGES HAVE BEEN CALCULATED, BUT SHOULD BE USED WITH CAUTION.

COASTAL WATERSHEDS
Bounds on Required Watershed Characteristics

Drainage area	(square miles)	0.28	to	673.49
2-year 24-hour precipitation intensity	(inches)	2.52	to	5.80
Soil permeability	(inches/hour)	0.72	to	4.76
Mean maximum January temperature	(degrees F)	42.41	to	53.88
Soil storage capacity	(inches)	0.10	to	0.23

PEAK DISCHARGE ESTIMATES BASED ON PREDICTION EQUATIONS

Return Period years	Peak Flow cfs	95% Confidence Lower Limit cfs	Upper Limit cfs
2	88.6	50.6	155
5	130	76.3	222
10	159	92.4	272
20	186	106	326
25	195	111	344
50	223	124	402
100	251	135	466
500	315	158	629

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NATURAL STREAMF CHECK FILE

HUBBARD CR > PACIFIC OCEAN - AT MOUTH
Watershed ID #: 72974 Basin: SOUTH COAST Exceedance Level: **50%**
Time: 12:38 PM Date: 1/28/2010

Number of Upstream Watersh 0

Month	Watershed Streamflow cfs	CSM cfs/sq mi	Sum % %	North Fork cfs	cfs/sq mi	acre-ft		days/mo	City use Raw gal/day	City use Raw CFS	acre-ft	Flow After City Use CFS	% of Total Flow In Entire Basin
1	56.1	7.74	17.3	8.16	7.74	502	**Jan**	31	134,534	0.21	13	7.95	0.37%
2	57	8.64	17.6	8.29	8.64	460	**Feb**	28	147,858	0.23	13	8.06	0.40%
3	48.8	7.39	16.5	7.10	7.39	436	**March**	31	142,881	0.22	14	6.88	0.45%
4	24.2	3.67	7.9	3.52	3.67	209	**Apr**	30	147,461	0.23	14	3.29	0.94%
5	10.1	1.53	3.4	1.47	1.53	90	**May**	31	180,355	0.28	17	1.19	2.76%
6	11	1.67	3.6	1.60	1.67	95	**June**	30	186,261	0.29	17	1.31	2.62%
7	8.01	1.21	2.7	1.17	1.21	72	**July**	31	250,140	0.39	24	0.78	4.83%
8	5.27	0.8	1.8	0.77	0.8	47	**Aug**	31	249,804	0.39	24	0.38	7.33%
9	3.54	0.54	1.2	0.51	0.54	31	**Sept**	30	210,229	0.33	19	0.19	9.19%
10	4.37	0.66	1.5	0.64	0.66	39	**Oct**	31	201,306	0.31	19	0.32	7.13%
11	23.6	3.58	7.7	3.43	3.58	204	**Nov**	30	173,177	0.27	16	3.16	1.14%
12	55.6	8.42	18.8	8.09	8.42	497	**Dec**	31	169,064	0.26	16	7.83	0.47%

Drainage	Area	6.6		0.96				
Maximum	Relief	1,370.00						
Mean	Slope	11.4						150
Mean	Elevation	500						
Mean	Annual	Precip	82.2		61.8	0.751825		
Mean	January	Min	Temp	33.9				
Mean	July	Min	Temp	49.4				
Yield	63.2							

NATURAL STREAMF CHECK FILE

HUBBARD CR > PACIFIC OCEAN - AT MOUTH
Watershed ID #: 72974 Basin: SOUTH COAST Exceedance Level: **80%**
Time: 10:39 AM Date: 2/2/2010

Number of Upstream Watersh 0

Month	Watershed Streamflow cfs	CSM cfs/sq mi	Sum % %	North Fork cfs	cfs/sq mi	acre-ft		days/mo	City use Raw gal/day	City use Raw CFS	acre-ft	Flow After City Use CFS	% of Total Flow In Entire Basin
1	22.7	3.44	15.9	3.30	3.44	203	Jan	31	134,534	0.21	13	3.09	0.9%
2	27.9	4.23	17.9	4.06	4.23	225	Feb	28	147,858	0.23	13	3.83	0.8%
3	25.4	3.85	17.8	3.69	3.85	227	March	31	142,881	0.22	14	3.47	0.9%
4	14.9	2.26	10.1	2.17	2.26	129	Apr	30	147,461	0.23	14	1.94	1.5%
5	7.7	1.17	5.4	1.12	1.17	69	May	31	180,355	0.28	17	0.84	3.6%
6	5.19	0.79	3.5	0.75	0.79	45	June	30	186,261	0.29	17	0.47	5.6%
7	4.37	0.66	3.1	0.64	0.66	39	July	31	250,140	0.39	24	0.25	8.9%
8	3.66	0.56	2.6	0.53	0.56	33	Aug	31	249,804	0.39	24	0.15	10.6%
9	3.3	0.5	2.2	0.48	0.5	29	Sept	30	210,229	0.33	19	0.15	9.9%
10	2.82	0.43	2	0.41	0.43	25	Oct	31	201,306	0.31	19	0.10	11.0%
11	8.23	1.25	5.6	1.20	1.25	71	Nov	30	173,177	0.27	16	0.93	3.3%
12	19.7	2.98	13.8	2.87	2.98	176	Dec	31	169,064	0.26	16	2.60	1.3%

Drainage	Area	6.6		
Maximum	Relief	1,370.00		
Mean	Slope	11.4		
Mean	Elevation	500		
Mean	Annual	Precip	82.2	
Mean	January	Min	Temp	33.9
Mean	July	Min	Temp	49.4
Yield	30.4			

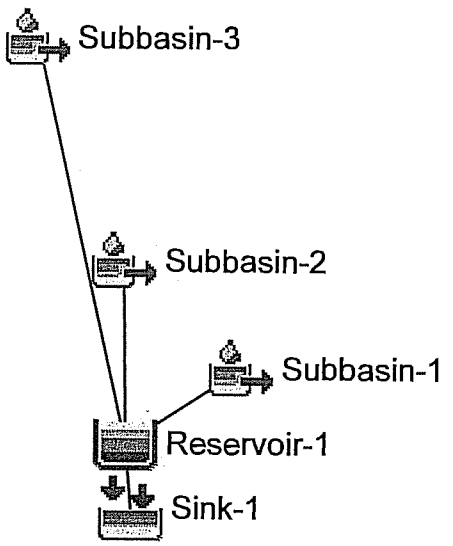
Hydrological Model
HEC-HMS

HEC-HMS

Project : Hubbard Cr

Basin Model : Basin 3

Apr 01 17:37:10 PDT 2010



Project: Hiubbard Cr Simulation Run: 500-year-F

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 500 Year F
Compute Time: 01Apr2010, 17:50:00 Control Specifications: 500-year F

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	324.6	31Dec2009, 15:0
Subbasin-2	0.16	81.2	31Dec2009, 15:0
Subbasin-1	0.16	81.2	31Dec2009, 15:0
Reservoir-1	0.96	486.9	31Dec2009, 15:0
Sink-1	0.96	486.9	31Dec2009, 15:0

Project: Hiubbard Cr

Simulation Run: 500-year-F Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	500 Year F
Compute Time:	17Feb2010, 09:18:43	Control Specifications:	500-year F

Volume Units: AC-FT

Computed Results

Peak Inflow :	487.1 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 15:00
Peak Outflow :	487.0 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 15:00
Total Inflow :	542.5 (AC-FT)	Peak Storage :	69.4 (AC-FT)
Total Outflow :	537.5 (AC-FT)	Peak Elevation :	1017.3 (FT)

Project: Hiubbard Cr Simulation Run: 500-year

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 500-year
Compute Time: 01Apr2010, 17:50:50 Control Specifications: 500-year

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	569.7	31Dec2009, 09:1
Subbasin-2	0.16	168.7	31Dec2009, 08:4
Subbasin-1	0.16	173.3	31Dec2009, 08:3
Reservoir-1	0.96	867.5	31Dec2009, 09:0
Sink-1	0.96	867.5	31Dec2009, 09:0

Project: Hiubbard Cr

Simulation Run: 500-year Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	500-year
Compute Time:	01Apr2010, 17:50:50	Control Specifications:	500-year

Volume Units: IN

Computed Results

Peak Inflow :	875.7 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 08:50
Peak Outflow :	867.5 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 09:00
Total Inflow :	10.72 (IN)	Peak Storage :	75.9 (AC-FT)
Total Outflow :	10.62 (IN)	Peak Elevation :	1018.8 (FT)

Project: Hiubbard Cr Simulation Run: 100-year

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 100-year
Compute Time: 01Apr2010, 18:11:55 Control Specifications: 100-year

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	466.1	31Dec2009, 09:1
Subbasin-2	0.16	138.0	31Dec2009, 08:4
Subbasin-1	0.16	141.8	31Dec2009, 08:3
Reservoir-1	0.96	707.0	31Dec2009, 09:0
Sink-1	0.96	707.0	31Dec2009, 09:0

Project: Hiubbard Cr

Simulation Run: 100-year-F Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	100-year F
Compute Time:	01Apr2010, 18:12:24	Control Specifications:	100-year F

Volume Units: IN

Computed Results

Peak Inflow :	411.0 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 15:00
Peak Outflow :	410.9 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 15:00
Total Inflow :	8.76 (IN)	Peak Storage :	69.8 (AC-FT)
Total Outflow :	8.68 (IN)	Peak Elevation :	1017.4 (FT)

Project: Hiubbard Cr Simulation Run: 100-year-F

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 100-year F
Compute Time: 01Apr2010, 18:12:24 Control Specifications: 100-year F

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	273.9	31Dec2009, 15:0
Subbasin-2	0.16	68.6	31Dec2009, 15:0
Subbasin-1	0.16	68.5	31Dec2009, 15:0
Reservoir-1	0.96	410.9	31Dec2009, 15:0
Sink-1	0.96	410.9	31Dec2009, 15:0

Project: Hiubbard Cr

Simulation Run: 50-year-f Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	50-year -F
Compute Time:	01Apr2010, 18:14:21	Control Specifications:	50-year F

Volume Units: IN

Computed Results

Peak Inflow :	369.9 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 15:00
Peak Outflow :	369.8 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 15:00
Total Inflow :	8.28 (IN)	Peak Storage :	69.0 (AC-FT)
Total Outflow :	8.20 (IN)	Peak Elevation :	1017.2 (FT)

Project: Hiubbard Cr Simulation Run: 50-year-f

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 50-year -F
Compute Time: 01Apr2010, 18:14:21 Control Specifications: 50-year F

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	246.5	31Dec2009, 15:0
Subbasin-2	0.16	61.7	31Dec2009, 15:0
Subbasin-1	0.16	61.7	31Dec2009, 15:0
Reservoir-1	0.96	369.8	31Dec2009, 15:0
Sink-1	0.96	369.8	31Dec2009, 15:0

Project: Hiubbard Cr Simulation Run: 50-year

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 50-year
Compute Time: 01Apr2010, 18:14:14 Control Specifications: 50-year

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	440.2	31Dec2009, 09:1
Subbasin-2	0.16	130.3	31Dec2009, 08:4
Subbasin-1	0.16	133.9	31Dec2009, 08:3
Reservoir-1	0.96	666.3	31Dec2009, 09:0
Sink-1	0.96	666.3	31Dec2009, 09:0

Project: Hiubbard Cr

Simulation Run: 50-year Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	50-year
Compute Time:	01Apr2010, 18:14:14	Control Specifications:	50-year

Volume Units: IN

Computed Results

Peak Inflow :	676.6 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 08:50
Peak Outflow :	666.3 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 09:00
Total Inflow :	8.28 (IN)	Peak Storage :	73.7 (AC-FT)
Total Outflow :	8.21 (IN)	Peak Elevation :	1018.3 (FT)

Project: Hiubbard Cr Simulation Run: 25-year

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 25-year
Compute Time: 01Apr2010, 18:13:03 Control Specifications: 25-year

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	414.3	31Dec2009, 09:1
Subbasin-2	0.16	122.7	31Dec2009, 08:4
Subbasin-1	0.16	126.0	31Dec2009, 08:3
Reservoir-1	0.96	625.1	31Dec2009, 09:0
Sink-1	0.96	625.1	31Dec2009, 09:0

Project: Hiubbard Cr

Simulation Run: 25-year Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	25-year
Compute Time:	01Apr2010, 18:13:03	Control Specifications:	25-year

Volume Units: IN

Computed Results

Peak Inflow :	636.8 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 08:50
Peak Outflow :	625.1 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 09:00
Total Inflow :	7.80 (IN)	Peak Storage :	73.2 (AC-FT)
Total Outflow :	7.72 (IN)	Peak Elevation :	1018.1 (FT)

Project: Hiubbard Cr

Simulation Run: 25-year Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	25-year
Compute Time:	01Apr2010, 18:13:03	Control Specifications:	25-year

Volume Units: IN

Computed Results

Peak Inflow :	636.8 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 08:50
Peak Outflow :	625.1 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 09:00
Total Inflow :	7.80 (IN)	Peak Storage :	73.2 (AC-FT)
Total Outflow :	7.72 (IN)	Peak Elevation :	1018.1 (FT)

Project: Hiubbard Cr Simulation Run: 25-year-F

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 25-year F
Compute Time: 01Apr2010, 18:13:12 Control Specifications: 25-year F

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	232.8	31Dec2009, 15:0
Subbasin-2	0.16	58.3	31Dec2009, 15:0
Subbasin-1	0.16	58.3	31Dec2009, 15:0
Reservoir-1	0.96	349.3	31Dec2009, 15:0
Sink-1	0.96	349.3	31Dec2009, 15:0

Project: Hiubbard Cr

Simulation Run: 25-year-F Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	25-year F
Compute Time:	01Apr2010, 18:13:12	Control Specifications:	25-year F

Volume Units: IN

Computed Results

Peak Inflow :	349.4 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 15:00
Peak Outflow :	349.3 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 15:00
Total Inflow :	7.81 (IN)	Peak Storage :	68.7 (AC-FT)
Total Outflow :	7.74 (IN)	Peak Elevation :	1017.1 (FT)

Project: Hiubbard Cr Simulation Run: 10-year

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 10-year
Compute Time: 01Apr2010, 18:12:39 Control Specifications: 10-year

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	336.6	31Dec2009, 09:1
Subbasin-2	0.16	99.7	31Dec2009, 08:4
Subbasin-1	0.16	102.4	31Dec2009, 08:3
Reservoir-1	0.96	504.4	31Dec2009, 09:1
Sink-1	0.96	504.4	31Dec2009, 09:1

Project: Hiubbard Cr

Simulation Run: 10-year Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	10-year
Compute Time:	01Apr2010, 18:12:39	Control Specifications:	10-year

Volume Units: IN

Computed Results

Peak Inflow :	517.4 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 08:50
Peak Outflow :	504.4 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 09:10
Total Inflow :	6.34 (IN)	Peak Storage :	71.3 (AC-FT)
Total Outflow :	6.28 (IN)	Peak Elevation :	1017.7 (FT)

Project: Hiubbard Cr Simulation Run: 10-year-F

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 10-year F
Compute Time: 01Apr2010, 18:12:51 Control Specifications: 10-year F

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	191.7	31Dec2009, 15:0
Subbasin-2	0.16	48.0	31Dec2009, 15:0
Subbasin-1	0.16	48.0	31Dec2009, 15:0
Reservoir-1	0.96	287.6	31Dec2009, 15:0
Sink-1	0.96	287.6	31Dec2009, 15:0

Project: Hiubbard Cr

Simulation Run: 10-year-F Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	10-year F
Compute Time:	01Apr2010, 18:12:51	Control Specifications:	10-year F

Volume Units: IN

Computed Results

Peak Inflow :	287.7 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 15:00
Peak Outflow :	287.6 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 15:00
Total Inflow :	6.25 (IN)	Peak Storage :	67.5 (AC-FT)
Total Outflow :	6.17 (IN)	Peak Elevation :	1016.9 (FT)

Project: Hiubbard Cr

Simulation Run: 5-yr Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	5-Year
Compute Time:	01Apr2010, 18:14:33	Control Specifications:	5-yr

Volume Units: IN

Computed Results

Peak Inflow :	437.8 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 08:50
Peak Outflow :	426.1 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 09:10
Total Inflow :	5.37 (IN)	Peak Storage :	70.0 (AC-FT)
Total Outflow :	5.31 (IN)	Peak Elevation :	1017.4 (FT)

Project: Hiubbard Cr Simulation Run: 5-yr

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 5-Year
Compute Time: 01Apr2010, 18:14:33 Control Specifications: 5-yr

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	284.8	31Dec2009, 09:1
Subbasin-2	0.16	84.3	31Dec2009, 08:4
Subbasin-1	0.16	86.7	31Dec2009, 08:3
Reservoir-1	0.96	426.1	31Dec2009, 09:1
Sink-1	0.96	426.1	31Dec2009, 09:1

Project: Hiubbard Cr

Simulation Run: 5-year-F Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	5-year F
Compute Time:	01Apr2010, 18:14:26	Control Specifications:	5-year -F

Volume Units: IN

Computed Results

Peak Inflow :	246.6 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 15:00
Peak Outflow :	246.5 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 15:00
Total Inflow :	5.31 (IN)	Peak Storage :	66.6 (AC-FT)
Total Outflow :	5.24 (IN)	Peak Elevation :	1016.7 (FT)

Project: Hiubbard Cr Simulation Run: 5-year-F

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 5-year F
Compute Time: 01Apr2010, 18:14:26 Control Specifications: 5-year -F

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	164.3	31Dec2009, 15:0
Subbasin-2	0.16	41.1	31Dec2009, 15:0
Subbasin-1	0.16	41.1	31Dec2009, 15:0
Reservoir-1	0.96	246.5	31Dec2009, 15:0
Sink-1	0.96	246.5	31Dec2009, 15:0

Project: Hiubbard Cr

Simulation Run: 2-year Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	2-year
Compute Time:	01Apr2010, 18:13:24	Control Specifications:	2-year

Volume Units: IN

Computed Results

Peak Inflow :	398.0 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 08:50
Peak Outflow :	386.9 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 09:10
Total Inflow :	4.88 (IN)	Peak Storage :	69.3 (AC-FT)
Total Outflow :	4.83 (IN)	Peak Elevation :	1017.3 (FT)

Project: Hiubbard Cr Simulation Run: 2-year

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 2-year
Compute Time: 01Apr2010, 18:13:24 Control Specifications: 2-year

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	258.9	31Dec2009, 09:1
Subbasin-2	0.16	76.7	31Dec2009, 08:4
Subbasin-1	0.16	78.8	31Dec2009, 08:3
Reservoir-1	0.96	386.9	31Dec2009, 09:1
Sink-1	0.96	386.9	31Dec2009, 09:1

Project: Hiubbard Cr Simulation Run: 2-year -F

Start of Run: 31Dec2009, 00:00 Basin Model: Basin 3
End of Run: 01Jan2010, 00:10 Meteorologic Model: 2-year F
Compute Time: 01Apr2010, 18:13:31 Control Specifications: 2-year F

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	150.6	31Dec2009, 15:0
Subbasin-2	0.16	37.7	31Dec2009, 15:0
Subbasin-1	0.16	37.7	31Dec2009, 15:0
Reservoir-1	0.96	225.9	31Dec2009, 15:0
Sink-1	0.96	225.9	31Dec2009, 15:0

Project: Hiubbard Cr

Simulation Run: 2-year -F Reservoir: Reservoir-1

Start of Run:	31Dec2009, 00:00	Basin Model:	Basin 3
End of Run:	01Jan2010, 00:10	Meteorologic Model:	2-year F
Compute Time:	01Apr2010, 18:13:31	Control Specifications:	2-year F

Volume Units: IN

Computed Results

Peak Inflow :	226.0 (CFS)	Date/Time of Peak Inflow :	31Dec2009, 15:00
Peak Outflow :	225.9 (CFS)	Date/Time of Peak Outflow :	31Dec2009, 15:00
Total Inflow :	4.84 (IN)	Peak Storage :	66.2 (AC-FT)
Total Outflow :	4.78 (IN)	Peak Elevation :	1016.6 (FT)

Project: Hiubbard Cr Simulation Run: 95% exceedence

Start of Run: 01Dec1999, 00:00 Basin Model: Basin 3
End of Run: 31Dec2000, 00:00 Meteorologic Model: 95 % exceedence
Compute Time: 01Apr2010, 18:15:32 Control Specifications: 95% exceedence

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	1.9	31Dec2000, 00:0
Subbasin-2	0.16	0.5	31Dec2000, 00:0
Subbasin-1	0.16	0.5	31Dec2000, 00:0
Reservoir-1	0.96	2.8	31Dec2000, 00:0
Sink-1	0.96	2.8	31Dec2000, 00:0

Project: Hiubbard Cr

Simulation Run: 95% exceedence Reservoir: Reservoir-1

Start of Run:	01Dec1999, 00:00	Basin Model:	Basin 3
End of Run:	31Dec2000, 00:00	Meteorologic Model:	95 % exceedence
Compute Time:	01Apr2010, 18:15:32	Control Specifications:	95% exceedence

Volume Units: IN

Computed Results

Peak Inflow :	2.8 (CFS)	Date/Time of Peak Inflow :	31Dec2000, 00:00
Peak Outflow :	2.8 (CFS)	Date/Time of Peak Outflow :	31Dec2000, 00:00
Total Inflow :	16.76 (IN)	Peak Storage :	59.4 (AC-FT)
Total Outflow :	16.75 (IN)	Peak Elevation :	1015.1 (FT)

Project: Hiubbard Cr

Simulation Run: 80% Reservoir: Reservoir-1

Start of Run:	01Dec1999, 00:00	Basin Model:	Basin 3
End of Run:	31Dec2000, 00:00	Meteorologic Model:	80% exceedence
Compute Time:	01Apr2010, 18:15:01	Control Specifications:	95% exceedence

Volume Units: IN

Computed Results

Peak Inflow :	4.8 (CFS)	Date/Time of Peak Inflow :	31Dec2000, 00:00
Peak Outflow :	4.8 (CFS)	Date/Time of Peak Outflow :	30Dec2000, 22:00
Total Inflow :	32.92 (IN)	Peak Storage :	59.6 (AC-FT)
Total Outflow :	32.91 (IN)	Peak Elevation :	1015.1 (FT)

Project: Hiubbard Cr Simulation Run: 80%

Start of Run: 01Dec1999, 00:00 Basin Model: Basin 3
End of Run: 31Dec2000, 00:00 Meteorologic Model: 80% exceedence
Compute Time: 01Apr2010, 18:15:01 Control Specifications: 95% exceedence

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	3.2	31Dec2000, 00:0
Subbasin-2	0.16	0.8	31Dec2000, 00:0
Subbasin-1	0.16	0.8	31Dec2000, 00:0
Reservoir-1	0.96	4.8	30Dec2000, 22:0
Sink-1	0.96	4.8	30Dec2000, 22:0

Project: Hiubbard Cr

Simulation Run: 50% Reservoir: Reservoir-1

Start of Run:	01Dec1999, 00:00	Basin Model:	Basin 3
End of Run:	31Dec2000, 00:00	Meteorologic Model:	50% exceedence
Compute Time:	01Apr2010, 18:14:00	Control Specifications:	95% exceedence

Volume Units: IN

Computed Results

Peak Inflow :	7.9 (CFS)	Date/Time of Peak Inflow :	31Dec2000, 00:00
Peak Outflow :	7.9 (CFS)	Date/Time of Peak Outflow :	30Dec2000, 20:00
Total Inflow :	57.74 (IN)	Peak Storage :	59.8 (AC-FT)
Total Outflow :	57.72 (IN)	Peak Elevation :	1015.2 (FT)

Project: Hiubbard Cr Simulation Run: 50%

Start of Run: 01Dec1999, 00:00 Basin Model: Basin 3
End of Run: 31Dec2000, 00:00 Meteorologic Model: 50% exceedence
Compute Time: 01Apr2010, 18:14:00 Control Specifications: 95% exceedence

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak
Subbasin-3	0.64	5.2	31Dec2000, 00:0
Subbasin-2	0.16	1.3	31Dec2000, 00:0
Subbasin-1	0.16	1.3	31Dec2000, 00:0
Reservoir-1	0.96	7.9	30Dec2000, 20:0
Sink-1	0.96	7.9	30Dec2000, 20:0

Appendix C

Distribution System Phased Improvement Costs

City of Port Orford Water System CIP
Water Distribution System Improvements Idaho Street

	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 51,000	$ 51,000
4" PVC Waterline	LF	0	$ 30	$ -
8" PVC Waterline	LF	260	$ 50	$ 13,000
10" PVC Waterline	LF	3020	$ 65	$ 196,300
Service Laterals (short)	LF	308	$ 15	$ 4,613
Service Laterals (long)	LF	1435	$ 30	$ 43,050
Connection to Existing	EA	15	$ 2,500	$ 37,500
Fire Hydrants	EA	5	$ 3,000	$ 15,000
AC Pavement	LF	2296	$ 25	$ 57,400
Gravel Surfacing	LF	984	$ 15	$ 14,760
Highway 101 Crossings	EA	0	$ 25,000	$ -
Misc Fittings	LS	ALL	$ 27,000	$ 27,000
Construction				$ 459,623
Contingency (10%)				$ 45,962
Engineering (17%)				$ 78,136
Administration (4%)				$ 18,385
Total Phase Cost				**$ 602,105**

City of Port Orford Water System CIP Water Distribution System Improvements Jackson Street				
	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 77,000	$ 77,000
4" PVC Waterline	LF	0	$ 30	$ -
8" PVC Waterline	LF	0	$ 50	$ -
10" PVC Waterline	LF	5591	$ 65	$ 363,415
Service Laterals (short)	LF	195	$ 15	$ 2,925
Service Laterals (long)	LF	910	$ 30	$ 27,300
Connection to Existing	EA	16	$ 2,500	$ 40,000
Fire Hydrants	EA	7	$ 3,000	$ 21,000
AC Pavement	LF	3914	$ 25	$ 97,843
Gravel Surfacing	LF	1677	$ 15	$ 25,160
Highway 101 Crossings	EA	0	$ 25,000	$ -
Misc Fittings	LS	ALL	$ 41,000	$ 41,000
Construction				$ 695,642
Contingency (10%)				$ 69,564
Engineering (17%)				$ 118,259
Administration (4%)				$ 27,826
Total Phase Cost				**$ 911,291**

City of Port Orford Water System CIP
Water Distribution System Improvements Phase 1

	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 113,000	$ 113,000
4" PVC Waterline	LF	0	$ 30	$ -
8" PVC Waterline	LF	8875	$ 50	$ 443,750
10" PVC Waterline	LF	420	$ 65	$ 27,300
Service Laterals (short)	LF	705	$ 15	$ 10,575
Service Laterals (long)	LF	3290	$ 30	$ 98,700
Connection to Existing	EA	6	$ 2,500	$ 15,000
Fire Hydrants	EA	6	$ 3,000	$ 18,000
AC Pavement	LF	6507	$ 25	$ 162,663
Gravel Surfacing	LF	2789	$ 15	$ 41,828
Highway 101 Crossings	EA	1	$ 25,000	$ 25,000
Misc Fittings	LS	ALL	$ 60,000	$ 60,000
Construction				$ 1,015,815
Contingency (10%)				$ 101,582
Engineering (17%)				$ 172,689
Administration (4%)				$ 40,633
Total Phase Cost				**$ 1,330,718**

City of Port Orford Water System CIP Water Distribution System Improvements Phase 2				
	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 103,000	$ 103,000
4" PVC Waterline	LF	0	$ 30	$ -
8" PVC Waterline	LF	5560	$ 50	$ 278,000
10" PVC Waterline	LF	2712	$ 65	$ 176,280
Service Laterals (short)	LF	690	$ 15	$ 10,350
Service Laterals (long)	LF	3220	$ 30	$ 96,600
Connection to Existing	EA	4	$ 2,500	$ 10,000
Fire Hydrants	EA	5	$ 3,000	$ 15,000
AC Pavement	LF	5790	$ 25	$ 144,760
Gravel Surfacing	LF	2482	$ 15	$ 37,224
Highway 101 Crossings	EA	0	$ 25,000	$ -
Misc Fittings	LS	ALL	$ 58,000	$ 58,000
Construction				$ 929,214
Contingency (10%)				$ 92,921
Engineering (17%)				$ 157,966
Administration (4%)				$ 37,169
Total Phase Cost				**$ 1,217,270**

City of Port Orford Water System CIP
Water Distribution System Improvements Phase 3

	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 150,000	$ 150,000
4" PVC Waterline	LF	0	$ 30	$ -
8" PVC Waterline	LF	7093	$ 50	$ 354,650
10" PVC Waterline	LF	4917	$ 65	$ 319,605
Service Laterals (short)	LF	878	$ 15	$ 13,163
Service Laterals (long)	LF	4095	$ 30	$ 122,850
Connection to Existing	EA	4	$ 2,500	$ 10,000
Fire Hydrants	EA	10	$ 3,000	$ 30,000
AC Pavement	LF	8407	$ 25	$ 210,175
Gravel Surfacing	LF	3603	$ 15	$ 54,045
Highway 101 Crossings	EA	0	$ 25,000	$ -
Misc Fittings	LS	ALL	$ 84,000	$ 84,000
Construction				$ 1,348,488
Contingency (10%)				$ 134,849
Engineering (17%)				$ 229,243
Administration (4%)				$ 53,940
Total Phase Cost				**$ 1,766,519**

City of Port Orford Water System CIP Water Distribution System Improvements Phase 4				
	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 151,000	$ 151,000
4" PVC Waterline	LF	0	$ 30	$ -
8" PVC Waterline	LF	9898	$ 50	$ 494,900
10" PVC Waterline	LF	1169	$ 65	$ 75,985
Service Laterals (short)	LF	518	$ 15	$ 7,763
Service Laterals (long)	LF	2415	$ 30	$ 72,450
Connection to Existing	EA	17	$ 2,500	$ 42,500
Fire Hydrants	EA	2	$ 3,000	$ 6,000
AC Pavement	LF	7747	$ 25	$ 193,673
Gravel Surfacing	LF	3320	$ 15	$ 49,802
Highway 101 Crossings	EA	8	$ 25,000	$ 200,000
Misc Fittings	LS	ALL	$ 66,000	$ 66,000
Construction				$ 1,360,072
Contingency (10%)				$ 136,007
Engineering (17%)				$ 231,212
Administration (4%)				$ 54,403
Total Phase Cost				**$ 1,781,694**

City of Port Orford Water System CIP
Water Distribution System Improvements Phase 5

	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 164,000	$ 164,000
4" PVC Waterline	LF	0	$ 30	$ -
8" PVC Waterline	LF	10321	$ 50	$ 516,050
10" PVC Waterline	LF	2781	$ 65	$ 180,765
Service Laterals (short)	LF	803	$ 15	$ 12,038
Service Laterals (long)	LF	3745	$ 30	$ 112,350
Connection to Existing	EA	6	$ 2,500	$ 15,000
Fire Hydrants	EA	9	$ 3,000	$ 27,000
AC Pavement	LF	9171	$ 25	$ 229,285
Gravel Surfacing	LF	3931	$ 15	$ 58,959
Highway 101 Crossings	EA	3	$ 25,000	$ 75,000
Misc Fittings	LS	ALL	$ 85,000	$ 85,000
Construction				$ 1,475,447
Contingency (10%)				$ 147,545
Engineering (17%)				$ 250,826
Administration (4%)				$ 59,018
Total Phase Cost				**$ 1,932,835**

City of Port Orford Water System CIP Water Distribution System Improvements Phase 6				
	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 90,000	$ 90,000
4" PVC Waterline	LF	1557	$ 30	$ 46,710
8" PVC Waterline	LF	6280	$ 50	$ 314,000
10" PVC Waterline	LF	0	$ 65	$ -
Service Laterals (short)	LF	615	$ 15	$ 9,225
Service Laterals (long)	LF	2870	$ 30	$ 86,100
Connection to Existing	EA	4	$ 2,500	$ 10,000
Fire Hydrants	EA	4	$ 3,000	$ 12,000
AC Pavement	LF	5486	$ 25	$ 137,148
Gravel Surfacing	LF	2351	$ 15	$ 35,267
Highway 101 Crossings	EA	1	$ 25,000	$ 25,000
Misc Fittings	LS	ALL	$ 47,000	$ 47,000
Construction				$ 812,449
Contingency (10%)				$ 81,245
Engineering (17%)				$ 138,116
Administration (4%)				$ 32,498
Total Phase Cost				**$ 1,064,308**

City of Port Orford Water System CIP Water Distribution System Improvements Phase 7				
	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 59,000	$ 59,000
4" PVC Waterline	LF	0	$ 30	$ -
8" PVC Waterline	LF	1549	$ 50	$ 77,450
10" PVC Waterline	LF	3264	$ 65	$ 212,160
Service Laterals (short)	LF	180	$ 15	$ 2,700
Service Laterals (long)	LF	840	$ 30	$ 25,200
Connection to Existing	EA	3	$ 2,500	$ 7,500
Fire Hydrants	EA	3	$ 3,000	$ 9,000
AC Pavement	LF	3369	$ 25	$ 84,228
Gravel Surfacing	LF	1444	$ 15	$ 21,659
Highway 101 Crossings	EA	0	$ 25,000	$ -
Misc Fittings	LS	ALL	$ 33,000	$ 33,000
Construction				$ 531,896
Contingency (10%)				$ 53,190
Engineering (17%)				$ 90,422
Administration (4%)				$ 21,276
Total Phase Cost				**$ 696,784**

City of Port Orford Water System CIP Deady Street Pump Station Improvements				
	Unit	Quantity	Unit Price	Cost
Mobilization	LS	ALL	$ 14,000	$ 14,000
Pump Station @ 9th & Jefferson	EA	1	$ 65,000	$ 65,000
8-inch Water Main -9th St.	LF	350	$ 50	$ 17,500
4-inch Water Main - 9th St.	LF	300	$ 25	$ 7,500
Re-establish Service Connections	EA	2	$ 1,200	$ 2,400
3/4" pressure regulator	EA	4	$ 150	$ 600
4-inch connection	EA	2	$ 2,500	$ 5,000
8-inch connection	EA	1	$ 3,000	$ 3,000
End cap and flush @ (E) 6" water	EA	2	$ 3,750	$ 7,500
AC Pavement Patch	LF	350	$ 20	$ 7,000
Valves & Appurtenances	LS	ALL	$ 3,000	$ 3,000
Total Construction Cost				**$ 132,500**

City of Port Orford Water System CIP Coast Guard Pump Station Improvements				
	Unit	Quantity	Unit Price	Cost
Construction Facilities	LS	ALL	$ 17,000	$ 17,000
Pump Station	LS	ALL	$ 95,000	$ 95,000
Building Modifications	LS	ALL	$ 12,500	$ 12,500
Mainline Meter	EA	1	$ 5,000	$ 5,000
Total Construction Cost				**$ 129,500**