MASON COUNTY PUD NO. 1

MASON COUNTY WASHINGTON

FINCH CREEK WASTEWATER FEASIBILITY STUDY

G & O No. 99680

AUGUST 2000

Gray & Osborne, Inc.

CONSULTING ENGINEERS

MASON COUNTY PUD NO. 1

MASON COUNTY **WASHINGTON**

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FINCH CREEK WASTEWATER FEASIBILITY STUDY

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CHAPTER 1

CHAPTER 1

INTRODUCTION

The community of Hoodsport lies on the western shoreline of Hood Canal, approximately five miles north of the canal's Great Bend. Finch Creek is the focus of this study and is located in north Hoodsport. Finch Creek flows from the west, travels through the community, and drains into Hood Canal. On several occasions, elevated fecal coliform levels have been measured in the lower portion of the creek, raising concerns about failing onsite septic systems located along the creek.

The Mason County PUD No. 1 (PUD No. 1) is designated to provide water and sewer service in the area. Therefore, PUD No. 1 contracted with Gray & Osborne, Inc. to investigate the feasibility of a community wastewater collection and treatment system for the area of Finch Creek. This feasibility report has been prepared to meet the planning requirements of Chapter 173-240-060 WAC, Engineering Report, as well as facility planning requirements of the Department of Ecology. The plan was funded by the Mason County PUD No. 1 with a grant from the USDA Forest Service.

BACKGROUND

The Finch Creek area has documented water quality problems that have resulted in restrictions to swimming and shellfish harvesting. For the most part, these problems have been attributed to failing commercial and residential onsite sewage systems along the Finch Creek corridor. The area has seasonally high ground water and soils underlain by a restrictive layer that limits infiltration of the septic effluent. It is suspected that these failing systems drain to Finch Creek, which, in turn, discharges contaminants directly into Hood Canal.

Since 1976, local volunteers and County health officials recorded elevated fecal coliform levels in Finch Creek. In 1987, the Washington Departments of Health (DOH) and Fish and Wildlife (WDFW) closed the Hoodsport fish hatchery due to the extremely high fecal coliform counts in the Creek. At that time, the public access tidelands at the mouth of Finch Creek were closed to shellfish harvesting. Private tideland owners were also warned against harvesting shellfish on local beaches. These restrictions and closures remain in place today.

In 1993, Mason County conducted an educational project under a grant from the Puget Sound Water Quality Authority's Public Involvement and Education (PIE) Fund. During that project, 30 failing onsite sewage systems were identified along Finch Creek. According to County records, only a few of these systems have been repaired. Some

systems were deemed not repairable due to small lot sizes and the high water table along the creek.

In 1996, Mason County approached the Department of Ecology (Ecology) about implementing Ecology's Small Town Environment Program (STEP) as a means of trying to solve the water quality problems on Finch Creek. The STEP program is a self-help approach that assists local residents to reduce the cost of solving their problems through the use of voluntary labor. In 1997, a group made up primarily of Finch Creek residents, known locally as the "Fecal Ferrets", began to work with Ecology staff to further document the occurrence of fecal coliform contamination within the community. For several months, the local group collected water samples from Finch Creek and several drainage ditches in the community. The samples showed varying levels of contamination. They also collected data from private wells along Finch Creek. The group held several public meetings that included presentations from Ecology, Mason County Health Services, and Taylor United Shellfish Company.

In 1996, a ballot measure was passed that gave PUD No. 1 the authority to provide wastewater service in Mason County. Since that time, the PUD No. 1 has taken the lead in seeking funding for solutions to the water quality problems along Finch Creek. This effort included obtaining funding for the feasibility study.

PURPOSE

The purpose of this study is to evaluate the feasibility of providing a community wastewater collection and treatment system for the residences and businesses located along the Finch Creek corridor. The study will determine the service area, develop design flows and organic loadings, and evaluate collection and treatment alternatives. The study will also present proposals for funding of the recommended approach.

SCOPE OF WORK

The following tasks constitute the Scope of Work for the Mason County PUD No. 1 Finch Creek Design and Feasibility Study.

TASK 1: BACKGROUND INFORMATION

Pertinent background information will be reviewed and presented including, but not limited to, Finch Creek water quality data, ground water quality data, well logs, onsite treatment system conditions and failures, soils information, and available aerial photography.

TASK 2: DEVELOPMENT OF THE SERVICE AREA

Service area boundaries will be identified and supporting data will be provided for the service area. The supporting information is to include the number of service connections; the potential for future service area growth and expansion; and identification of essential physical environmental features such as area land use, topography, geology, hydrogeology, soils, surface waters, and climate.

TASK 3: DEVELOPMENT OF WASTEWATER DESIGN FLOWS AND ORGANIC LOADINGS

For the proposed service area, design flows and organic loadings for the proposed community collection and treatment system will be determined.

TASK 4: IDENTIFICATION OF REGULATORY AUTHORITY AND REQUIREMENTS

The regulatory framework for oversight of wastewater collection and treatment will be presented. The regulatory requirements for the design flows and loadings will be discussed with respect to their impact upon the proposed project.

TASK 5: EVALUATION OF TREATMENT AND DISPOSAL ALTERNATIVES

An evaluation of appropriate alternatives for wastewater treatment and disposal, including enhanced onsite treatment and disposal, community land treatment and disposal, and wastewater reuse will be performed. Each alternative will be discussed in reference to advantages, disadvantages, capital costs, and operation and maintenance costs. A preliminary layout for each alternative will also be provided.

In addition, sites for potential land treatment and disposal system will be identified and evaluated. As part of the evaluation, a soils analysis will be performed to determine the feasibility of the site for effluent disposal.

TASK 6: EVALUATION OF COLLECTION SYSTEM ALTERNATIVES, INCLUDING PUMP STATIONS

Alternatives for collection and transport of the wastewater to a community treatment system will be discussed and evaluated. For upland site treatment and disposal alternatives, force main routing, possible pump stations and their locations will be evaluated. The advantages, disadvantages, capital costs, and operation and maintenance costs will be presented for each alternative.

TASK 7: DEVELOPMENT OF RECOMMENDED APPROACH

A preferred approach will be recommended based on both cost and non-costs factors. Non-cost factors generally include items such as reliability and community acceptance. The recommended approach will be presented with proposed location of the facilities, routing and siting layouts, hydraulic analysis, cost estimates, a financing plan, an environmental assessment, and a discussion of required permits and a proposed implementation schedule. The study will present a cost per service in terms of both debt service and maintenance costs for all proposed facilities.

TASK 8: ENVIRONMENTAL ANALYSIS/IDENTIFICATION OF NECESSARY EASEMENTS AND PERMITS

The necessary environmental analysis, e.g. the State Environmental Policy Act (SEPA) checklist, will be prepared. The SEPA analysis will consider the direct and indirect impacts of the recommended approach, permits and easements that will be required, as well as a discussion of pertinent governmental regulations and polices that may affect the implementation of the plan. At a minimum, the SEPA review will include consideration of land use plans, ordinances, floodplain maps, shoreline management programs, sensitive areas, wetland regulations, growth management plans, fish and wildlife habitat plans.

TASK 9: COMMUNITY INVOLVEMENT

With concurrence from the PUD, a plan for community involvement will be developed, which is to be implemented throughout the project. The Engineer will represent the PUD in any public meetings concerning the development of the planned approach. The Engineer will also represent the PUD in meetings with regulatory and funding agencies with regard to this project.

TASK 10: FEASIBILITY REPORT DEVELOPMENT

A written report that incorporates all of the aforementioned tasks will be developed and presented in draft form to the PUD. The PUD's comments and those of the regulatory agencies will be addressed in writing and incorporated into the final report, as appropriate.

PREVIOUS REPORTS

Mason County Comprehensive Plan, Department of Community Development, April 1996. Comprehensive Plan Amendment, Department of Community Development, May 2000.

The Mason County Comprehensive Plan, which is required under the Growth Management Act (GMA), is intended to serve as a guide for public decision making within Mason County. The plan is developed through a public process and presents the goals identified for the County by local citizens. The Comprehensive Plan guides development and sets county-wide planning policies and land use regulations to manage growth. It addresses how the County will meet the needs and demands of anticipated growth and sets policies for County investment in roads, water, sewer, parks, and other public facilities.

The plan must ensure that resource lands and the environment are protected. The elements of the Comprehensive Plan include land use and rural lands, housing, capital facilities, utilities, and transportation.

PROJECT CONTACTS

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CHAPTER 2

CHAPTER 2

STUDY AREA ENVIRONMENT

AREA LOCATION

The Finch Creek study area is located in the unincorporated community of Hoodsport in Mason County, Washington. A description of the study area location is presented below.

MASON COUNTY

Mason County is situated in western Washington on the southeastern portion of the Olympic Peninsula. The county is bordered on the north by Jefferson County, on the west and southwest by Grays Harbor County, on the south and southeast by Thurston County, on the east by Pierce, and on the east and northeast by Kitsap County (Figure 2-1). Mason County, which is approximately 968 square miles, remains predominately rural despite recent urban migration from Thurston, Kitsap and Pierce Counties. The economy is supported by natural resource industries such as timber products and shellfish.

The City of Shelton serves as the county seat and is currently the only incorporated area in the county. Two Native American Tribes, the Skokomish Nation and the Squaxin Nation, have reservations within the boundaries of the county. Mason County includes over 90 miles of marine shoreline and nearly 100 freshwater lakes. There are also three major river systems, including the Skokomish, the Hamma Hamma, and the Satsop; as well as a number of smaller tributaries and creeks.

HOODSPORT

Hoodsport is a rural community lying on the western shoreline of Hood Canal, along US Highway 101 (Olympic Highway). The community is approximately ten miles north of the City of Shelton and about five miles north of the canal's Great Bend. The small community of Lilliwaup lies three miles to the north and the Potlatch community is about three miles to the south. The Olympic National Forest and Lake Cushman, including their recreational resources, lie directly west of Hoodsport.

Hoodsport is primarily residential, with a limited number of small businesses, including one motel. Most of the businesses provide essential services, while several are oriented to the many tourists that pass through the area. The USDA Forest Service maintains a Visitor Center and compound in Hoodsport. There is also a fish hatchery at the mouth of Finch Creek, owned and operated by the Washington Department of Fish and Wildlife (WDFW).

In the Mason County Comprehensive Plan, the Hoodsport area was designated as a "Rural Activity Center" (RAC). The Comprehensive Plan defines the RAC as follows:

> "Concentrated settlements within rural lands that may include a variety of residential, small scale commercial, resource-based and rural light industrial, recreation, and public
> uses. They may also include a compact, pedestrian-oriented core. They may be served by community water systems and have community sewage treatment facilities but have only rural governmental services. They reflect an existing development pattern, but they are not intended to expand."

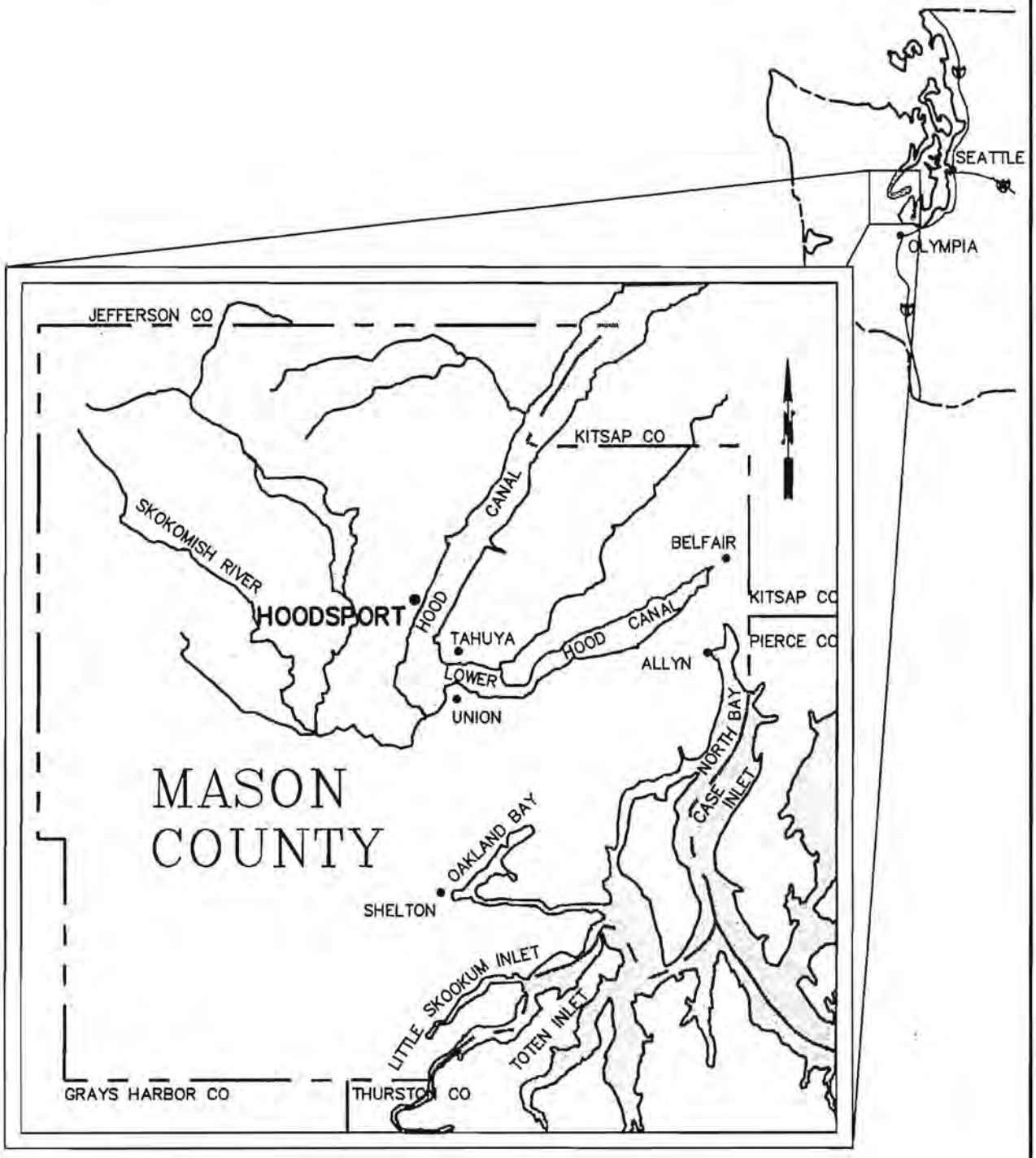
The Mason County Shoreline Management Program (SMP) has designated Highway 101 from Lilliwaup Falls to Potlatch as "Urban Commercial". The Urban Commercial designation is defined in the SMP as follows:

> "The Urban Commercial Environment is an area of high intensity commercial land use. The environment does not necessarily include all shorelines within an incorporated city, but is particularly suitable to those areas presently subjected to extremely intensive use pressure, as well as areas planned to accommodate commercial expansion. Shorelines planned for future commercial expansion should not have a high priority for designation as an alternative environment.
>
> Because shorelines suitable for urban commercial uses are a limited resource, emphasis should be given to development within already developed areas and particularly to water-dependent commercial uses requiring frontage on navigable waters."

FINCH CREEK STUDY AREA

Finch Creek flows out of a wetland area in the Olympic Mountains, beginning at an elevation of about 860 feet. The creek flows south and then east through a narrow valley about three miles before coming into the western edge of Hoodsport. The creek valley spreads out at about elevation 100 feet, but continues to be lined with tracts of forest and steep slopes.

From the point of 100-foot elevation, the creek flows down about one mile to where it empties into Hood Canal. Much of this mile long stretch of creek is bordered by homes with the lower portion occupied by several businesses as well as the Forest Service Visitor Center. The WDFW fish hatchery, which diverts its source water from Finch Creek, is located at the mouth of the creek. It is this lower stretch of the creek, bordered



MASON COUNTY PUD No. 1
FINCH CREEK WASTEWATER FEASIBILITY STUDY
FIGURE 2-1
MASON COUNTY LOCATION MAP

Gray & Osborne, Inc.
CONSULTING ENGINEERS

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by homes and businesses, that is the primary study area and focus of this report. The Finch Creek study area is shown in Figure 2-2.

AREA DESCRIPTION

Various natural features of the study area location are discussed below, such as climate and precipitation, geology, soils, critical areas, flood plains, wetlands, air quality, and surface and ground water resources. Information on public the utilities available in the area is also discussed.

CLIMATE AND PRECIPITATION

The Olympic Mountains lying to the west are a major influence on the climate of Mason County. Hoodsport lies on the eastern side in the shadow of the Olympic Mountains. As a result, the community receives a relatively high rate of the precipitation that moves east over the mountains from the Pacific Ocean. Precipitation levels measured at the Cushman Powerhouse Station are presented in Table 2-1. During the recording period ranging from 1988 to 1989, annual rainfall averaged about 103 inches. The lowest annual average rainfall was 72.2 inches (1993), and the highest average annual rainfall was 115.5 inches (1997). November had the highest monthly average rainfall during this period, while July had the lowest monthly rainfall.

TABLE 2-1

Total Monthly Precipitation (Inches)
Cushman Powerhouse Station, 1988 to 1998

	1988	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	Ave
January	11.1	7.75	23.5	10.6	24.7	9.78	10.1	17.2	14.1	18.7	22.2	15.4
February	3.37	8.34	12.8	12.5	10.0	0.49	12.7	12.8	15.6	6.75	14.7	10.0
March	10.4	13.6	6.79	7.69	1.64	10.4	9.16	12.6	2.38	22.6	6.91	9.5
April	6.64	6.22	2.74	11.1	8.86	11.3	5.50	5.38	13.4	7.06	2.63	8.1
May	5.40	2.95	2.61	3.11	0.04	5.10	3.78	1.40	4.52	5.10	1.32	3.2
June	1.91	1.60	4.09	1.32	2.08	3.61	2.36	1.37	1.20	2.73	0.63	2.1
July	0.92	3.51	0.27	0.83	0.81	2.93	0.75	1.66	1.03	1.76	0.44	1.4
August	0.35	1.03	1.53	4.94	0.74	0.49	0.34	3.18	1.10	2.88	0.14	1.5
September	4.20	0.30	0.01	T	2.83	0.00	1.91	3.46	3.10	10.6	1.42	2.8
October	3.91	7.30	9.26	2.14	6.47	3.03	10.0	9.53	12.4	16.8	5.57	7.9
November	21.0	10.2	25.8	16.4	10.0	5.75	17.4	22.9	13.0	11.0	32.0	16.9
December	9.24	9.39	12.3	10.9	9.22	19.0	26.8	18.0	24.2	9.66	25.3	15.8
Annual Total	78.4	72.2	101.7	97.9	77.4	71.9	100.8	109.2	106.0	115.5	113.3	

Temperature readings recorded at the Cushman Powerhouse for the period ranging from 1988 to 1998 are shown in Table 2-2. The average temperature over the ten-year period was 52°. July was the warmest month with an average monthly temperature of 65.0°, while December has the coolest average monthly temperature at 39.3°.

TABLE 2-2

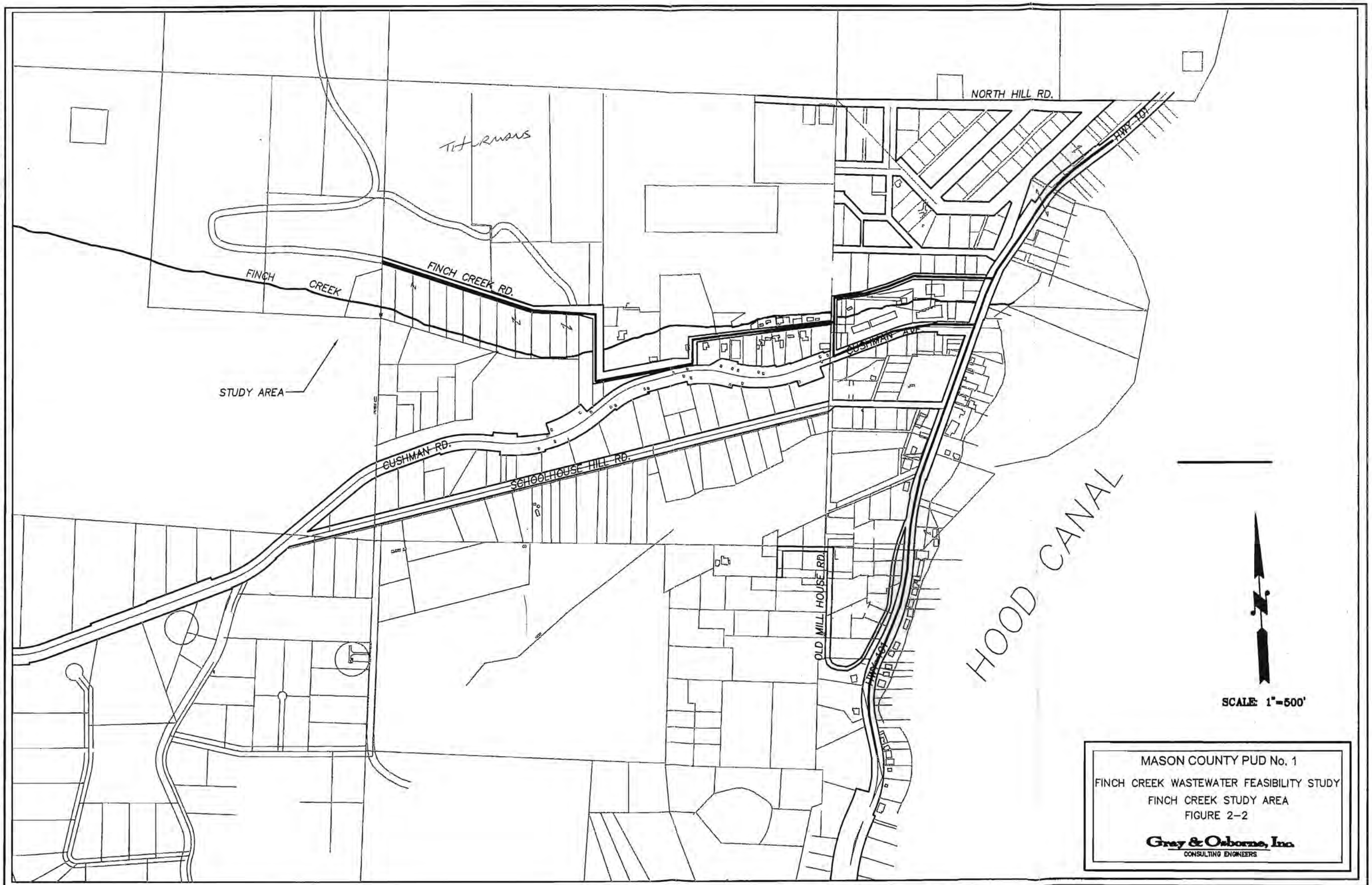
Average Temperature (F°)
Cushman Powerhouse Station, 1988 to 1998

	1988	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	Ave
January	39.1	40.0	41.3	39.2	42.8	36.7	43.3	42.6	39.1	39.9	41.4	40.5
February	43.8	36.3	38.4	46.4	45.6	40.3	38.6	43.1	41.0	41.9	43.7	41.7
March	44.9	42.6	45.9	43.5	50.1	46.4	46.9	45.3	46.4	43.8	46.1	45.6
April	50.6	52.4	52.4	48.2	52.4	50.1	51.1	49.1	50.2	48.4	49.8	50.4
May	54.7	55.7	54.3	54.1	59.2	59.1	56.2	57.4	52.0	58.1	55.6	56.0
June	59.0	61.9	59.6	57.1	63.6	59.1	58.5	61.0	59.0	58.9	60.1	59.8
July	64.5	62.4	67.2	65.0	65.7	61.3	65.8	65.1	66.1	64.5	67.1	65.0
August	60.2	63.0	66.8	65.4	65.6	63.9	65.1	61.4	65.6	66.6	62.9	64.2
September	59.6	61.3	62.9	61.2	58.6	60.0	61.7	63.2	57.0	61.0	62.9	60.9
October	54.9	52.2	50.1	51.7	53.4	54.2	50.3	51.1	50.5	51.3	51.7	51.9
November	44.8	45.6	45.6	46.8	44.3	41.6	39.5	48.0	42.7	47.6	45.7	44.7
December	40.5	42.1	34.6	42.6	38.1	39.3	38.9	40.2	37.2	40.7	38.4	39.3
Ann. Ave	51.4	51.3	51.6	51.8	53.3	51.0	51.3	52.3	50.6	51.9	52.1	

GEOLOGY AND TOPOGRAPHY

Three geological provinces combine to form Mason County, the Puget Sound Lowlands, the Olympic Mountains, and the Black Hills. The County is divided by seven watershed drainage basins; including Case Inlet, Chehalis, Lower Hood Canal, Oakland Bay, Skokomish, Totten-Little Skookum, and West Hood Canal. Hoodsport and the Finch Creek area are located within the Olympic Mountains geological province and the West Hood Canal Watershed.

The region is similar in geology and topography to other coastal regions in the Puget Sound basin, reflecting the influence of mountain building and glacial activity. Uplifting of the Cascade and Olympic Mountains caused the lowland areas of the Puget Sound trough to become sites of deposition for sedimentary materials washed down from the surrounding mountain ranges. The underlying volcanic bedrock is overlain with several thousand feet of marine sedimentary rock. Periods of glaciation, followed by erosion and deposition, resulted in the rolling topography found in the Hood Canal watersheds. The topography surrounding Finch Creek is relatively steep. Elevations bordering the lower portion of the creek reach about 200 feet and increase to 700 and 800 feet at the upper reaches. Land bordering the creek may have slopes as great as 30 to 45 percent (Figure 2-3).



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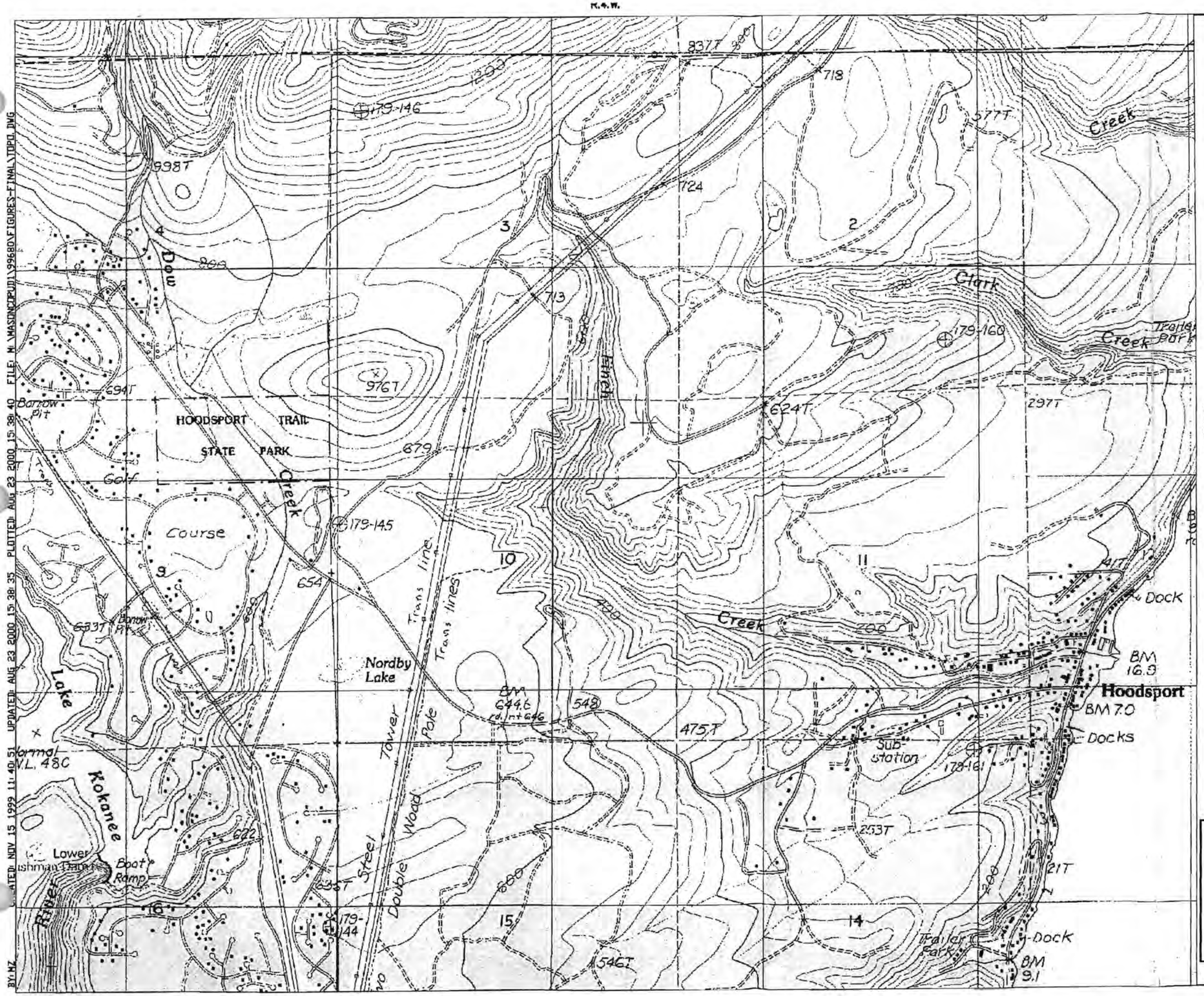

MASON COUNTY PUD No.1
FINCH CREEK WASTEWATER FEASIBILITY STUDY
USGS TOPOGRAPHIC MAP
FIGURE 2-3

Gray & Osborne, Inc.
CONSULTING ENGINEERS

BY: MZ DATED: NOV 15 1999 11:40:51 UPDATED: AUG 23 2000 15:38:35 PLOTTED: AUG 23 2000 15:38:40 FILE: M:\MASONCOPUD1\99680\FIGURES-FINAL\TOPO1.DWG

SOILS

The soils along the shoreline of Hoodsport have been classified by the Natural Resource Conservation Service (formerly Soil Conservation Service) as Cloquallum silt loam and Hoodsport gravelly sandy loam. Upstream along Finch Creek the soils include Grove gravelly sandy loam, Juno sandy loam, and to a minor extent Riverwash. Soils along the upper slopes of Finch Creek include Hoodsport gravelly sandy loam, and to a lesser extent, Grove gravelly sandy loam (Figure 2-4). Table 2-3 describes the important characteristics of these soils, including natural drainage, occurrence of high water table, moisture-supplying capacity, root penetration limits, depth to layer that limits root penetration, natural fertility, and special management problems.

VEGETATION

The Cloquallum soils series supports native vegetation of primarily Douglas fir with smaller areas of maple and alder. The undergrowth in these soils is mainly salal. The vegetation supported by the Grove series is coniferous forest, with an understory dominated by the lower growing mosses, kinnikinnick, and snowberry; and a mix of huckleberry, salal, and Oregon-grape. The Hoodsport series provides growing conditions that supports a dense forest of Douglas-fir and hemlock.

CRITICAL AREAS

The Mason County Interim Resource Ordinance identifies three types of geologic hazard areas: landslide, seismic, and erosion. Landslide hazard areas have an increased potential for landslide and other earth movement, while seismic hazard areas are particularly susceptible to damage from earthquakes and other seismic activity. Erosion hazard areas are more susceptible to excessive erosion that can result in bank destabilization and increased sedimentation to streams, rivers, lakes, and wetlands. Each critical hazard area for the Finch Creek area is described below:

Landslide Hazard Areas

The Mason County Comprehensive Plan identifies the upper slopes of Finch Creek as a "land slide hazard area". Though the area in the immediate vicinity of the creek has relatively flat slopes (0 -5 percent), the slopes at the upper end and surrounding Finch Creek range from 30 to 45 percent. Unconsolidated glacial deposits on hillsides greater than 40 percent are particularly susceptible to land slide hazard. The steep slopes in the Finch Creek area are Hoodsport gravelly sandy loam, relatively unconsolidated, underlain by compact till. To maintain the stability of these steep slopes, forest practices activities and residential development would be limited or prohibited under the Interim Resource Ordinance.

TABLE 2-3

Soils Characterization
Hoodsport and Finch Creek

Soil	Natural Drainage	Occurrence of High Water Table	Moisture-Supplying Capacity[1]	Root Penetration Limits	Depth to Layer that Limits Root Penetration[2]	Natural Fertility	Special Management Problems
Cloquallum silt loam (Cc, 5 to 15 percent slopes)	Moderately Good	None	Good	None	Moderately Shallow	Medium	Low Fertility
Grove gravelly sandy loam (Gh, 0 to 5 percent slopes)	Somewhat Excessive	None	Low	Gravel	Moderately Deep	Low	Low Fertility Droughtiness
Grove gravelly sandy loam (Gk, 5 to 15 percent slopes)	Somewhat Excessive	None	Low	Gravel	Moderately Deep	Low	Low Fertility Droughtiness Steep Slopes
Hoodsport gravelly sandy loam (He, 15 to 30 percent slopes)	Good	None	Low	Compact Till	Moderately Deep	Very Low	Low Fertility Droughtiness Slopes
Hoodsport gravelly sandy loam (Hf, 30 to 45 percent slopes)	Good	None	Low	Compact Till	Moderately Deep	Very Low	Low Fertility Droughtiness Slopes
Juno Sandy Loam (Jd, 0 to 3 percent slopes)	Somewhat Excessive	Occasionally in winter	Low	Gravel	Shallow	Medium to Low	Low Fertility Droughtiness
Riverwash (Ra, 0 to 3 percent slopes).	This land type consists of poorly assorted sand, gravel, and cobbles bordering streams and rivers or recently abandoned river channels. The soil unit is frequently overflowed and changed by erosion and deposition. The material is sparsely covered with vegetation, and it has no agricultural value.						

(1) Refers to capacity to supply moisture that plants can use
(2) Refers to the depths to which roots can readily penetrate: Moderately Shallow (20 to 36 inches) and Moderately Deep (36 to 60 inches)
SOURCE: Soil Survey, Mason County Washington, Soil Conservation Service, September 1960.

Rc
Tb
1
He
Gk
Gh
3
Hf
4
Hd
Hg
Me
Dow
Mf
Gh
He
Gk
Nb
Le
Nb
Tb
9
Gk
Tb
Le
Gb
Gm
Hf
Gk
LOWER DAM
Gh
16
Hf
Gk
Le
Ne
Nordby Lake
Gk
10
Hd
Hd
15
Gm
Gk
Hf
Finch
Ra
Creek
Hoodsport
Gh
Jd
Gk
Hf
He
Tb
Cc
Hd
Hf
11
He
Hd
Hf
Clark
Creek
Mg
2
He
Tb
Hd
He
Hd
He
12
Gk
Hf
Gk
Gp
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Tb
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Gk
Gk

Seismic Hazard Areas

Conditions that precipitate earthquake damage include settlement and soil liquefaction due to seismic activity. Soils within these areas contain high organic matter (e.g., wetland soils), loose sand and gravel, artificial fills, landslide deposits, and fine-grained soils with high water tables. The critical areas regulations do not allow public buildings in seismic hazard area. The Comprehensive Plan did not identify seismic hazard areas within the Finch Creek area.

Erosion Hazard Areas

Erosion hazard is generally due to water draining or running over land, picking up soils or pollutants in its path. Erosion can be accelerated through a combination of factors such as man-made impervious surfaces (roofs and driveways), disturbed or exposed soils, newly cleared or graded land, non-vegetated streambanks, or natural-geologic processes. Erosion hazards typically occur on erosive soils with slopes that exceed 15 percent. The steeply sloped, relatively unconsolidated deposits along Finch Creek are susceptible to excessive erosion during heavy rainfall. Aerial photos show a sizable buildup of sediment at the mouth of the creek, indicating the transport of sediment down the creek.

FLOOD PLAINS

The Federal Emergency Management Agency (FEMA) flood plain map for the Hoodsport area of Mason County indicates that the populated portion of Finch Creek lies within the 100-year flood plain (see Figure 2-5).

WETLANDS

The identified wetland areas in the Hoodsport area are shown in Figure 2-6. As the figure indicates, there are no designated wetland areas in the upper reaches of the Creek. However, the mouth of the creek discharges to a beach area on Hood Canal. This beach has a wetland designation as an "estuarine intertidal aquatic bed" (E2AB) with further classification as "semipermanently flooded" (F), "subtidal" (L), and "regularly flooded" (N).

The coastal beach that parallels Highway 101 to the north and south of Hoodsport is designated as "estuarine intertidal unconsolidated shore" (E2US). The beach to the south has an added designation as "regularly flooded" (N).

SURFACE WATER RESOURCES

Finch Creek and Hood Canal are the two primary surface waters found in the immediate study area for this report. Lake Cushman and the Skokomish River are also found in the vicinity of Hoodsport but are not included within the study area.

Hood Canal

Surface water bodies are classified in state regulation according to their identified characteristic beneficial uses. Chapter 173-201A Washington Annotated Code (WAC), the Water Quality Standards for Surface Waters, classifies Hood Canal as Class AA, Extraordinary.

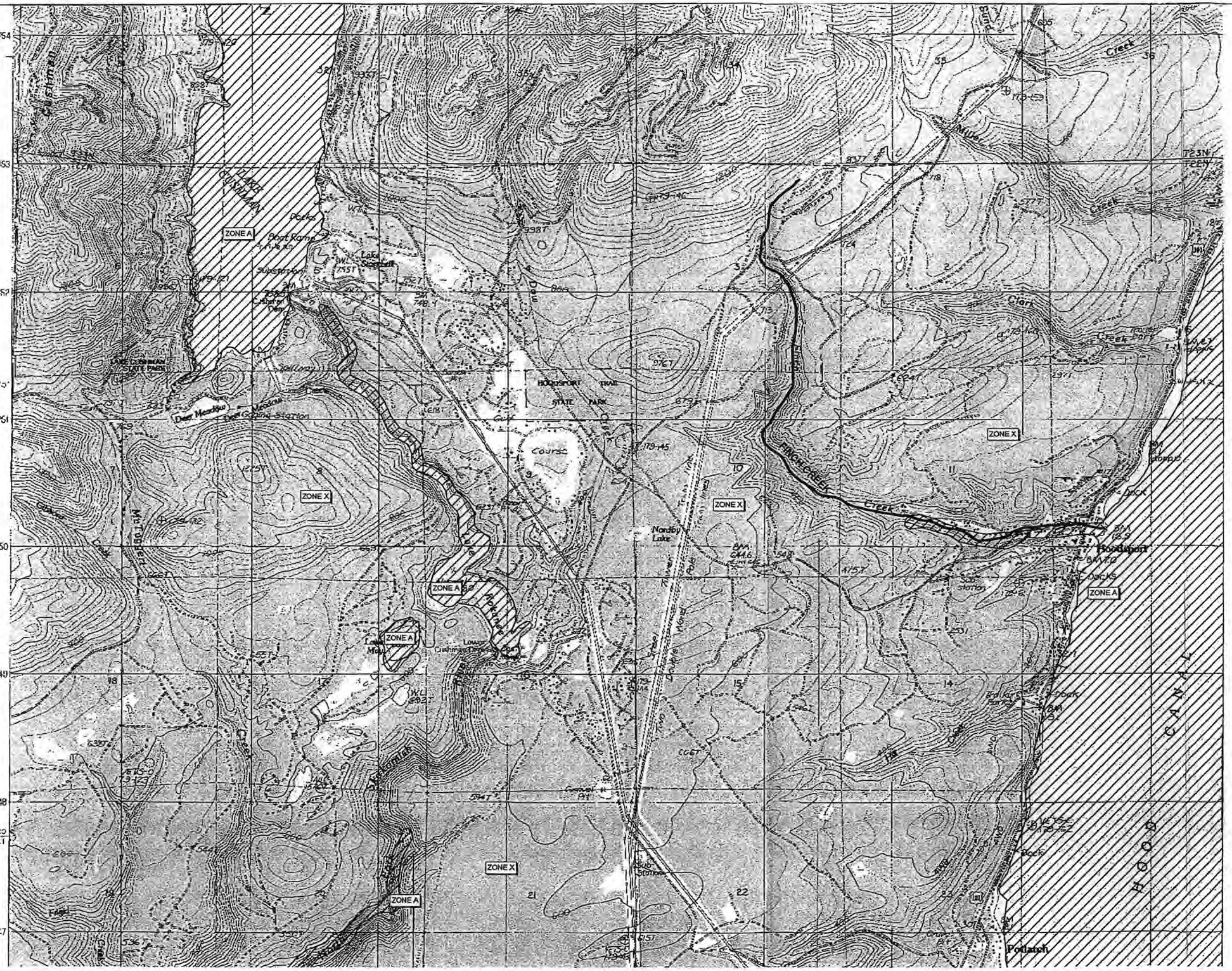
Designated uses for Class AA waters are: water supply and stock watering (freshwater), fish and shellfish migration, rearing, spawning and harvesting, wildlife habitat, primary contact recreation, sport fishing, boating and aesthetic enjoyment, and commerce and navigation. The surface water classifications have established minimum water quality requirements to maintain designated beneficial uses. The water quality standards for Class AA marine waters are presented in Table 2-4.

TABLE 2-4

Class AA Marine Water Quality Standards

Parameter	Standard
Dissolved oxygen[1]	>7.0 mg/l
Temperature[1]	≤13.0° C
PH[1]	Within range of 7.0 to 8.5
Fecal coliform	≤14 colonies/100ml
Turbidity[1]	≤5 NTU
Total dissolved gas	≤ 110 percent of saturation
Toxic, radioactive, or deleterious material concentrations shall be below those which have the potential, either singularly or cumulatively, to adversely affect characteristic water uses, cause acute or chronic conditions to the most sensitive biota dependent upon those waters, or adversely affect public health.	
Aesthetic values shall not be impaired by the presence of materials or their effects, excluding those of natural origin, which offend the senses of sight, smell, touch, or taste.	

(1) When natural conditions cause non-compliance with the water quality standard, limited impacts due to human-caused activities are allowable based on Chapter 173-201A-030.


ZONE A
ZONE X
Boat Ramp
Picnic
Shelter
Dam
Lake Cushman
Lake Cushman State Park
Deer Meadow
Deer Creek Station
Hoodsport Trail State Park
Golf Course
Finch Creek
Nandoy Lake
Hoodsport
Potlatch
Clark Creek
Hood Canal
Docks
Dock
Boat Launch
Pump Station
Lower Cushman Dam
Upper Cushman Dam
Lower Mud
Upper Mud
Cushman Creek
Mc Dougall Creek
Fishman Creek
Dow Creek
Bunn Creek
Mud Creek
Tacoma Pump Line
Tacoma Pump Line Road
Tacoma Pump Line Trail
Tacoma Pump Line Road Trail
Cushman Road
Hoodsport Road
Pump Station
Gate
Bridge
Pole
Tacoma Pump Line Road Trail
T23N
T22N
R1W
R2W
R3W
R4W
R5W
R6W
R7W
R8W
R9W
R10W
R11W
R12W
R13W
R14W
R15W
R16W
R17W
R18W
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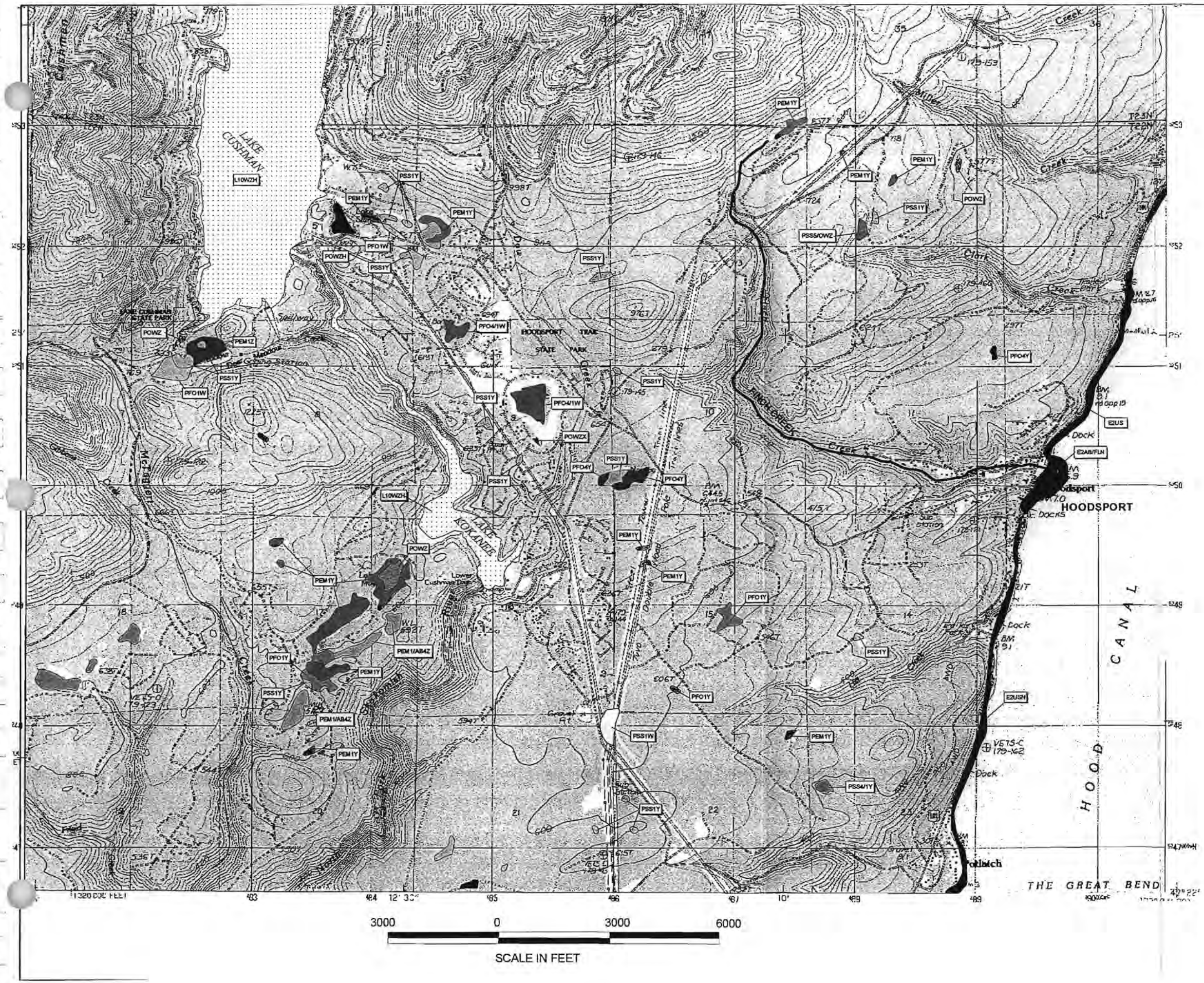
R40W



N

LEGEND

FINCH CREEK

WETLAND CLASSIFICATION

E2AB/FLN

E2USN

MASON COUNTY PUD No. 1

FINCH CREEK WASTEWATER
FEASIBILITY STUDY
NATIONAL WETLAND INVENTORY MAP
FIGURE 2-6

Gray & Osborne, Inc.
CONSULTING ENGINEERS

Finch Creek

Finch Creek is not specifically classified in the Water Quality Standards. However, the regulation states that all unclassified surface waters that are tributaries to Class AA waters are classified as Class AA. Therefore, Finch Creek would also be designated as a Class AA water body. The Class AA water quality standards for freshwater are presented in Table 2-5.

TABLE 2-5

Class AA Freshwater Water Quality Standards

Parameter	Standard
Dissolved oxygen[1]	>9.5 mg/l
Temperature[1]	≤16.0° C
PH[1]	Within range of 6.5 to 8.5
Fecal coliform	≤50 colonies/100ml
Turbidity[1]	≤5 NTU
Total dissolved gas	≤ 110 percent of saturation
Toxic, radioactive, or deleterious material concentrations shall be below those which have the potential, either singularly or cumulatively, to adversely affect characteristic water uses, cause acute or chronic conditions to the most sensitive biota dependent upon those waters, or adversely affect public health.	
Aesthetic values shall not be impaired by the presence of materials or their effects, excluding those of natural origin, which offend the senses of sight, smell, touch, or taste.	

(1) When natural conditions cause noncompliance with the water quality standard, limited impacts due to human-caused activities are allowable based on Chapter 173-201A-030.

As part of its ambient monitoring program, the Department of Ecology has collected periodic data near the Highway 101 bridge on Finch Creek to assess its overall water quality based on Class AA standards. These data, which also include nutrients such as nitrogen and phosphorous, are presented in Table 2-6. The Ecology data indicate that, with the exception of fecal coliform, Finch Creek meets Class AA water quality standards. Ecology considers the creek impaired due to the exceedances of the standard for fecal coliform

AIR QUALITY

The Olympic Air Pollution Authority (OAPA) is the local air quality monitoring agency for the Hoodsport area. The OAPA reports that air quality in the Hoodsport area is considered to be within ambient air quality standards. During winter months, however,

TABLE 2-6

Finch Creek Water Quality Data

Date	Temp (°C)	Flow (cfs)	Conduct. (umho/cm)	Dissolv. Oxygen (mg/L)	Oxygen Saturation (%)	pH	TSS (mg/L)	Total N (mg/L)	Ammonia (mg/L)	Dissolv. Phosph. (mg/L).	Ortho Phosph. (mg/L)	Turbid (NTU)	Fecal Coli (#/100ml)	Nitrate/ Nitrite (mg/L)
10/93	8.2		82	12	98.9	7.5	2	0.249	0.013	0.028	0.024	1	19	0.166
11/93	8		84	11.2	94.8	7.7	3	0.243	0.013	0.032	0.022	2.1	240	0.179
12/93	6.7		56	12	95.9	7.4	3	0.335	0.027	0.021	0.016	0.9	13	0.298
01/94	7.7		67	11.8	98.6	7.7	1	0.242	0.011	0.017	0.015	0.5	2	0.2
02/94	6.4		62	12	97	7.3	2	0.263	0.010	0.015	0.013	0.9	9	0.213
03/94	6.8		71	12.1	98.2	7.8	1	0.131	0.010	0.010	0.015	0.5	2	0.114
04/94	7.8		67	11.7	98.4	7.9	1	0.088	0.01	0.024	0.014	0.5	10	0.093
05/94	9.6		79	11.7	101.2	7.8	1	0.107	0.010	0.010	0.017	0.5	11	0.079
06/94	9.3		84	11.4	97.7	7.7	1	0.128	0.010	0.018	0.018	0.5	5	0.103
07/94	10.4		74	11.2	98.7	7.6	1	0.136	0.010	0.024	0.020	0.5	18	0.096
08/94	10.6		85	11.7	104	7.3	1	0.165	0.020	0.030	0.027	0.6	15	0.090
09/94	11.4		88	11.7	105.4	7.9	1				0.015	0.5	035	0.053
10/98	8	1.5	83	10.9	92.1	7.5	1	0.093	0.010	0.026	0.010	0.5	44	0.078
11/98	7.4	35	62	8.7	72	7.6	13	0.642	0.131	0.033	0.016	2.9	5	0.424
12/98	4.2	15	72	12.4	92.5	8	1	0.18	0.023	0.018	0.013	1	85	0.155
01/99	5.9	14	66	12	95	7.7	1	0.166	0.010	0.018	0.016	0.8	130	0.146
02/99	5.6	37	40	11.7	92.5	7.6	3	0.122	0.025	0.019	0.009	2	3	0.11
03/99	6.7	39	67	11.6	94.6	8	1	0.127	0.010	0.038	0.022	0.8	10	0.09
04/99	8.1	22	81	11.5	96.7	8.1	1	0.064	0.010	0.035	0.019	0.5	6	0.063
05/99	8.3	13	78	11.7	99.2	7.8	1	0.104	0.03	0.044	0.020	0.5	27	0.071
06/99	7.9	21	88	11.5	96	7.7	2	0.111	0.010	0.040	0.021	0.5	24	0.067
07/99	8.7	9	74	11.7	99.9	7.9	2	0.107	0.026	0.046	0.020	1.2	48	0.073
08/99	9.6	1.4	75	11.3	98.6	7.9	1	0.09	0.010	0.043	0.022	0.5	86	0.05
09/99	8.7	1.5	77	11.5	97.1	7.5	1	0.098	0.031	0.043	0.021	0.5	6	0.074

smoke from wood stoves could exceed the standards depending on meteorological conditions. (Personal Conversation, Mr. Jim Werner, Monitoring Technician, OAPA, June 20, 2000). There are no air quality monitoring stations located within the Hoodsport area, the closest stations are in Shelton.

OUTDOOR RECREATION

Hoodsport enjoys outdoor recreational opportunities from east Hood Canal, Lake Cushman, and Olympic National Forest, which ~~all~~ lie to the west. Hood Canal offers marine water recreational activities such as beach combing, shellfish harvesting, fishing, boating and sailing. Lake Cushman provides fresh water activities such as fishing, swimming, boating, hiking, and camping. Recreational facilities on and around the lake include Lake Cushman State Park and the ONF Staircase Ranger Station. Hoodsport Trail State Park also is located along Lake Cushman Road. In addition, the Potlatch State Park is located approximately three miles south of Hoodsport on the water side of Highway 101.

PUBLIC UTILITIES

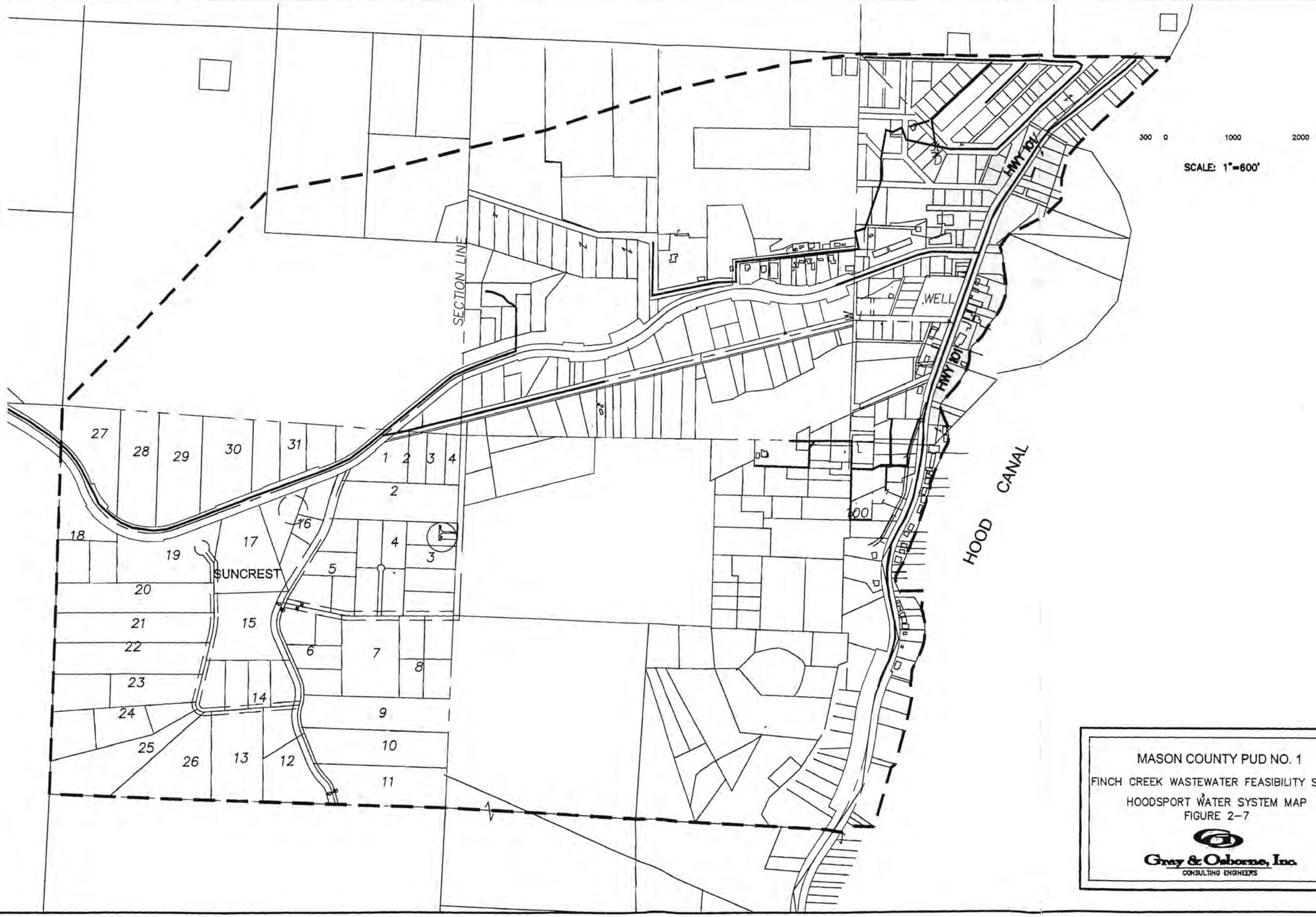
Public utilities in the Hoodsport area include water, power, and telephone. Currently, there is no community wastewater collection in the area. Telephone service to the Hoodsport area is provided by US West Communications. Mason County PUD No. 1 supplies both electrical power and water service to the community. The electrical service area for PUD No. 1 extends from the Potlatch area north along the Canal for approximately 50 miles and east along the south shore of Hood Canal near Twanoh State Park. Mason County PUD No. 1 provides power to approximately 4,200 customers.

The PUD No. 1 manages a large number of water systems in the county, including the Hoodsport/Suncrest water system that provides water supply too much of Hoodsport as well as the Suncrest development. A number of homes around Hoodsport, including those along the upper portion of Finch Creek Road, are served by private drinking water wells.

The PUD No. 1 has operated the Hoodsport Water System since 1995. Prior to that time it was known under a different name and owned and operated by a private owner. The Washington Department of Health (DOH) had put the system into receivership and requested the PUD No. 1 to take over management due to serious the operational and water quality problems. The PUD No. 1 eventually purchased the system and since 1996 has been implementing a series of improvements to bring the system into compliance with state requirements. Phases 1 and 2 of the planned improvements have been completed; a third phase of work is yet to be completed. The Hoodsport Water System serves 186 customers, 165 from the original Hoodsport system and 21 from the Suncrest system, and covers an area of approximately 650 acres. A layout of the Hoodsport Water System is presented in Figure 2-7.

Wastewater is handled in the area by individual onsite septic systems. If a community wastewater collection and treatment system is developed for the Finch Creek/Hoodsport area, the PUD No. 1 would assume ownership and responsibility for the system. In 1996, the PUD No. 1 was authorized by a public vote to provide wastewater services in the area. The PUD No. 1 operates a recently constructed community onsite facility for the residents of the Madrona community on Hood Canal.

There are only a few wastewater facilities in Mason County, and only a couple proximate to the Hoodsport area. The City of Shelton owns and operates a wastewater system for its service area. The Washington Department of Corrections also owns and operates a facility for the Washington Corrections Center, located just north of Shelton. The Alderbrook Resort near Union operates a small privately owned wastewater facility that serves only the resort facilities. Construction began on a new water reclamation facility in the North Bay area in the spring of 2000. This facility will serve the communities of Allyn, Victor, and Lakeland Village. Start-up for the facility is expected in summer of 2001. The Skokomish Tribe has proposed a wastewater collection and treatment system for the reservation area, but has not yet been successful in moving the project forward.



MASON COUNTY PUD NO. 1

FINCH CREEK WASTEWATER FEASIBILITY STUDY

HOODSPORT WATER SYSTEM MAP

FIGURE 2-7

Gray & Osborne, Inc.

CONSULTING ENGINEERS

CHAPTER 3

CHAPTER 3

LAND USE AND POPULATION

This feasibility study is being developed for a limited area and population, and an exhaustive analysis of land use and population projections for the Hoodsport and surrounding areas is beyond the scope of the report. However, a review of these issues is appropriate for consideration in the development of alternatives for wastewater collection and treatment for the Finch Creek area.

LAND USE

In the Mason County Comprehensive Plan and Development Regulations (Ordinance No. 82-96), the County has adopted Growth Management Act (GMA) land use designations in lieu of typical residential and non-residential zones. Land use planning within Mason County is organized by watershed drainage basins. For this study, the watershed of interest is West Hood Canal. The shorelines within the study areas are additionally regulated by the Mason County Shoreline Master Program. The following sections describe land use designations and general policies for wastewater treatment in the West Hood Canal watershed and for the shorelines within the watershed.

MASON COUNTY LAND USE DESIGNATIONS

For the purposes of regulating development activity within Mason County, the County has designated three general types of development areas. These development areas include urban growth areas (UGA); resource lands; and rural lands, of which there are several categories. These land use designations contain characteristics that have been identified in the Comprehensive Plan as worthy of preservation and/or enhancement. The purpose of these designations is to promote orderly development in a manner that is consistent with the Comprehensive Plan. Each development area type has been assigned a specific development density. A description of the three land use designations and their sub-types is presented below.

Urban Growth Areas

Urban growth areas are to accommodate the majority of the growth that is expected to occur within the County in the foreseeable future. These areas have urban characteristics and services, the widest variety of uses, and the highest densities that will be allowed. Within Mason County, only the City of Shelton and the communities of Belfair and Allyn have been designated as urban growth areas.

Resource Lands

Resource land is a term covering a variety of land use types that have value due to their soil types, ground cover, or capacity for mineral extraction. Lands with intrinsic aesthetic value are also included in this category. Examples of resource lands in Mason County include national park lands, national forest lands, long-term commercial forests, and mineral resource lands.

Rural Lands

The recent *Amendment to the Mason County Comprehensive Plan and Development Regulations, May 1, 2000* (Amendment) enacted a number of "performance districts" in order to regulate land use and provide an organizing structure for rural areas. The Rural Lands element of the County's Comprehensive Plan emphasizes the need to maintain the rural character of the County as it accommodates growth over the next 20-year planning horizon. Each performance district allows for a variety of uses and has a distinct set of performance standards for development. The Amendment updated the County's Rural Lands classifications with the following performance districts:

- Limited Areas of More Intensive Rural Development (LAMIRD)

    a) Rural Activity Centers (RAC)
    b) Hamlets
    c) Rural Commercial/Industrial Areas
    d) Rural Tourist/Recreational Areas

- Fully Contained Community (FCC)
- Master Planned Resort (7MPR)
- Rural Area (RA) with residential densities of 1 unit per 5, 10, or 20 acres

Rural Activity Centers

Hoodsport continues to be designated as a Rural Activity Center. The Amendment suggests Mason County plat new development for a residential density of 1 dwelling unit per 2.5 acres, with an average density of 1 dwelling unit per acre for new development.

The Amendments provide an updated description of the organization of a RAC, as follows:

> *Rural Activity Centers (RAC) include existing communities with an established settlement pattern. These communities include a mix of uses, typically on small lots. RACs also include concentrations of commercial, service, industrial, and civic uses but are not served by urban levels of*

> *facilities and services. Residential areas include small lot, single-family neighborhoods and some small-scale and low-rise multifamily housing. Businesses are typically found near the highway that runs through the community.*

The Amendment suggests a limited rate of growth will occur within RACs over the next 20 years. It is anticipated that growth within the RACs will result from an increase in "retail, commercial, tourism and industrial uses necessary to support the residential growth in the Rural Areas." The Amendment recommends concentration of growth within RACs in order to "protect the surrounding rural and resource lands from sprawling patterns of development." Development within isolated or remote rural locations is discouraged due to the difficulty of providing emergency services, such as fire and police protection or medical attention.

The amendment supports the needs of this study by the following acknowledgement:

> *Because of their past development and location next to water bodies, some RACs may require some means of collective wastewater treatment to protect water quality.*

The County's general land use policies for RAC areas state that new development should be served by adequate facilities for water supply, wastewater disposal, electricity and storm drainage. However, these facilities are not intended to be urban levels of service. New development within RACs should only be encouraged in areas where public services can be provided.

MASON COUNTY SHORELINE MASTER PROGRAM

Mason County is required by the Shorelines Management Act of 1971 to establish and maintain a shoreline management program. This legislation designated the shoreline areas of Hood Canal as Shorelines of Statewide Significance. The Mason County Shoreline Master Program (SMP) designated the shoreline environments as either urban residential, urban commercial, rural, conservancy, and natural (see Figure 3-1).

Urban Residential

Designation of shoreline areas as urban residential allows optimum utilization of the shoreline for residential development. This designation is used for areas that are currently under intensive development, as well as areas planned to accommodate future urban level growth. Clustering of dwelling units is encouraged in order to preserve natural features, minimize physical and visual impacts, and reduce utility and road costs. The minimum lot size for shoreline urban residential areas is 12,500 square feet (sf).

Urban Commercial

Shoreline uses designated as urban commercial typically involve wholesale or retail trade, restaurants, hotel/motel facilities, or other business activities. Commercial developments are encouraged to locate buildings, parking, or accessory structures inland from shorelines. Commercial development may be allowed along the shoreline only if the business is water dependent, water-related, or water-oriented. All other non-water dependent or oriented business activities will not be allowed to develop within or on shorelines.

Rural

The rural environment designation identifies shorelines that should be protected from urban expansion, particularly those in agricultural use. The intent of the designation is to restrict intensive development along undeveloped shorelines. Development is restricted to one residence per 20,000 sf. New development in the rural environment is to reflect the character of the surrounding area by limiting residential density, providing permanent open space, and by maintaining adequate building setbacks from the water to prevent shoreline resources from being destroyed.

Conservancy

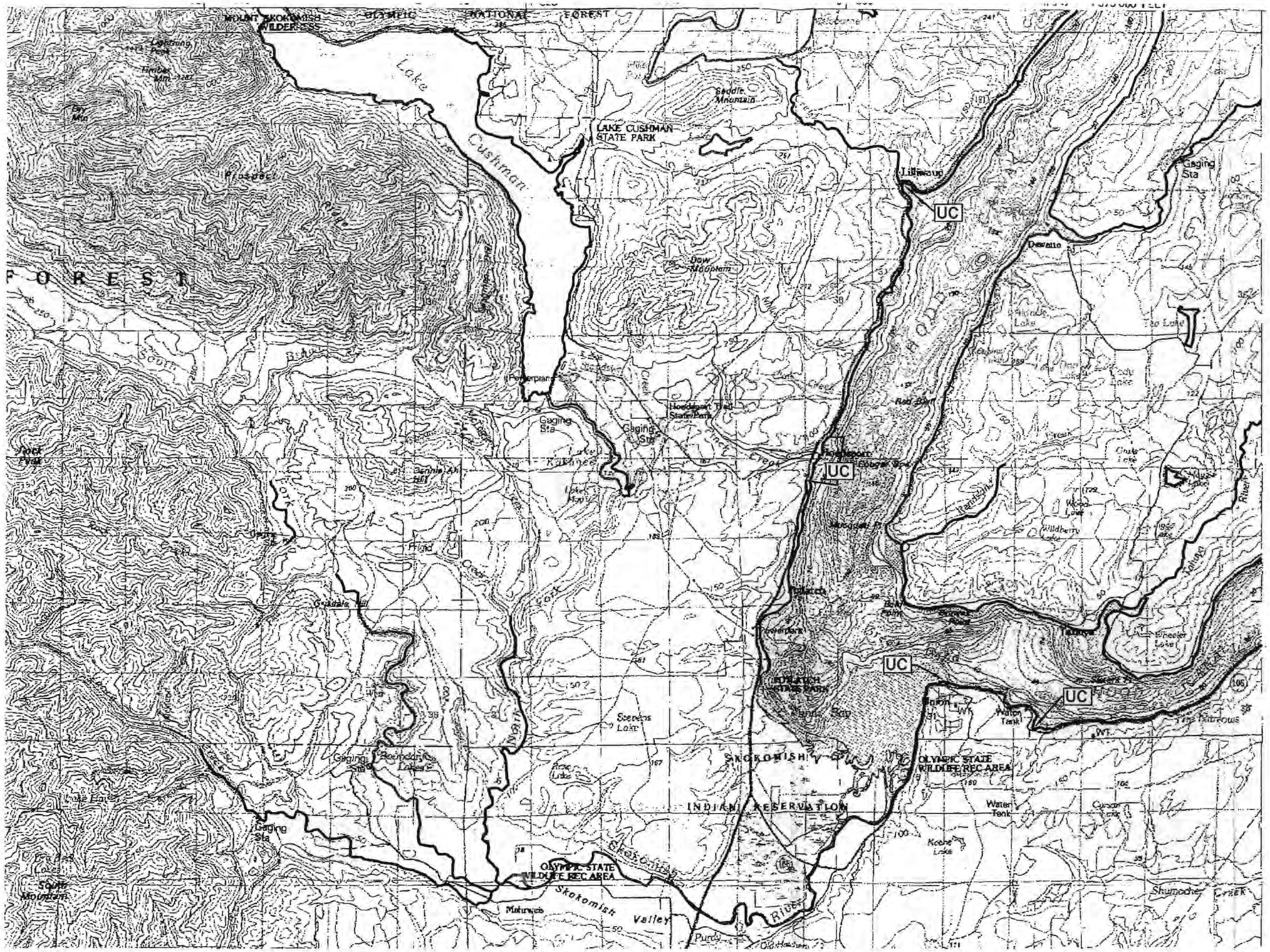
The conservancy environment designation is intended to protect, conserve and manage shorelines that contain existing natural resources and valuable historic and cultural areas. It is intended that these areas maintain their existing character through non-consumptive uses such as outdoor recreation, sustained-yield timber harvesting, and pasture and range land. Residential development is allowed at only one residential unit per 5 acres and 200 feet of shoreline.

Natural

The natural shoreline designation identifies undeveloped shorelines that should be protected from development. The natural designation is intended to keep natural shoreline systems free from human influence. The County's SMP holds these lands as "unique and valuable" and, therefore, seeks to "maintain them in a natural state".

LAND USE IN THE HOODSPORT AREA

As shown in Figure 3-2, the current land use in Hoodsport is primarily residential, with homes along the shoreline and Finch Creek, and in the upland above Highway 101. The community has a small commercial district along Highway 101 that includes a small hotel, a new Lumberman's store, a restaurant, and a tavern, as well as several smaller



0 1000 5000 8000

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LEGEND

FINCH CREEK

SHORELINE MASTER LANDUSE PROGRAM DESIGNATIONS :

- URBAN
- URBAN COMMERCIAL AND RESIDENTIAL (UC)
- RURAL
- CONSERVANCY
- NATURAL

MASON COUNTY PUD No. 1

FINCH CREEK WASTEWATER FEASIBILITY STUDY

FIGURE 3–1

SHORELINE MASTER PROGRAM LANDUSE DESIGNATIONS

Gray & Osborne, Inc.

CONSULTING ENGINEERS

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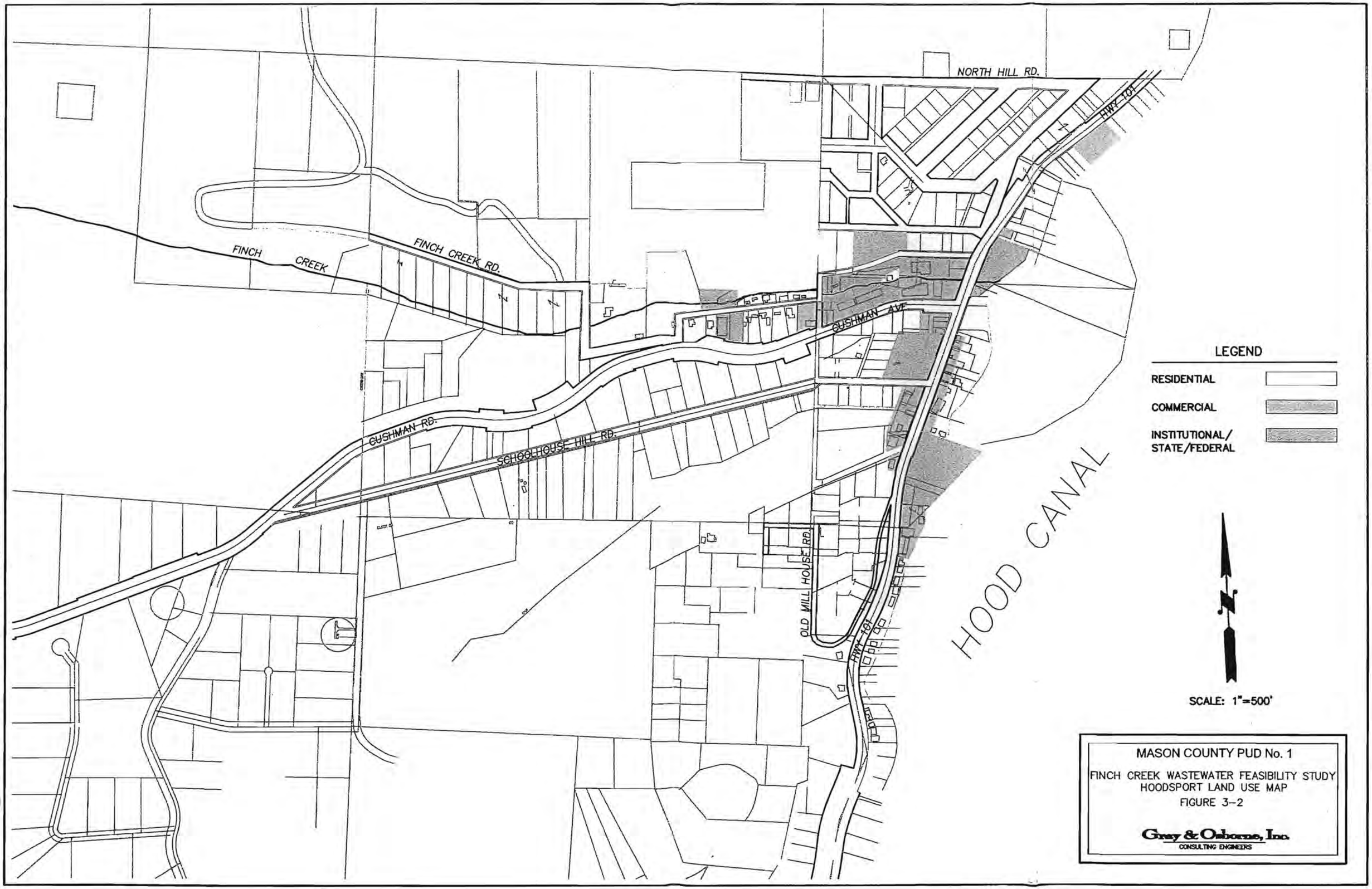

BY: MZ ATED: SEP 30 1996 10: 36: 49 UPDATED: AUG 23 2000 16: 16: 34 PLOTTED: 23 2000 16: 16: 56 FILE: M:\MASONCOPUD1\99680\FIGURES-FINAL\FIG3-2.

eating establishments. Hoodsport also has a Timberland Library, a WDFW fish hatchery and several facilities for the USDA Forest Service, including a visitors' center. There are no schools in Hoodsport.

In the Mason County Comprehensive Plan, the Hoodsport area has been designated as a Rural Activity Center (RAC), which was defined in the previous section. The County seriously considered designating the community as an Urban Growth Area, but at present it remains an RAC. The boundary of the RAC has also been a matter of significant discussion during development of the Comprehensive Plan. The current proposed boundary of the Hoodsport RAC is shown in Figure 3-3. It must be noted that this boundary is quite restrictive, excluding areas contiguous to the Hoodsport community that have a significant amount of existing residential development.

As defined, undeveloped areas within an RAC boundary can be developed, however, the recommended density is quite low at 1 unit per 2.5 acres. Lots platted prior to 1996 are not subject to this density requirement and may be able to develop at an average density of one dwelling unit per acre. The main area of undeveloped land within the designated RAC boundary is to the west of Old Mill House Road where there are as many as twenty vacant properties. There are also several undeveloped properties in the North Hill area.

The County has developed a matrix of allowable non-residential uses that is too extensive to reproduce here. Commercial development is restricted primarily to essential services, small-scale businesses, and resource-based and light industrial uses. The intent is to not allow a significant change in use or size from the type of businesses currently in existence in order to maintain the existing character of the community.

As for shoreline designation of the Hoodsport community, the Mason County Shoreline Master Program has designated US Highway 101 from Lilliwaup to Potlatch as "Urban Commercial". The Hoodsport shoreline is already heavily developed, and this designation is consistent with current development uses.

POPULATION

As stated previously, this feasibility study is being developed for a specifically defined area and population within the community of Hoodsport. It is not within the scope of the report to prepare an in-depth analysis of population projections and future design flows and loadings for a potential larger, expanded Hoodsport service area. Also, because it is not known how long Hoodsport will remain an RAC, versus becoming an Urban Growth Area, a detailed analysis would be speculative at this point.

However, a brief evaluation of current and future population within the RAC boundary and the contiguous Hoodsport community is appropriate for consideration in the development of this report. Such information will be beneficial to the Mason County

PUD No. 1 in evaluating considering the wastewater alternatives presented later in the report.

MASON COUNTY

Population data for Mason County over the past 10 years is presented in Table 3-1. As the table indicates, Mason County experienced a fairly consistent increase in population of 2 to 3 percent from 1990 through 1997, but in 1998 and 1999, the rate of growth decreased significantly to less than 1 percent. The County's population in 1990 was 38,341; by 1999 it had increased to 48,600, an overall increase of more than 25 percent over a nine-year period.

TABLE 3-1

Mason County Population Growth Rate (1990-1999)

Year	Population	Population Increase	Rate of Increase
1990	38,341		
1991	39,900	1,559	3.91%
1992	41,200	1,300	3.16%
1993	42,900	1,700	3.96%
1994	44,300	1,400	3.16%
1995	45,300	1,000	2.21%
1996	46,700	1,400	3.00%
1997	47,900	1,200	2.51%
1998	48,300	400	0.83%
1999	48,600	300	0.62%

Source: Mason County Dept. of Planning

Much of the increase was due to residential growth, while overall economic growth was more limited. Much of the residential growth is believed to be due to retirees locating to the area, as well as Pierce, Thurston, and Kitsap County commuters seeking less expensive housing.

HOODSPORT

Historical population information specific to the Hoodsport community is limited. Hoodsport is currently an unincorporated community with no established boundary. It has been designated as an RAC, with the proposed boundary as shown in Figure 3-3. Population within the proposed RAC boundary was estimated using land use information provided by Mason County. Based on a review of the type of residential development, e.g. one bedroom, two bedroom, etc., an estimated average of 2.2 persons per household

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was developed. There are approximately 150 homes within the proposed RAC, resulting in and estimated current population of 330 persons.

County records indicate approximately 60 undeveloped property parcels within the designated RAC boundary. Many of these parcels are believed to be double lots and parcels not appropriate for development, e.g. too steep or too small. A conservative assumption is that 60 percent of these properties will be developed with one residence within 20 years or by the year 2020. Based on an average of 2.2 persons per household, the estimated 2020 population for the proposed RAC area would be 410 persons.

The proposed RAC boundary does not encompass all of the developed areas contiguous to the Hoodsport community. For example, the boundary excludes the upper portion of Finch Creek Road, part of the area of concern being addressed in this report. Therefore, a second population estimate was developed for an area that includes these additional developed areas that are contiguous to Hoodsport. This "greater Hoodsport" community area includes the proposed RAC area, all of Finch Creek Road, the development along Cushman and Schoolhouse Hill Roads to the point of their intersection, and shoreline development to and including the Hoodsport Winery. The current population for this larger area was estimated to be 410 persons, using the process previously discussed. There are approximately 70 vacant properties in this larger area. If it is assumed that 50 percent of the vacant properties will be developed for residences within 20 years, at an average of 2.2 persons per household, the estimated population of the greater Hoodsport community in 2020 will be 490 people. A summary of the population estimates is provided below in Table 3-2.

TABLE 3-2

Hoodsport Estimated Population

	2000	2020
Hoodsport RAC	310	410
Greater Hoodsport	410	490

There are sizable parcels and tracts of undeveloped land surrounding the Hoodsport community. If land use designations change, and these large tracts of land are developed, the population and the overall character of the area will change significantly. As previously stated, to attempt to analyze these impacts is well beyond the scope and intent of this study.

CHAPTER 4

CHAPTER 4

REGULATIONS AND DESIGN GUIDELINES

Within the state of Washington, wastewater treatment systems are regulated at three different levels based on their peak day design flow and the complexity of the treatment process. Treatment systems with a design flow less than 3,500 gallons per day (gpd), regardless of the means of treatment, are the responsibility of the owner and are regulated by the county or local health department. Individual and larger onsite systems designed for peak day flows between 3,500 gpd and 14,500 gpd are regulated by the Washington Department of Health (DOH).

Mechanical treatment facilities with a design flow greater than 3,500 gpd and any type of treatment system with a design flow greater than 14,500 gpd are under the regulation of the Washington Department of Ecology (Ecology). For each of these levels, specific standards and regulations apply. These standards and regulations are discussed in the following sections.

ONSITE SEWAGE SYSTEMS

Chapter 246-272 WAC (Washington Annotated Code) is the primary regulation governing onsite sewage treatment systems in the state of Washington. The purpose of the regulation is to protect public health by minimizing effects to the following circumstances:

- The potential for public exposure to sewage from onsite systems;
- The adverse effects that on-site sewage discharge may have on surface water and ground water resources.

In the regulation, onsite sewage systems (OSS) are defined as a system that provides subsurface soil treatment and disposal on or near the property where it originates for residences or facilities that are not connected to a public sewer system. Chapter 246-272 addresses both individual onsite systems and large onsite systems.

The Mason County Department of Health Services implements the regulation at the local level, overseeing systems with a design flow up to 3,500 gpd. Large onsite systems (LOSS) are regulated at the state level by DOH. Large onsite systems are defined in the regulation as onsite systems that have a design flow, at any common point, greater than 3,500 gpd. As stated above, any system with a design flow greater than 14,500 gpd is regulated by Ecology.

INDIVIDUAL ONSITE SEWAGE SYSTEMS

Chapter 246-272 WAC

Chapter 246-272 WAC presents the minimum design requirements for onsite systems with regard to design flow, horizontal and vertical separation, soil and site evaluation, loading rate, and minimum lot sizes. The regulation also addresses the responsibilities of the state and local authorities with regard to system installation, inspection, operation and maintenance, and repairs of existing systems.

Design Flow

The design flow for a residential onsite system must be based on 120 gallons per day (gpd) per bedroom. However, the minimum design flow for any system must be at least 240 gpd, unless technical justification is provided to support approval of a lower design flow. This design flow is considered to be a peak day flow for the system, and all components must be sized to transport and treat this amount of flow. It is critical that extraneous water from surface runoff, footing drains, roof drains, and other non-sewage flows be kept from the onsite system components and the areas where the system is located. Excessive flows into the system will limit treatment effectiveness and greatly increase the potential for system failure.

Design flows for non-residential facilities should be determined according to the *"Design Manual: Onsite Wastewater Treatment and Disposal Systems,"* (USEPA, EPA-625/1-80-012, October, 1980); the *"Design Standards for Large Onsite Sewage Systems,"* (DOH, December 1993); or *"Criteria for Sewage Works Design,"* (Ecology, December 1998), as appropriate.

Horizontal Separation

To reduce the potential for impacts to public health and surface and ground waters, various onsite system components are required to be located minimum distances from drinking water sources such as wells and springs, surface waters, drainage courses, building foundations, and easement and property lines.

Table 4-1 presents the minimum horizontal separation distances required under Chapter 246-272 WAC. These requirements may be increased if a condition exists that creates a greater potential for contamination or pollution. Such conditions include excessively permeable soils, unconfined aquifers, shallow or saturated soils, dug wells, and improperly abandoned wells. The horizontal setback requirements may be decreased if certain specified protective physical conditions exist and enhanced treatment beyond a conventional system is provided.

TABLE 4-1

Minimum Horizontal Separations for Residential Onsite Systems

Items Requiring Setback	From Edge of Disposal Component and Reserve Area	From Septic Tank, Holding Tank, Containment Vessel, Pump Chamber and Distribution Box	From Building Sewer, Collection, and Nonperforated Distribution Line[1]
Non-public well or suction line	100 ft.	50 ft.	50 ft.
Public drinking water well	100 ft.	100 ft.	100 ft.
Public drinking water spring[2]	200 ft.	200 ft.	100 ft.
Spring or surface water used as drinking water source[2,3]	100 ft.	50 ft.	50 ft.
Pressurized water supply line[4]	10 ft.	10 ft.	10 ft.
Properly decommissioned well[5]	10 ft.	NA	NA
Surface[3]			
Marine water	100 ft.	50 ft.	10 ft.
Fresh water	100 ft.	50 ft.	10 ft.
Building foundation	10 ft.[6]	5 ft.[6]	2 ft.
Property or easement line[6]	5 ft.	5 ft.	NA
Interceptor/curtain drains/drainage ditches			
Downgradient[7]	30 ft.	5 ft.	NA
Upgradient[7]	10 ft.	NA	NA
Downgradient cuts or banks with at least 5 ft. of original, undisturbed soil above a restrictive layer due to a structural or textural change	25 ft.	NA	NA
Downgradient cuts or banks with at least 5 ft. of original, undisturbed soil above a restrictive layer due to a structural or textural change	50 ft.	NA	NA

1. "Building sewer" as defined by the most current edition of the Uniform Plumbing Code. "Nonperforated distribution" includes pressure sewer transport lines.
2. If surface water is used as a public drinking water supply, the designer shall locate the OSS outside of the required sanitary control area.
3. Measured from the ordinary high water mark.
4. the local health officer may approve a sewer transport line within ten feet of a water supply line if the sewer line is constructed in accordance with section 2.4 of the Department of Ecology's *"Criteria for Sewage Works Design," revised October 1985,* or equivalent.

5. Before any component can be placed within one hundred feet of a well, the designer shall submit a "decommissioned water well report" provided by a licensed well driller, which verifies that appropriate decommissioning procedures noted in chapter 173-160 WAC were followed. Once the well is properly decommissioned, it no longer provides a potential conduit to ground water, but septic tanks, pump chambers, containment vessels, or distribution boxes should no be places directly over the site.
6. The local health officer may allow a reduced horizontal separation to not less than two feet where the property line, easement line, or building foundation is upgradient.
7. The item is downgradient when liquid will flow toward it upon encountering a water table or a restrictive layer. The item is upgradient when liquid will flow away from it upon encountering a water table or restrictive layer.

Soil and Site Evaluation

Evaluation of the proposed treatment site and the soils on the site is very critical to the design of the onsite system. The regulation requires that this evaluation be completed by an engineer, a qualified onsite system designer, or a soil scientist. The evaluation must include a sufficient number of soil logs to determine the classification of soils in primary and reserve disposal areas and the presence of a restrictive layer within five feet of the surface. Table 4-2 presents the soil classification information used to evaluate soils for onsite sewage systems.

TABLE 4-2

Soil Textural Classification for Onsite Systems

Soil Type	Soil Classification Description
1A	Very gravelly[1] coarse sands or coarser, all extremely gravelly soils[2].
1B	Very gravelly medium sand, very gravelly find sand, very gravelly very fine sand, very gravelly loamy sands.
2A	Coarse sands (also includes ASTM C-33 sand)
2B	Medium sands
3	Fine sands, loamy coarse sands, loamy medium sands
4	Very fine sands, loamy fine sands, loamy very fine sands, sandy loams, loams.
5	Porous, well developed structure in silt and silt loams
6	Other silt loams, silty clay loams, clay loams
Unsuitable	Sandy clay, clay, silty clay, and strongly cemented or firm soils.

1. Very gravelly + >35% and <60% gravel and coarse fragments, by volume.
2. Extremely gravelly = >60% gravel and coarse fragment, by volume.

The site evaluation must determine the ground water conditions and the expected maximum ground water table level. The proposed site also must be reviewed for

drainage, slide and erosion potential, flood potential, cuts, banks, fill areas, and existing appurtenances such as wells, surface water, and existing utilities.

Vertical Separation

Vertical separation is defined by Chapter 246-272 WAC as follows:

> "...means the depth of unsaturated, original, undisturbed soil of soil types 1B-6 between the bottom of a disposal component and the highest seasonal water table, a restrictive layer, or soil type 1.A..."

The vertical separation that exists between the bottom of the disposal component and the high ground water table, restrictive layer, or a Type 1A soil, along with soil classification and horizontal separation requirements, have a significant bearing on the level of treatment and type of disposal required. The effects of vertical separation and soil classification on treatment and effluent distribution requirements are presented in the Table 4-3.

TABLE 4-3

Onsite System Treatment and Effluent Distribution Requirements for Various Soil Types and Depths

	Vertical Separation			
Soil Type	<1 Foot	≥ 1 Foot to < 2 Feet	≥ 2 Feet to < 3 Feet	≥ 3 Feet
1A	Not allowed	Pressure Distribution[1,2]	Pressure Distribution[1]	Pressure Distribution[1]
2A	Not allowed	Pressure Distribution[1,2]	Pressure Distribution	Pressure Distribution
1B-6	Not Allowed	Pressure Distribution[1,2]	Pressure Distribution	Gravity Distribution

1. System meeting Treatment Standard 2 required.
2. Mound systems installed where the original, undisturbed, unsaturated soil depth is between twelve and eighteen inches require pretreatment by an intermittent sand filter.

As indicated in Table 4-3, onsite systems are not allowed where less than one foot of vertical separation exists. Where less than 3 feet of vertical separation exists and where soil types 1A and 2A are found, pressure distribution of effluent is required. Also, where only 1 to 2 feet of separation exists and wherever soil type 1A is found, enhanced treatment beyond that provided by a septic tank is required. The treatment required must meet or exceed Treatment Standard 2.

Treatment Standards 1 and 2, as defined in Chapter 246-272 WAC, indicate treatment systems that produce an effluent with less than 10 milligrams per liter (mg/l) of biochemical oxygen demand (BOD_5) and total suspended solids (TSS). Treatment Standard 1 must have less than 200 fecal coliforms per 100 milliliters (FC/100ml); Treatment Standard 2 must have less than 800 FC/100ml. The treatment requirements are based on a monthly average. If mound systems are used to provide treatment and additional separation where the vertical separation is only 12 to 18 inches, an intermittent sand filter must be used for pretreatment in order to meet the treatment requirements.

Loading Rate

The volume of effluent that can be applied to a disposal area is determined by the soil classification and, for soil type 1A, also by the type of treatment provided. Table 4-4 indicates the maximum loading rates per soil type for domestic sewage disposal. This table can be used to calculate the total soil absorption area required to be provided for the given design flows. The available soil absorption area is based on the total area of the bottom of the disposal trench.

TABLE 4-4

Maximum Hydraulic Load Rate for Residential Sewage Onsite Systems[1]

Soil Type	Loading Rate (gal/sf/day)
1A	Varies according to system selected to meet Treatment Standard 2[2]
1B	Varies according to soil type of non-gravel portion[3]
2A	1.2
2B	1.0
3	0.8
4	0.6
5	0.45
6	0.2

1. Compacted soils, cemented soils, and/or poor soil structure may require a reduction of the loading rate or make the soil unsuitable for conventional OSS systems.
2. Due to the highly permeable nature of type 1A soil, only alternative systems that meet or exceed Treatment Standard 2 can be installed. However, a conventional gravity system may be used if it meets all criteria listed under Chapter 246-272-11501(2)(h). The loading rate for these systems is provided in the appropriate guideline.
3. The maximum loading rate listed for the soil described as the non-gravel portion is to be used for calculating the absorption surface area required. The value is to be determined from this table.

Soil type 1A is extremely coarse and gravelly, resulting in a very high permeability. Because soil type 1A is so permeable, it provides minimal soil treatment, and therefore, requires that enhanced treatment be provided. The soil classification ascension from Type 1A to Type 6 indicates soils that are increasingly less permeable. Table 4-4 demonstrates that, as the soil type becomes less permeable, the allowable loading rate decreases. The decreased loading rate results in an increasing area requirement for absorption.

Repair and Replacement of Existing Systems

When existing onsite systems fail and the proposed repair or replacement system cannot conform to the previously discussed requirements, Chapter 246-272 WAC provides for a means of repair or replacement through enhanced treatment requirements. The enhanced requirements are found in Table VI of the regulation and are, therefore, commonly referred to as "Table 6 repairs." These requirements are presented in Table 4-5.

TABLE 4-5

Onsite System Requirements for Repair or Replacement of Disposal Components Not Meeting Vertical and Horizontal Separations[1,2]

Vertical Separation in Feet	Horizontal Separation in Feet[3]		
	<25	25-50	>50-<100
<1	Treatment standard 1	Treatment standard 1	Treatment standard 2[4]
1-2	Treatment standard 1	Treatment standard 2[4]	Pressure Distribution
>2	Treatment standard 2[4]	Pressure Distribution	Pressure Distribution

1. The treatment standards refer to effluent quality before discharge to unsaturated, subsurface soil.
2. The local health officer may permit ASTM C-33 sand to be used as fill to prevent direct discharge of treated effluent to ground water, surface water or upon the surface of the ground.
3. The horizontal separation indicated is the distance between the disposal component and the surface water, well, or spring. If the disposal component is upgradient of a surface water, well or spring to be used as a potable water source, the next higher standard level of treatment shall apply unless treatment standard 1 is already being met.
4. Mound systems are not allowed to meet treatment standard 2.

The horizontal separation requirements presented in Table 4-5 refer to the distance to surface water, a well, or a spring. If the surface water, well, or spring is used as a potable water source, then the next higher standard of treatment is required. Treatment Standard 1 was previously defined as a system that produces an effluent with less than 10 mg/l of BOD_5 and TSS and less than 200 FC/100ml, based on a monthly average. Treatment

Standard 2 has the same requirement for BOD_5 and TSS, but a less stringent fecal coliform requirement of less than 800 FC/100ml.

Minimum Land Area

Chapter 246-272 WAC also contains minimum land requirements for new developments and subdivisions to be served by onsite systems. These minimum land area requirements are based on soil type and the proposed source of drinking water supply. Table 4-6 presents the minimum land area requirements for new onsite systems being served by a public water supply and by private wells.

TABLE 4-6

Minimum Onsite System Land Area Requirement Single Family Residence or Unit Volume of Sewage

Type of Water Supply	Soil Type					
	1A, 1B	2A, 2B	3	4	5	6
Public	0.5 acre[1] 2.5 acres[2]	12,500 sf	15,000 sf	18,000 sf	20,000 sf	22,000 sf
Individual on each lot	1 acre[1] 2.5 acres[2]	1 acre	1 acre	1 acre	2 acres	2 acres

1. Due to the highly permeable nature of type 1 soil, only alternative systems which meet or exceed Treatment Standard 2 can be installed.
2. A conventional gravity system in type 1 soil is only allowed if it is in compliance with all conditions listed under WAC 246-272-11501 (2)(h). One of these limiting conditions is a 2.5 acres minimum lot size.

Mason County Regulations

Mason County Department of Health Services (County) is the local authority that implements Chapter 246-272 WAC for onsite systems with design flows up to 3,500 gpd. At a minimum, the County must implement the requirements of the state regulation. The County may elect to make certain provisions more stringent or implement additional requirements to address local conditions. The County implements the state regulation by means of the Mason County Board of Health Onsite Sewage Regulation (revised April 1998).

Mason County Board of Health Onsite Sewage Regulation

The stated purpose of the County regulation is to:

- Minimize the public health effects of onsite sewage systems on surface water and ground water;
- Minimize the potential for public exposure to sewage;
- Establish design, installation, and management requirements for onsite sewage systems to accommodate effective treatment and disposal of sewage on a long term basis;
- Require a known level for sewage disposal in Mason County; and
- Enhance protection of environmentally sensitive areas within Mason County.

The County's onsite system regulation adopts and incorporates by reference, as minimum standards, Chapter 246-272 WAC. The County regulation also adopts and incorporates by reference the *Mason County Standards for the Design, Construction, Operation, and Maintenance of Sewage Systems.* This document is also known as the *Mason County Department of Health Services Onsite Standards* (revised January 1999). These design standards expand and add to the requirements of Chapter 246-272 WAC. The County's onsite regulation also establishes requirements for certification of pumpers, installers, designers, and maintenance specialists and outlines rules for enforcement and appeals.

Mason County Standards for the Design, Construction, Operation, and Maintenance of Sewage Systems (Mason County Department of Health Services Onsite Standards).

The design standards outline procedures for obtaining approval of an onsite sewage system, as well as design, installation and operation and maintenance requirements. The system must be designed by a septic designer or professional engineer certified by the County.

Design Criteria

For the most part, the design criteria are consistent with Chapter 246-272, with the same requirements for design flow and vertical and horizontal separation. One difference is in the sizing of the septic tank, which is slightly more conservative under the County design criteria. The minimum tank volume is based on the number of bedrooms in the home, as shown in Table 4-7.

TABLE 4-7

Minimum Septic Tank Volume for Residential Onsite Systems

Number of Bedrooms	Volume of Tank (Gallons)
1-2	1,000
3-4	1,200
5+	Add 250 Gallons per Bedroom

The design standards incorporate guidelines from the Washington State Technical Review Committee (TRC) for the design of alternative systems and pressure distribution.

Operation and Maintenance

In June 1993, the County entered into an Agreed Order with Ecology to address deteriorating water quality in four Mason County watersheds. Failing onsite systems along marine shorelines were believed to be a major cause of the water quality problems. One outcome of the Agreed Order was the County's commitment to implement an operation and maintenance (O&M) program for onsite sewage systems. Table 4-8 summarizes the minimum frequency of operation and maintenance required for each type of onsite system, and the owner's options for completing the necessary operation and maintenance work. As the table indicates, with increasing technical complexity of the onsite system, more frequent O&M oversight is required. There is also a greater need to have the work completed by appropriately trained technical personnel.

TABLE 4-8

Mason County Requirements for Onsite System Operation and Maintenance

Inspection Interval	Conventional Gravity or Graveless Chambers w/ or w/o Reduction	Pressure Distribution or Siphon	Mound or Sandfilter	Aerobic Units	Disinfection Units
First 6 weeks					PRO, CMS
First 6 months		CMS		PRO, CMS	
As required by the manufacturer or NSF, but no less than every 6 months				PRO, CMS	PRO, CMS
Year 1 of cycle		HO, CMS			
Year 2 of cycle		HO, CMS			
Year 3 of cycle	HO, PUM, INS, DES, CMS	CMS			

HO = Home Owner
PRO = Proprietary Device Licensee
PUM = Certified Pumper
INS = Certified Installer
DES = Certified Designer
CMS = Certified Maintenance Specialist

The design standards outline the County's implementation plan for a phased O&M program that was initiated in 1997. In Phase 1 (1997), participation in the O&M program was mandatory for all new permits where the design required an aerobic treatment device or was a 'Table 6' repair requiring Treatment Standard 1 or 2 due to site limitations. Participation was also required for existing systems located within critical aquifer recharge areas, along lakes designated as the primary source of drinking water, and along marine shorelines within the Oakland Bay, Skokomish, and West Hood Canal watersheds, and for existing systems serving food service establishments.

In Phase 2 (1998), participation was to be mandatory for new permits requiring a sandfilter or mound design and all existing systems located along designated freshwater shorelines. In Phase 3 (1999), participation was to be mandatory for new permits requiring pressurized distribution and all existing systems located along marine shorelines. In Phase 4 (2000), participation was to be mandatory for all new permitted systems, regardless of the type of system design. The program is voluntary for existing

systems that are not located over critical aquifer recharge areas or along designated shorelines.

The County has been implementing the O&M program on a much less aggressive schedule than outlined in the design standards due to a lack of resources and other problems. However, it is assumed that ultimately the program will be fully implemented.

LARGE ONSITE SEWAGE SYSTEMS

Onsite systems with design flows, at any common point, greater than three thousand five hundred gallons per day are considered "large" or "community" onsite sewage systems. The Washington Department of Health (DOH) approves and regulates large onsite sewage systems (LOSS) up to a peak day flow of 14,500 gpd.

Chapter 246-272 WAC

While the overall general requirements of Chapter 246-272 WAC apply to LOSS, the large systems are specifically addressed in Section 246-272-08001 of the regulation. This section identifies information that must be submitted to DOH in order to obtain approval of a LOSS, as well as management, operation and maintenance, and permitting requirements.

Due to the potential for greater impacts from a LOSS, the proponent must submit an engineering report that includes the proposed design information, as well as a thorough analysis of the proposed site. The site analysis must discuss site topography, geology, surface and ground water, drainage, zoning, surrounding land use, etc. The proponent must also present how the system will be managed, operated, and maintained. The engineering report, as well as the design plans and specifications, are required to be prepared by a professional engineer. A certified installer is required to construct the system, and the design engineer must inspect the construction of the system.
Once the engineer documents that construction is complete and the DOH engineer performs the final inspection, the LOSS is issued an operating permit from DOH. As-built drawings and an operation and maintenance (O&M) manual, prepared by the design engineer, are also required. The operating permit is renewed annually, if the management, treatment adequacy, and O&M is acceptable to DOH.

DOH Design Standards

Chapter 246-272-08001 WAC states that a proposal to construct a LOSS must meet the requirements of the *Design Standards for Large Onsite Sewage Systems,* (revised DOH, 1994). These standards are based on the regulation and outline in more detail the planning and design requirements for large onsite systems.

Design Flows and Loadings

Design flows for multiple single homes or multi-family dwellings that discharge to a LOSS are based on 120 gallons per day per bedroom. Table 4-9 indicates the allowable flows per dwelling unit based on number of bedrooms and estimated individuals per dwelling unit. These design flows are considered to be typical peak flows for residences of the respective size. Therefore, if actual peak flows are expected to be greater than these estimates, the higher flow estimates should be used.

TABLE 4-9

Residential Design Flows for Large Onsite Sewage Systems

Bedrooms/Unit	Individuals/Unit	Design Flows (gpd)
1	2	200*
2	3	240
3	3.5	360
4	4.5	480

*Minimum design flow per unit
Note: Minimum design flow for subdivisions and new mobile home parks is 360 gpd per lot or space.

Design flows for facilities other than residential, including commercial, recreational, and institutional facilities, should be based on Table 4-10 or other appropriate design flow information. Also, if water use data is available, this information can be used to justify design flows.

Septic Tanks

Septic tank volume must be sized to provide 1.5 times the peak day design flow. This volume can be provided with a single tank or with multiple tanks. If multiple tanks are used, the first tank should have a minimum of 24 hours detention time at peak flow. Enhanced treatment beyond a septic tank may be required based on factors such as soil type, strength of the wastewater, and location in an area of special concern. Typical domestic wastewater strength is assumed to be 230 mg/l BOD_5, 150 mg/l TSS, and <50 mg/l of Total Oil and Grease (TOG). If wastewater strength was expected to be significantly greater than these levels, then enhanced treatment beyond that provided by a septic tank would likely be required.

TABLE 4-10

Design Flows for Non-Residential Development

Discharge Facility	Design Units	Flow** (gpd)	Flow Duration (hr)
School w/ showers and cafeteria	Per person	16	8
School w/o showers and w/ cafeteria	Per person	12.6	8
School w/o showers and w/o cafeteria	Per person	10	8
Boarding schools	Per person	75	16
Motels @ 65 gal/person (rooms only)	Per room	130	24
Restaurants*	Per seat	50	16
Interstate or through highway restaurants*	Per seat	180	16
Interstate rest areas	Per person	5	24
Service stations	Per vehicle served	10	16
Factories w/ showers	Per person/8-hr shift	25	Operating period
Factories w/o showers	Per person/8-hr shift	15	Operating period
Shopping centers*	Per 1,000 sf of floor space	200-300	12
Hospitals*	Per bed	300	24
Nursing homes*	Per bed	200	24
Homes for the aged*	Per bed	100	24
Doctor's office in medical center*	Per 1,000 sf	500	12
Laundromats	Per machine	500	16
Community colleges	Per student and faculty	15	12
Swimming pools	Per swimmer	10	12
Theaters, drive-in type	Per car	5	4
Theaters, auditorium type	Per seat	5	12
Churches w/o kitchen	Per seat	3	4
Churches w/ kitchen	Per seat	5	4
Day care centers	Per person	20	12
Picnic areas	Per person	5	12
Campgrounds w/ limited comfort stations, no laundry, no sewer hookup	Per camp site	50	24
Campgrounds/RV parks w/ flush toilets, showers, laundry, no sewer hookup	Per camp site	75	24

TABLE 4-10 – (continued)

Design Flows for Non-Residential Development

Discharge Facility	Design Units	Flow** (gpd)	Flow Duration (hr)
Campgrounds/RV parks w/ flush toilets, showers, w/ or w/o laundry, and sewer hookup	Per camp site	100	24
Campgrounds/RV parks w/ sewer hookup only, no comfort station	Per camp site	50	24
Trailer dump stations*	Per dump	20	24

*Indicates potential high waste strengths facilities requiring pretreatment
**Includes normal infiltration
Source: Criteria for Sewage Works Design (Ecology, revised December 1998)

Disposal Field Design

Vertical separation at the proposed LOSS disposal site must be a minimum of 3 feet to any restrictive layer, high ground water table, or Type 1A soil. Mounds or sand fill cannot be used to achieve the required minimum separation.

Pressure distribution is required for all LOSS disposal fields, regardless of soil type or treatment design. Design must be according to the TRC "*Guidelines for the Use of Pressure Distribution Systems.*"

The disposal field is required to be constructed in two drainfields, each at 50 percent of the required area. A third disposal field equal to 50 percent of the required area must also be constructed to provide for alternating disposal. The alteration of discharges should be designed according to the TRC "*Guidelines for Alternating and Dosing Systems*". An additional area equal to 50 percent of the required disposal area must be reserved for construction of a future replacement disposal field.

Disposal field loading rates are as shown in Chapter 246-272 WAC, however, a LOSS would not be allowed in Type 6 soils. A LOSS would only be allowed in Type 1A soils if enhanced treatment was provided.

Ground Water Discharge

Within the state of Washington, a discharge of wastewater effluent to ground water greater than 14,500 gallons per day (gpd) is regulated by the Department of Ecology (Ecology). The discharges to ground water are required to comply with Chapter 173-200

WAC, the Ground Water Quality Standards. Ecology implements the Ground Water Standards through issuance of a State Waste Discharge Permit, which details permit limits, special conditions, and monitoring requirements. Procedures for issuance of a State Waste Discharge Permit are presented in Chapter 173-216 WAC.

GROUND WATER QUALITY STANDARDS

Chapter 173-200 applies to all ground waters of the state that occur in the saturated zone beneath the land surface. According to the regulation, discharges must be applied in a manner that will not cause pollution of any ground waters in the saturated zone. Compared to surface water, ground water is relatively immobile. Ground water residence times can vary from a few weeks to thousands of years. This fact alone makes the assimilative capacity of ground water limited. Once reaching an underground aquifer, the physical and chemical characteristics of water change slowly. While ground water may support a number of beneficial uses, the overriding basis for the regulations is to protect all ground water as a potential drinking water source.

Ground Water Quality Criteria

Accordingly, the Ground Water Standards contain numerical criteria that cannot be exceeded in ground water, with exception of natural causes. The ground water quality criteria in WAC 173-200 are human health based standards that, for many parameters, are equivalent to the Washington State Department of Health Drinking Water Standards. The Drinking Water Standards do not directly apply to the discharge, but are mentioned for comparison and explanation of the Ground Water Quality Standards.

Anti-Degradation Policy

The state of Washington has very high quality ground water, such that it often does not require treatment to be used directly for drinking water. The goal of the Ground Water Standards is to maintain that high quality, and to protect it to the level of a drinking water source. The intent of the regulation is to prevent degradation of ground water quality beyond existing background conditions. Degradation above background levels can only be allowed when "an overriding consideration of the public interest will be served" and "all contaminants have been provided with all known, available, and reasonable methods of prevention, control and treatment (AKART) prior to entry." This policy of "anti-degradation" often becomes the prime determinant of what can be discharged to ground water.

As stated above, the Ground Water Standards contain numerical criteria that cannot be exceeded in ground water (with the exception of natural causes) and that are comparable to the numerical criteria established for drinking water. The most significant standard is for nitrate, which is 10 mg/l. The Ground Water Standards do not allow a discharge to

cause an increase in nitrate, or any other parameter, in the ground water up to the value cited in the ground water quality criteria. Quite to the contrary, as previously mentioned, the intent is to maintain the water quality that is present prior to the onset of the discharge (i.e., the background water quality).

The background water quality would be the quality of a sample taken upgradient of the discharge area. In essence, the intent of the Ground Water Standards is to maintain the quality of the ground water downgradient of the discharge area at the same quality as the upgradient ground water. This policy is known as the "Antidegradation Policy", which is found in WAC 173-200-030. Section (2)(a) and (c) of this policy are reiterated as follows:

(a) Existing and future beneficial uses shall be maintained and protected and degradation of ground water quality, that would interfere with or become injurious to beneficial uses, shall not be allowed.

(c) Whenever ground waters are of a higher quality than the criteria assigned for said waters, the existing water quality shall be protected, and contaminants that will reduce the existing quality thereof shall not be allowed to enter such waters. Exceptions to this standard can be issued in those instances where it can be demonstrated to the department's satisfaction that: (i) An overriding consideration of the public interest will be served; and (ii) All contaminants proposed for entry into said ground water shall be provided with all known, available, and reasonable methods of prevention, control, and treatment prior to entry.

In actuality, most waste discharges to the ground water will result in an increase in contaminants above the level of the background water quality in the area downgradient of the discharge. When this occurs, Ecology must make a determination of what constitutes degradation and impact to beneficial use, what constitutes an overriding consideration of the public interest, and what constitutes all known, available, and reasonable methods of prevention, control, and treatment (AKART) for the effluent discharge.

Design Requirements

Nitrogen is the major parameter of concern with the discharge of treated wastewater into ground water. Nitrate nitrogen (NO_3-N) is the form of nitrogen of highest concern because of its potential impact on human health. The ground water standard for nitrate is 10 mg/L, the same as the current drinking water standard. Nitrate is a highly soluble and mobile species. If it is not taken up in the root zone, it will readily migrate to groundwater. Reduced forms of nitrogen, such as organic nitrogen and ammonia, are readily oxidized to nitrate. Therefore, reduction of total nitrogen to less than 10 mg/L prior to ground water discharge is generally recommended. To be accepted as AKART,

treatment technologies for wastewater effluent discharges to ground water must be capable of reducing total nitrogen in the discharge to less than 10 mg/l.

In situations where the background nitrate level is low (i.e. less than 0.5 mg/l), Ecology will generally seek to limit the downgradient increase in nitrate to no more than 2 to 3 mg/l. This higher contaminate limitation is viewed as not resulting in degradation of beneficial uses. Increases significantly above this level would be required to demonstrate that there is a significant "overriding consideration of the public interest" that will be served by the project. As a result, in designing a disposal system such as a drainfield or sprayfield, nitrogen loading must be evaluated along with hydraulic loading to determine area requirements.

Hydrogeological Site Investigation

The proponent of a discharge to ground water would be required to perform a hydrogeological evaluation of the proposed discharge area. The evaluation must include the following elements:

- Soil investigation (test pits)
- Monitoring well installation
- Aquifer testing and characterization
- Ground water sampling
- Hydrogeologic analysis
- Site characterization and monitoring report

Soil samples should be analyzed for parameters such as grain size, pH, salinity, organic content, and cation exchange capacity. The ground water would need to be evaluated to establish background water quality. The ground water should be tested for parameters such as nitrogen compounds (i.e., nitrate, TKN, ammonia), BOD5, total dissolved solids (TDS), total suspended solids (TSS), coliform, alkalinity, chloride, sulfate, iron, manganese, pH, conductivity and temperature.

Ground Water Modeling

Modeling of the proposed discharge would be required to evaluate the necessary discharge area and to estimate potential impacts to ground water, primarily with respect to nitrate loading. The model would estimate annual nitrogen loads infiltrating from the discharge area, and then estimate the resulting nitrate-nitrogen concentration in ground water when nitrate from the infiltrating source is added to background concentrations.

State Waste Discharge Permit

As stated previously, Ecology implements the Ground Water Standards through issuance of a State Waste Discharge Permit. The permit details the effluent limits, monitoring requirements, and special conditions. Procedures for issuance of a State Waste Discharge Permit are presented in Chapter 173-216 WAC.

It is anticipated that monitoring would be required for a ground water recharge project in order to ascertain impacts and to ensure there is no significant degradation. Ongoing monitoring requirements would be established by Ecology in the State Waste Discharge Permit. Parameters that would generally be monitored under a State Waste Discharge Permit include the following:

- Nitrate
- Total Kjeldahl nitrate (TKN)
- Ammonia
- Biological oxygen demand (BOD)
- Total dissolved solids (TDS)
- Total suspended solids (TSS)
- Total coliform bacteria
- Calcium
- Potassium
- Sodium
- Bicarbonate
- Carbonate
- Chloride
- Sulfate
- Iron
- Manganese

The following field parameters would also be measured in the ground water and reported to the Department:

- pH
- Conductivity
- Dissolved oxygen
- Temperature

SURFACE WATER DISCHARGE

A discharge to surface water is regulated by the Washington Department of Ecology (Ecology), which implements the Clean Water Act under delegation from the Environmental Protection Agency. Permitting of a surface water discharge would also involve the Washington Department of Fish and Wildlife (WDFW). For marine water discharge, permitting and regulatory oversight would involve the Washington Departments of Natural Resources (DNR) and Health (DOH), as well.

WASHINGTON DEPARTMENT OF ECOLOGY

Ecology regulates surface water discharges of wastewater effluent under the Clean Water Act through the issuance of National Pollutant Discharge Elimination System (NPDES) permits. Procedures for the issuance of an NPDES permit are presented in Chapter 173-220 WAC. The state regulation that implements the Clean Water Act, Chapter 173-201A WAC, the Surface Water Quality Standards, is administered through the issuance of NPDES permits.

Surface Water Quality Standards

Chapter 173-201A WAC establishes the standards for surface water quality in the state, and any surface water discharge would be required to meet those requirements. The Surface Water Quality Standards are based on maintaining public health and recreational uses and for the protection of fish, shellfish, and wildlife.

The surface water quality standards consist of five classifications; including AA (extraordinary), A (excellent), B (good), C (fair), and Lake Class. Each class has specific characteristic uses and measurable criteria. Parameters used to distinguish the different surface water classifications include fecal coliform, dissolved oxygen, temperature, pH and turbidity. The surface water quality criteria include 29 toxic substances, including ammonia, residual chlorine, several heavy metals, polychlorinated biphenyls (PCBs), and common and persistent pesticides.

The goal of the surface water quality standards is to maintain existing beneficial uses of surface water by preventing degradation of existing water quality. However, some allowances are made for discharges into a surface water that enable a temporary or mitigated degradation to occur. These allowances are made by establishing *mixing zones* and determining the *assimilative capacity* of the receiving water.

Mixing Zones

Mixing zones may be granted when water quality criteria cannot be achieved by the wastewater treatment process. Before a mixing zone is granted, the discharger is required

to apply AKART ("all known, available, and reasonable methods of prevention, control, and treatment") prior to discharge. AKART represents the most current methodology that can be reasonably required for preventing, controlling, or treating the pollutants in the discharge.

The mixing zone is that portion of the water body where the discharge mixes with the receiving water, thereby resulting in dilution. If a mixing zone is granted, water quality standards may be exceeded within the mixing zone. However, at the outer limit of the mixing zone, water quality standards must be met. For toxic substances, the standards include acute and chronic toxicity criteria that may result in both acute and chronic mixing zones.

Assimilative Capacity

Assimilative capacity describes the surface water's ability to accept pollutant loadings without a permanent degradation of water quality and impairment of beneficial uses. Ecology annually conducts waste load capacity studies for watersheds across the state. The studies are used to determine the assimilative capacity of the water body for various pollutants such as BOD_5, chlorine, ammonia, metals, etc. For example, the assimilative capacity of a surface water with respect to BOD_5 will be based on the mass of an oxygen-depleting substance (e.g. organic matter and ammonia) that can be discharged into a surface water without depleting dissolved oxygen to levels that would be detrimental to aquatic life.

Technology-Based Versus Water Quality-Based Standards

Treatment of discharges to surface water from wastewater treatment facilities are required to meet minimum "technology-based" standards for secondary treatment as presented in Chapter 173-221-040 WAC. The secondary treatment standards are presented in Table 4-1.

TABLE 4-11

Secondary Treatment Standards for Wastewater Treatment

Parameter	Monthly Average	Weekly Average
BOD_5	30 mg/l[2]	45 mg/l
TSS	30 mg/l[2]	45 mg/l
Fecal Coliform[1]	200/100 ml	400/100 ml
pH	Between 6.0 and 9.0	

1. geometric average
2. Monthly average removal cannot be less than 85 percent

If these minimum limits are not sufficient for the discharge to meet the "water quality-based" standards established in WAC 173-201A, more stringent limits and/or additional limits must be developed by Ecology. Limits for toxic compounds, such as ammonia and chlorine, are always determined using water quality-based standards, since they are not addressed in the secondary standards.

Parameters of Concern

The primary parameters of concern in surface water discharge are dissolved oxygen, temperature, nitrogen, phosphorous, and chlorine. Dissolved oxygen is critical for the viability of aquatic life in the stream. Dissolved oxygen can be depleted through the process of breaking down of organic matter and the conversion of ammonia-nitrogen to nitrate-nitrogen. Ammonia and chlorine are toxic to fish and other aquatic life at levels above the water quality criteria. Elevated temperature is detrimental and even fatal to aquatic life and results in lower dissolved oxygen levels.

NPDES Permit

A project to discharge to surface water would require an NPDES permit from Ecology. As stated previously, the procedures for issuance and administration of NPDES permits are presented in Chapter 173-220 WAC. The NPDES permit would include discharge limits for the effluent discharge, as well as operation, monitoring, and reporting requirements. The discharge limits would be developed based on the water quality classification of the water body receiving the discharge. Hood Canal and Finch Creek are the water bodies of concern for this report.

Hood Canal

Hood Canal is classified as Class AA, extraordinary, according to Chapter 173-201A. The water quality standards for Class AA marine water were presented in Chapter 2. Currently, there is only one point source wastewater effluent discharge to Hood Canal, that being Alderbrook Inn Resort, which was permitted in 1991. Ecology staff have stated that no new point source discharges to Hood Canal would be allowed due to historical water quality problems and potential shellfish impacts, particularly in Lower Hood Canal. However, there is no known official written policy to that effect.

Finch Creek

As discussed in Chapter 2, Finch Creek is not specifically classified in Chapter 173-201A. The regulation states that for water bodies not specifically classified, they are automatically classified the same as the water body they discharge into. Therefore, since Hood Canal is classified as Class AA, Finch Creek also would be classified as Class AA.

The water quality standards for Class AA freshwater bodies were also presented in Chapter 2.

WASHINGTON DEPARTMENT OF FISH AND WILDLIFE

In-stream work such as construction of a wastewater effluent into freshwater or marine water requires a Hydraulic Project Approval (HPA) permit from the Washington Department of Fish and Wildlife (WDFW). Over the past ten years or more, WDFW has closely scrutinized the issuance of HPA permits in order to limit potential impacts to fisheries, shellfish, and other aquatic life. Application for an HPA permit for a marine water discharge would require documentation to show that an upland discharge was not feasible. It is believed that WDFW would take a very strong stance against issuance of an HPA permit for any proposed discharge to Hood Canal.

WASHINGTON DEPARTMENT OF HEALTH

The Washington Department of Health (DOH) is responsible for protecting the public safety and health with regard to potential impacts to surface waters or shellfish resources. If a surface water discharge has the potential to impact shellfish resources, DOH must establish a closure area where commercial harvest would be prohibited and recreational harvest would be discouraged. If there is a discharge to a freshwater body that flows into a marine water, the potential for public health impacts must also be assessed.

WASHINGTON DEPARTMENT OF NATURAL RESOURCES

The Washington Department of Natural Resources (DNR) is responsible for the administration and sale of state shellfish and other aquatic resources for the state of Washington. If these resources are impacted due to a freshwater or marine effluent outfall (i.e., a closure zone is established by DOH), DNR will seek compensatory damages from the discharger.

WATER REUSE

The legislation authorizing water reuse in the state of Washington, Chapter 90.46 RCW (Revised Code of Washington), the Reclaimed Water Use Act, was passed in 1992. The intent of this initial legislation was to encourage the use of reclaimed water while assuring the health and safety of the public and the protection of the environment. The underlying belief was that implementing water reuse would enable the state to use its water resources in the best interest of present and future generations.

The legislation was amended in 1995 to provide for non-consumptive uses of reclaimed water such as ground water recharge, streamflow augmentation, and wetland augmentation. The 1995 amendment also clearly established that reclaimed water is no

longer to be considered wastewater. Reclaimed wastewater would be classified as a water resource.

Chapter 90.46 RCW required the Washington Departments of Health (DOH) and Ecology (Ecology) to jointly develop interim standards for the implementation of water reclamation and reuse. The interim standards were issued in 1994, and the Water Reclamation and Reuse Standards (Reuse Standards) became final in 1997. The Reuse Standards are the primary guidance provided for the development of water reclamation projects.

The use of reclaimed water must also meet the applicable water quality standards. An application to a surface water body is required to comply with the Surface Water Quality Standards (WAC 173-201A). Likewise, an application of reclaimed water to ground water is required to comply with the Ground Water Quality Standards (WAC 173-200). Ecology and DOH have a memorandum of understanding (MOU) that outlines responsibility for review and permitting of reclaimed water projects. Both agencies are required to review and approve the proposed project.

WATER RECLAMATION AND REUSE STANDARDS

The Reuse Standards apply to reclaimed municipal wastewater that is utilized for a direct beneficial use. The primary focus of the Reuse Standards is to provide for the protection of public health. Subsequently, the standards require the reclaimed water to be continuously and reliably treated. As a result, a high level of redundancy and reliability must be provided with each water reclamation facility.

Treatment Classifications

The Reuse Standards classify reclaimed water based on the level of treatment the water receives. The classification is dependent upon the degree of stabilization and disinfection provided by the treatment process prior to any reuse application. There are four classifications for reclaimed water, Class A, B, C and D, with Class A being the highest. The treatment requirements for these four classes are presented in Table 4-12.

Based on the requirement of a higher level of treatment, Class A water has the most reuse potential and the least restrictions on its use. For reclaimed water to be classified as Class A, it must have been continuously oxidized, which is essentially the equivalent of secondary wastewater treatment, and continuously coagulated and filtered, which is similar to conventional drinking water treatment. Finished water turbidity must not exceed a monthly average of 2 NTU and cannot exceed 5 NTU at any time. The reclaimed water must also be highly disinfected; a 7-day median value of $\leq$ 2.2 total coliform per 100 milliliters (100 ml) with no single sample exceeding 23 total coliform per 100 ml. Classes B, C, and D, respectively, are of lesser quality than Class A, based

on lesser levels of treatment received. The primary differences of these classes from Class A are elimination of the coagulation and filtration requirement and decreasing disinfection requirements.

TABLE 4-12

Reclaimed Water Classification Requirements

Reuse Class	Continuously Oxidized[1]	Continuously Coagulated[2]	Continuously Filtered[3]	Disinfection[4] (Total Coliform Density)	
				7-Day Median Value	Single Sample
A	YES	YES	YES	<2.2/100 ml	23/100 ml
B	YES	NO	NO	≤2.2/100 ml	23/100 ml
C	YES	NO	NO	≤23/100 ml	240/100 ml
D	YES	NO	NO	≤240/100 ml	no standard

1. **Continuous Oxidation.** Oxidized wastewater is defined as wastewater in which the organic matter has been stabilized. The concentration of five-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) cannot exceed 30 mg/L on a monthly average basis. The wastewater must be non-putrescable, i.e. no foul smell, and must contain dissolved oxygen.
2. **Continuous Coagulation.** Coagulated wastewater is defined as an oxidized wastewater in which colloidal and finely divided suspended matter have been destabilized and agglomerated prior to filtration by the addition of chemicals or an equally effective method.
3. **Continuous Filtration.** Filtered wastewater is defined as an oxidized, coagulated wastewater that has been passed through natural undisturbed soils or filter media, such as sand or anthracite. The resulting turbidity, as determined by an approved laboratory method, cannot exceed an average operating turbidity of 2 nephelometric turbidity units (NTU) on a monthly average basis. The turbidity cannot exceed 5 NTU at any time. The turbidity must be monitored continuously.
4. **Disinfection.** Disinfected wastewater is defined as wastewater in which pathogenic organisms have been destroyed by physical, chemical or biological means. Disinfection standards generally use coliform density as the representative measure of pathogen destruction.

The classification requirements are "performance-based". This means that if the designated levels of treatment are performed on the reclaimed water and the water meets the designated water quality requirements, the reclaimed water is accepted or classified as the corresponding class of reclaimed water. Based on its reclaimed water classification, it is then accepted as safe for uses identified as requiring the corresponding class of reclaimed water. Further verification of the safety of the reclaimed water is not required, e.g. pathogen testing. This acceptance of public safety is based on extensive research of the public health effects of various treatment levels and uses of reclaimed water.

Potential Uses

The Reuse Standards identify approved uses for reclaimed water and the class of treatment necessary for application to various uses. Potential uses are presented in Table

4-13, along with the classification of reclaimed water necessary for each use. The list is not considered comprehensive, other proposed uses can be evaluated on a case-by-case basis. The primary factor to be considered is the potential for impact to public health. Therefore, the higher the potential for public contact with the reclaimed water, the higher the classification that will be required. Streamflow augmentation and ground water recharge are also potential uses of reclaimed water not included in Table 4-13. Streamflow augmentation and ground water recharge projects must meet the applicable water quality standards for surface water and ground water, as well as the Reuse Standards.

TABLE 4-13

Reclaimed Water Potential Uses and Classification Requirements

Use	Class of Reclaimed Water Required			
	Class A	Class B	Class C	Class D
Irrigation of Non-Food Crops				
Trees and fodder, fiber, and seed crops	YES	YES	YES	YES
Sod, ornamental plants for commercial use, pasture to which milking cows or goats have access	YES	YES	YES	NO
Irrigation of Food Crops				
Spray Irrigation:				
All food crops	YES	NO	NO	NO
Food crops which undergo physical or chemical processing sufficient to destroy all pathogenic agents	YES	YES	YES	YES
Surface Irrigation:				
Food crops where there is no reclaimed water contact with edible portion of crop	YES	YES	NO	NO
Root crops	YES	NO	NO	NO
Orchards and vineyards	YES	YES	YES	YES
Food crops which undergo physical or chemical processing sufficient to destroy all pathogenic agents	YES	YES	YES	YES
Landscape Irrigation				
Restricted access areas (e.g. cemeteries, freeway landscaping)	YES	YES	YES	NO
Open access areas (e.g. golf courses, parks, playgrounds, etc.)	YES	NO	NO	NO
Impoundments				
Landscape impoundments	YES	YES	YES	NO
Restricted recreational impoundments	YES	YES	NO	NO

TABLE 4-13 – (continued)

Reclaimed Water Potential Uses and Classification Requirements

Use	Class of Reclaimed Water Required			
	Class A	Class B	Class C	Class D
Nonrestricted recreational impoundment's	YES	NO	NO	NO
Fish Hatchery Basins	YES	YES	NO	NO
Decorative Fountains	YES	NO	NO	NO
Flushing of Sanitary Sewers	YES	YES	YES	YES
Street Cleaning				
Street sweeping, brush dampening	YES	YES	YES	NO
Street washing, spray	YES	NO	NO	NO
Washing of Corporation Yards, Lots, and Sidewalks	YES	YES	NO	NO
Dust Control (Dampening Unpaved Roads, Other Surfaces)	YES	YES	YES	NO
Dampening of Soil for Compaction (Construction, Landfills, etc)	YES	YES	YES	NO
Water Jetting for Consolidation of Backfill Around Pipelines				
Pipelines for reclaimed water, sewage, storm drainage, gas, electrical	YES	YES	YES	NO
Fire Fighting and Protection				
Dumping from aircraft	YES	YES	YES	NO
Hydrants or sprinkler systems in buildings	YES	NO	NO	NO
Toilet and Urinal Flushing	YES	NO	NO	NO
Ship Ballast	YES	YES	YES	NO
Washing Aggregate and Making Concrete	YES	YES	YES	NO
Industrial Boiler Feed	YES	YES	YES	NO
Industrial Cooling				
Aerosols or other mist not created	YES	YES	YES	NO
Aerosols or other mist created (e.g. cooling towers, spraying)	YES	NO	NO	NO
Industrial Process				
Without exposure of workers	YES	YES	YES	NO
With exposure of workers	YES	NO	NO	NO

Fish Hatchery

Both Class A and Class B reclaimed water may be used by fish hatcheries as an acceptable water supply for the fish hatchery basins. The only difference between Class

A and Class B, discussed above, is that Class B reclaimed water is not required to be continuously coagulated and filtered. Class B reclaimed water must still meet the same disinfection requirements as Class A. Experience has shown that the disinfection requirements are difficult to meet without some level of either conventional or membrane filtration.

Approval for a proposal to use reclaimed water at the state hatchery in Hoodsport would be required from WDFW, as well as Ecology and DOH. It is anticipated that the use of reclaimed water at the fish hatchery would be permitted under an NPDES permit, since the ultimate fate of the reclaimed water would be to surface water. The permit would be issued to the Mason County PUD No. 1 from the Department of Ecology. The permit would include reclaimed water requirements, as well as monitoring requirements. The primary parameters of concern for the use of reclaimed water at a fish hatchery would be ammonia, dissolved oxygen, and pathogens. The permit would address these and other parameters of concern through reclaimed water quality limits established in the permit.

Streamflow Augmentation

Reclaimed water may be discharged to surface water to provide flow augmentation to a stream for flow enhancement, fish and wildlife habitat, irrigation supply, or water right replenishment or transfer. A beneficial use of the reclaimed water must be established for the project to be accepted as a streamflow augmentation project. Discharge of reclaimed water for the purpose of streamflow augmentation must comply with Chapter 173-201A WAC, the Surface Water Quality Standards. Additional information about compliance with the Surface Water Quality Standards was presented previously in the chapter. A streamflow augmentation project would require review and approval by WDFW. If instream work was necessary, a Hydraulic Project Approval (HPA) from WDFW might also be required.

The use of reclaimed water for streamflow augmentation would be permitted under an NPDES permit issued to the Mason County PUD No. 1 from the Department of Ecology. The permit would include reclaimed water requirements, as well as monitoring requirements. Similar to use of reclaimed water in fish hatcheries, the primary parameters of concern would be ammonia, dissolved oxygen, and pathogens. The permit would address these and other parameters of concern through reclaimed water quality limits established in the permit.

Groundwater Recharge

Chapter 90.46 RCW identifies ground water recharge as an allowable and desirable use of reclaimed water. Ground water recharge involves the application of reclaimed water in volumes over and above that required for irrigation needs, for the purpose of replenishing

ground water sources. Reclaimed water may be used to recharge ground water by surface percolation or through direct injection.

In order to be used for ground water recharge by means of surface percolation, the reclaimed water must include an additional step to reduce nitrogen prior to the final discharge to ground water. The basic water quality requirement is that the reclaimed water must meet the ground water recharge criteria, which are defined in the Reuse Standards as the equivalent of the DOH Drinking Water Standards. A major contaminant of concern in ground water recharge is nitrate nitrogen (NO_3-N). Therefore, the treatment process provided must be capable of reducing nitrogen content in the final reclaimed water to the level required by the ground water recharge criteria, i.e. 10 mg/l.

As stated previously, ground water recharge projects must also be in compliance with the state's ground water regulation, Chapter 173-200 WAC. The regulation contains ground water quality criteria that are to be met in the saturated zone. The ground water quality criteria are very similar to the Drinking Water Standards, and subsequently, similar to the ground water recharge criteria mentioned above. Chapter 90.46 RCW, however, includes language that allows additional flexibility for the use of reclaimed water. The legislation states that reclaimed water may be applied to ground water if the reclaimed water meets the "ground water recharge criteria". The ground water recharge criteria are defined in RCW 90.46 as "contaminant criteria found in the drinking water quality standards adopted by the State Board of Health". This allowance, in effect, supercedes the anti-degradation policy stated in the Ground Water Quality Standards. When reclaimed water is used to recharge ground water by surface spreading, it is only necessary that the application not cause the DOH Drinking Water Standards to be exceeded in the ground water that is being recharged. This is a less stringent requirement than the anti-degradation policy presented under the Ground Water Standards.

Direct injection is permitted to a drinking water aquifer as well as a non-drinking water aquifer. The treatment criteria for direct injection, particularly to a drinking water aquifer, require additional treatment beyond that required to produce Class A reclaimed water. Since direct injection will not be evaluated in this study, it will not be discussed further.

Recharge of ground water with reclaimed water would require a State Waste Discharge Permit issued by Ecology. Ecology would also require the development of a ground water monitoring program to ensure degradation does not occur. Application of Class A reclaimed water to land for the purpose of irrigation, if applied at agronomic rates with no intent to recharge the ground water, does not require a permit or a ground water monitoring program.

DESIGN CONSIDERATIONS

Along with the treatment criteria previously discussed, the Reuse Standards and Ecology's *Criteria for Sewage Works Design* (revised 1998) contain requirements for providing reliability and additional protection of public health. The design considerations defined in these documents are discussed below.

Reliability Criteria

The level of treatment reliability required for water reclamation facilities is a major difference from conventional wastewater treatment. The Reuse Standards require *continuous* compliance, which means that the treatment standard must be met on a continuous basis or the treated water cannot be used as reclaimed water. The Reuse Standards contain a number of operational and reliability requirements for the water reclamation facility. Several key requirements are summarized below:

- Equipment and process failures must be signaled by an alarm condition.
- Emergency storage or disposal must be provided in the event of plant failure.
- Bypassing of treatment facilities to the point of use is not allowed for untreated or partially treated water.
- Stand-by power supply must be provided for alarms and automatic actuating valves and other equipment and devices necessary to provide immediate short-term disposal or storage upon failure of any treatment system component.
- Operators must be certified to the levels required of municipal wastewater treatment plants with knowledge of similar process equipment.
- Operating records must be provided to DOH, as well as Ecology.

To address potential situations where the reclaimed water cannot be used due to bad weather, reduced demand, etc., short-term storage or an alternative disposal system, e.g. an outfall to a surface water, must be provided. Provision must also be made for storage or disposal of water that does not meet the treatment and water quality criteria, perhaps due to a treatment upset, equipment failure, etc. If the facility does not have an outfall discharge or some other disposal alternative, emergency storage must be provided for at least 24 hours of flow. Diversion to a different type of reuse requiring a lesser quality of reclaimed water is also possible if the quality is sufficient for that particular use. Storage

areas must be equipped with pumps and valving to route the inadequately treated water back through the reclamation facility once it is operating properly.

Reuse Area Criteria

Class A reclaimed water has no public health limitations on its use, with the exception that it cannot be used for potable drinking water. However, Class A reclaimed water must receive additional treatment such as reverse osmosis in order to be used for direct aquifer recharge through ground water injection. Other classes of reclaimed water have some level of limitations on their use.

Setbacks from Potable Water Systems

The Reuse Standards establish criteria for siting and identifying water reclamation projects and their facilities. Water reclamation storage facilities, valves, and piping must be clearly labeled and adequate measures must be taken to prevent cross connections between potable water and reclaimed water lines. Minimum separation distances between reclaimed water lines and potable water lines are similar to those required for sewer lines. A minimum horizontal separation of 10 feet and a minimum vertical separation of 18 inches must be maintained between the two lines. The Reuse Standards also include setback requirements for the four classifications of reclaimed water. Setback distances are required from potable water supply pipelines, wells, property lines, water bodies, etc. The Class A reclaimed water has the least setback requirements of the four classes. Table 4-14 summarizes the setback requirements for water reclamation facilities.

TABLE 4-14

Setback Distances for Reclaimed Water

Reclaimed Water Use/ Facility	Minimum Distance to Potable Water Well			
	Class A (ft)	Class B (ft)	Class C (ft)	Class D (ft)
Spray or Surface irrigation	50	50	100	300
Unlined storage pond or impoundment	500	500	500	1000
Lined storage pond or impoundment	100	100	100	200
Pipeline	50	100	100	300
Minimum distance between irrigation area and public areas	0	50	50	100

Cross Connection Control Requirements

The proponent of a water reuse program is required to establish a cross-connection control and inspection program consistent with WAC 246-290-490 to prevent contamination of the potable water supply. The program must be developed in coordination with the potable water supplier and be reviewed and approved by DOH. Documentation must address cross-connection control equipment, oversight responsibilities, and operation, inspection, and testing activities.

All valves, piping, outlets, and storage facilities in the reclaimed water system must be clearly identified as containing reclaimed water with a warning against use as drinking water. The public must be provided notification of the use of reclaimed water at all use areas with signs, written notices, or other methods.

Residual Chlorine Requirements for Distribution System Protection

The Reuse Standards recommend that a chlorine residual of 0.5 mg/L be maintained during conveyance from the reclamation plant to the use area. This provision is meant as an operation and maintenance measure to prevent biological growth in the pipeline and sprinkler heads. If this requirement would be detrimental to the end user (e.g., a fish hatchery), then the requirement may be waived.

WATER RIGHTS

The Department of Ecology has indicated that the water rights to reclaimed water belong to the entity producing the reclaimed water. However, the reclaimed water must be retained and utilized in the general use area or basin from which the water originated and for which the original water right was issued. The Reuse Standards require proponents of ground water recharge projects to provide information on the intended water rights status (i.e., artificial storage or abandonment) of the reclaimed water. All reclaimed water projects must be addressed in the proponent's comprehensive water plan.

BIOSOLIDS TREATMENT AND UTILIZATION

Domestic wastewater biosolids are regulated by federal and state regulations. Mason County does not have additional regulations controlling the treatment and utilization of wastewater biosolids, but have adopted the state regulations. This section summarizes the state and federal biosolids regulations most pertinent to this proposed project.

FEDERAL REGULATIONS

The 1977 amendments to the Clean Water Act required the Environmental Protection Agency (EPA) to develop regulations governing the disposition of municipal wastewater

biosolids. In response to this mandate, EPA issued final rules in 1993 that govern the treatment of wastewater biosolids, as well as their utilization or disposal. The rules also address management practices and record keeping. These rules are commonly referred to as "the 503 regulations" because the primary regulation is found in 40 CFR (Code of Federal Regulation) Part 503, although there were other regulations affected. The 503 rules apply only to the biosolids generated from municipal wastewater systems and domestic septic tanks.

WASHINGTON STATE REGULATIONS

The federal regulation allows EPA to delegate implementation of the biosolids rule to individual states. However, the states' implementation regulation must be at least as stringent as the federal regulation. The State of Washington has adopted the federal requirements in its own regulation, WAC 173-308, which governs the use and disposal of biosolids in this state. This regulation became effective in March 1998 and is enforced by the Department of Ecology (Ecology). In addition, Ecology has been granted the authority to issue biosolids permits, a requirement that resulted from more recent federal rule revisions. The requirements in WAC 173-308 that pertain to pollutant limits, operational standards and management practices are essentially the same as the requirements of the federal regulations.

Land Application

Land application is the most common method of utilization in this state, and is generally the most cost-effective approach for smaller wastewater treatment facilities. While incineration and surface disposal of biosolids are addressed in the federal and state regulations, they are used infrequently in this state. Therefore, requirements for the beneficial use of biosolids through land application will be presented here.

There are three fundamental elements of the biosolids regulations that establish minimum criteria for beneficial use of biosolids through land application. These elements are:

1. pollutant concentrations and application rates
2. pathogen reduction measures
3. vector attraction reduction measures

Each of these elements is discussed below.

Trace Pollutant Concentrations and Application Rates

The chemical composition of wastewater biosolids is directly related to the sources of wastewater influent. Communities with greater industrialization tend to have higher concentration of heavy metals and persistent organics within the biosolids. Biosolids

produced solely from municipal sources is typically low in heavy metals and toxic organics.

The regulations establish maximum allowable concentrations for ten heavy metals in biosolids. If a biosolids sample exceeds the ceiling concentration of any of these heavy metals, it cannot be land applied. Annual trace pollutant loading rates are also applied to the same metals. These are the maximum rates at which a pollutant can be applied to a particular area of land on an annual basis. Table 4-15 presents the pollutant concentration limits and annual loading rates for heavy metals allowed for the land application of wastewater biosolids.

TABLE 4-15

Pollutant Concentration Limits and Annual Pollutant Loading Rates

Pollutant	Concentration (mg/kg)	Annual Loading Rate (kg/ha/yr)
Arsenic	41	2.0
Cadmium	39	1.9
Chromium	1,200	150
Copper	1,500	75
Lead	300	15
Mercury	17	0.85
Molybdenum	18	0.90
Nickel	420	21
Selenium	36	5.0
Zinc	2,800	140

Cumulative trace pollutant loading rates for biosolids are also designated for heavy metals. These rates cannot be exceeded during the life of an application site. Once a cumulative loading limit is reached for a particular limiting pollutant, the land can no longer receive biosolids containing any level of the limiting pollutant.

Pathogen Reduction Requirements

In order for biosolids to be land applied, they must meet specific criteria demonstrating a minimum level of treatment to reduce the density or limit the growth of pathogenic bacteria. By meeting these minimum criteria, a biosolids sample is referred to as meeting Class B pathogen reduction requirements. The term "Class B biosolids" is sometimes erroneously referred to as any biosolids meeting all minimum criteria that allow the biosolids to be land applied, which is not the case. Biosolids must meet vector attraction reduction requirements and minimum pollutant concentration standards, as well as Class B pathogen reduction requirements, at a minimum, in order to be land applied.

Class B biosolids must meet one or more of the criteria for pathogen reduction found in WAC 173-308-170. The most common of these is treatment with a 'process to significantly reduce pathogens' (PSRP). Such processes include aerobic digestion, air drying, anaerobic digestion, composting and lime stabilization. A higher level of treatment known as a 'process to further reduce pathogens' (PFRP) will permit biosolids to meet Class A pathogen reduction requirements. When biosolids meet the Class A standard, they are subject to fewer restrictions for land application as long as they meet certain lower pollutant concentration thresholds.

Vector Attraction Reduction Requirements.

The third minimum requirement for biosolids to be land applied is the vector attraction requirement. This measure is designed to make the biosolids less attractive to disease-causing pests such as rodents and insects. These measures typically reduce the liquid content and/or volatile solids content of the biosolids, or they make the biosolids relatively inaccessible to vector contact by soil injection or tilling. Several vector attraction reduction alternatives are available and are summarized below:

- Reduce by 38% the mass of volatile solids in sewage sludge prior to stabilization.

- For an aerobically digested sewage sludge of less than 38% volatile solids reduction, digest a portion of the sludge in a bench scale unit for an additional 40 days at a temperature between 30° and 37°C. If at the end of 40 days, volatile solids reduction is less than 17%, vector attraction reduction is achieved by the original process.

- For anaerobically digested sewage sludge of less than 38% volatile solids reduction, digest a portion of the sludge with 2% or less solids in a bench scale unit for an additional 30 days at 20°C. If at the end of 30 days, volatile solids reduction is less than 15%, vector attraction reduction is achieved by the original process.

- Treat the sewage sludge in an aerobic process and reduce the specific oxygen uptake rate (SOUR) to 1.5 milligram of oxygen per hour per gram of total suspended solids or less.

- Treat sewage sludge in an aerobic process for 14 days or longer at a temperature in excess of 40°C and an average temperature of 45°C.

- Raise the pH of sewage sludge to 12 or above by alkali addition and without additional alkali maintain the sewage sludge at a pH of 11.5 or above for 2 hours and a pH of 12.0 for an additional 22 hours.

- Raise the solids content of sewage sludge to 75% before mixing with other materials.

Management Practices

Once the three basic criteria discussed above have been met, the regulations identify specific management practices that must be followed during land application of biosolids. The biosolids must be applied at a rate that is equal to or less than the agronomic rate. The placement of biosolids on land cannot adversely affect a threatened or endangered species. Biosolids cannot be applied to ground in a manner that would cause it to enter wetlands or a surface water body, e.g. on frozen or snow-covered ground, nor can it be applied within 10 meters of a surface water. Local requirements for additional buffer distances may be more stringent. Biosolids applied to a lawn or garden must meet Class A standards for pathogen reduction under the 503 regulations.

Monitoring Requirements

Monitoring frequencies are based on quantities of biosolids produced. It is not generally necessary to verify pathogen and vector attraction reduction measures are met for each individual load of biosolids that is land applied, per WAC 173-308-150 (3). The actual monitoring frequencies will depend on the frequency of applications.

Record keeping, Reporting and Certifications

The regulations have specific record keeping, reporting and certification requirements for land application of domestic septage and biosolids. Records must be kept for meeting all pathogen reduction and vector attraction reduction requirements for biosolids and domestic septage. For biosolids, records must be kept of analyses performed for meeting trace pollutant criteria. For publicly owned treatment works with design flow rates greater than 1.0 million gallons per day or serving more than 10,000 persons, annual reports to the Department of Ecology must be made.

Specific certifications are required for meeting pathogen and vector attraction reduction requirements for biosolids. For biosolids, these certifications must be provided by the individual(s) who both prepare and land apply the biosolids. The language in the certifications stress the individual accountability associated with meeting the pathogen/vector attraction reduction provisions of the regulations.

Permitting

All entities that prepare biosolids, as well as those who apply biosolids, are subject to permit requirements, as presented in WAC-173-308-310. The primary permit required for biosolids management activities is the State General Permit for Biosolids Management, which is issued by Ecology. Treatment facilities that apply for coverage under this permit must submit either a complete permit application, or a notice of intent that is followed at a later date by complete permit information.

The permittee must carry out public notice as required under WAC 173-308-310(11), and public hearings if required, in accordance with WAC 173-308-310(12), and comply with requirements of the State Environmental Policy Act (SEPA) as stipulated under proposed WAC 173-308-310(030).

CHAPTER 5

CHAPTER 5

EXISTING SYSTEM EVALUATION

Currently, all developed properties within the Hoodsport community are served by individual onsite wastewater systems. In general, the area has poor soils and a seasonally high ground water table. Many of the systems are in close proximity to Hood Canal or too close to fresh water streams, such as Finch Creek, that flow into Hood Canal. These conditions have resulted in a history of onsite system failures and water quality contamination.

STATUS OF EXISTING ONSITE SEWAGE SYSTEMS

There has not been a comprehensive survey or review of the onsite systems in Hoodsport. However, in 1993, under a grant from the Puget Sound Water Quality Authority, Mason County staff conducted an educational project that surveyed onsite systems along Finch Creek. Apparently, records of the survey no longer exist, but according to information from County staff and from some Finch Creek residents, a significant number of failing systems were identified. Since the survey was considered an educational project, no enforcement action was taken by the County. Since that time, three systems along Finch Creek have been replaced.

Also, in June and July 1997, staff from the Washington Department of Health (DOH), Office of Shellfish Programs, conducted a shoreline survey of approximately one mile of the Hoodsport shoreline. The survey included the examination of approximately 50 sites located along the Hood Canal shoreline; the owners at two sites refused access for inspection. There were no sites found to be discharging waste to the surface of the ground or directly into surface waters. However, 37 of the 50 sites surveyed were categorized as "suspect" (i.e., the site had the potential to contaminate a shellfish harvest area). Most were considered suspect for one or more of the following reasons: insufficient setback distance of the onsite system from surface water, a lack of sufficient drainfield area, or onsite system components that are covered by structures or impervious surfaces.

County onsite systems records were reviewed for systems within the proposed service areas. It is estimated there are about 40 onsite systems within Service Area 1, based on the approximately 40 properties that have been developed with residential, commercial, institutional or some other level of improvement. There are records on file with the County for 19 of these systems. Therefore, for about half the existing systems, no design or installation records exist.

Only a few records exist for systems installed prior to 1990, and information is limited within those files. Since the beginning of 1990, county records indicate that three systems in Service Area 1 were replaced or underwent major repairs. Two others systems required septic tank replacement. The records also indicate two systems were reviewed and found to be operating with no "overt" failure. Three other files noted potential problems and recommendations for repairs, however, no record of repairs was found.

Within proposed Service Area 2, which includes much of the shoreline and business district, as well as Service Area 1, there are approximately 90 properties that have been developed with residential, commercial, institutional or some other level of improvement. If one onsite system is assumed for each developed property, there are about 90 onsite treatment systems in Service Area 2. County records were found for 40 systems within this area, indicating that 55 percent of the systems have no records on file. Within Service Area 2, eight systems have been replaced or undergone major repairs since the beginning of 1990. One other system required septic tank replacement. The records also indicate three other systems were reviewed and found to be operating adequately.

Of the eight systems that were repaired or replaced since 1990, all have been required to meet enhanced treatment design standards due to poor soils, high water table, limited replacement area, proximity to shorelines, etc. The designs for these systems have included sand filters, pressurized drainfields, sand-lined drainfields, "Glendon Biofilter" treatment systems, and chlorination/dechlorination equipment.

From this information, it can be reasonably concluded that a large percentage of the existing onsite systems within the study areas do not meet current design standards. The County records, as well as the absence of records, suggest that many of the systems have been in use for 25 years.

ISSUES WITH CONTINUED USE OF ONSITE SEWAGE SYSTEMS

There are several issues with the continued use of onsite sewage systems in Hoodsport, particularly along Hood Canal and Finch Creek. The marine shoreline is considered unsafe for the harvest of shellfish due to concerns for public health. The creek is subject to frequent contamination from fecal coliform sources and, as a result, further impacts the Hood Canal shoreline. The cost of repairing or replacing onsite systems or constructing new systems in the Hoodsport area to meet current regulations is very significant. There are also requirements for the new systems to be maintained and inspected by trained personnel, further increasing their cost. Finally, it is anticipated that these and other regulatory requirements will increase due to concerns about water quality and impacts to shellfish resources.

IMPACTS ON WATER QUALITY AND PUBLIC HEALTH

As discussed previously in this report, there is a long history of water quality impacts to Finch Creek from failing onsite systems, primarily due to fecal contamination. A group of local citizens known as the "Fecal Ferrets" collected water samples from Finch Creek and around Hoodsport in the fall of 1997 to assess the extent of fecal contamination problems. Mason County tested the samples, and the results for Finch Creek are provided below in Table 5-1.

TABLE 5-1

"Fecal Ferrets" Finch Creek Fecal Coliform Data (1997)

Sample Site	Comments	Fecal Coliform/100ml		
		6/30/97	9/16/97	10/13/97
F-1- At fish hatchery outfall	Mixed sample from creek and outfall	17	130*	110*
F-2- Upstream of Hwy 101 bridge	Culvert entering on north side	1600*	130*	2
F-3- Upstream of bridge @ MCPUD well	Sample downstream of culvert on south side of creek	7	30	70*
F-4- Upstream of Earsley bridge	At confluence with ditch collecting water from Cushman Rd.	11	13	4
F-5- Downstream of bridge near Rioux property		11	17	13
F-6- Upstream of Lewallen property		4	<2	<2

*Denotes value exceeding Class AA water quality standard

As the data indicate, fecal coliform levels can vary significantly from location to location and at different times of the year. Variations can be due to seepage or runoff from onsite systems, stormwater drains, or other such discharges. Though the data do not necessarily suggest it, fecal levels often increase with rainfall, due to runoff and shallow groundwater movement to the water body. The water quality standard for fecal coliform in freshwater is 50 colonies per 100 ml, so the standard was exceeded at three of the six sites on at least one occasion. The two sites closest to Hood Canal exceeded the standard on two of the three dates. It also should be noted that the fecal coliform standard in marine water is 14

colonies per 100 ml. This marine water standard is also the level at which shellfish harvest is considered safe.

The 1997 DOH Shoreline Survey noted that the commercial harvest of shellfish has not been allowed and recreational harvest has not been recommended along the Hoodsport shoreline area since the early 1980s. The survey recommended that the marine shoreline remain closed to both commercial and recreational shellfish harvest due to the density of development currently on onsite systems and the proximity of systems to the shoreline. The survey also recommended periodic bacteriological sampling of bulkhead drains to help further identify onsite systems that periodically discharge to the shoreline.

Due to inquiries for this study, the DOH Office of Shellfish Programs recently began collecting periodic water samples along Finch Creek to assess fecal contamination. The data from these samples are presented in Table 5-2. Similar to the samples collected by the local volunteers, the DOH data indicate that fecal contamination is quite variable from location to location and even at a single location. From both sets of data, it is apparent that there are noticeable sources of fecal contamination entering Finch Creek, particularly downstream of the Mason County PUD No. 1 wellhouse.

TABLE 5-2

Department of Health Fecal Coliform Data – Spring 2000

Sample Site	Fecal Coliform/100ml					
	3/28/00	4/3/00	4/19/00	4/27/00	5/9/00	5/18/00
At fish hatchery outfall	8	<2	13	8	13	350*
At Hwy 101 bridge	11	<2	8	46	79*	140*
Below footbridge near Lumberman's	22	<2	2	130*	70*	130*
At bridge near Rioux property	23	4	2	<2	11	4

*Denotes values exceeding Class AA water quality standard

There is also concern that drainfield effluent from onsite systems has the potential to contaminate private wells in the area. The residences along upper Finch Creek Road are all served by fairly shallow private wells. In 1998, the local group, Fecal Ferrets, measured static water levels in several wells along the creek. These data are shown in Table 5-3. All of the wells consistently showed static water levels at less than 10 feet; and in the winter months levels generally were less than 5 feet from the ground surface. Shallow ground water affects the effectiveness of the onsite systems, as well as provides a conduit for the effluent to reach the wells.

TABLE 5-3

Finch Creek Area Wells - Static Water Levels

Well Site	Top of Well Elevation (ft)[1]	Well Depth (ft)	Static Water Level Below Ground Surface											
			1/98	2/98	3/98	4/98	5/98	6/98	7/98	8/98	9/98	10/98	11/98	12/98
Lewallen	130.14	39	4.5	3.3	2.4	3.2	4.4	5.5	5.6		7.9	8.1		7.1
Winne	91.84	30	4.9	4.9	4.7	4.8	4.9	6.1	5.6		6.2	6.2		5.4
Simmons	75.84		3.6	3.6	3.5	2.3	3.7	4.0	4.3		5.8	5.8		3.1
Jensen	57.44		2.8	2.8	2.6	2.6	2.8	3.7	3.4		3.8	3.8		3.0
Rioux	47.42	32	3.2	3.1	3.1	3.1	3.2	3.2	3.3		3.5	3.5		3.2
PUD#1-1	20.54	45	5.8	5.1	5.3	3.5	5.5							
PUD#1-2	20.63		5.7		6.2	6.8								
Rainfall (in)			22.2	14.7	6.9	2.6	1.3	0.6	0.4	0.1	1.4	5.6	32.0	25.2

1. Elevation above mean sea level (AMSL)

ONSITE SYSTEM COST

Another issue with the continued use of onsite systems in the study areas is cost. The design and construction of the alternative systems that are now being required along Finch Creek and the marine shoreline are quite expensive. Costs range as high as $15,000 to $20,000 for a single residence. The costs for commercial systems can be significantly higher, depending on the size of the system. After they are installed, County regulations require the owner to have the system maintained by a certified maintenance specialist (CMS), resulting in additional ongoing maintenance costs to the owner.

There is also the issue of cost to the community in terms of lost tourism dollars due to the closed beach, restrictions on commercial and public shellfish harvests, and limitations on new or expanded businesses. It would be difficult to assign a dollar value to these costs, but they are believed to be significant.

REGULATORY ISSUES

Finally, over the past decade, the design requirements have increased significantly for onsite systems located in areas with marginal soils, high ground water table, and in close proximity to shoreline areas. It is anticipated that protective regulations will increase and become more stringent as more attention is given to public health, improving water quality, and increasing shellfish harvest potential, particularly in important water bodies such as Hood Canal.

It is also feasible that the Department of Ecology will conduct a Total Maximum Daily Load (TMDL) study on Finch Creek in the next 10 to 15 years. The TMDL study is required by the Clean Water Act for impacted water bodies that do not meet water quality standards. The study identifies sources of pollution, estimates the water body's assimilation capacity, and lays out a strategy to address the problems. As a result of a recent settlement agreement, Ecology has committed to perform as many as 600 TMDL studies over the next 15 years. Therefore, it is quite possible that a TMDL study will be completed on Finch Creek during this period, and a response plan put into place. Such a plan would by necessity address non-point sources on the creek such as onsite systems and stormwater runoff.

CHAPTER 6

CHAPTER 6

WASTEWATER FLOWS AND LOADINGS

This study was initiated to address the feasibility of providing community wastewater service to the homes and businesses in the Finch Creek area, due to the history of water quality problems along the creek. However, in earlier public meetings and in the scoping of this study, there was also concern and interest expressed about the feasibility of service to the business district and shoreline areas. Therefore, with concurrence from a local advisory group, two service area scenarios were developed and evaluated. The first service area is limited to homes and businesses along the Finch Creek corridor. This service area is shown in Figure 6-1, and hereafter is referred to as Service Area 1. The second service area includes approximately 3,000 feet of the Hood Canal shoreline and the business district, as well as the Finch Creek corridor. This service area is shown in Figure 6-2, and hereafter is referred to as Service Area 2.

As previously discussed, the Hoodsport community is a non-incorporated area that is currently designated as a Rural Activity Center (RAC), according to the Mason County Comprehensive Plan. Current and future population information for the Hoodsport community and the designated RAC area was presented in Chapter 3. Design flows and loadings were developed here for the designated RAC area for informational and comparative purposes. During past public meetings and local discussions, there has been limited interest from the larger residential community for a community wastewater system. Therefore, the feasibility of service to the larger Hoodsport community was not evaluated in this report.

WASTEWATER FLOW CONTRIBUTIONS

Flow within the study areas would be contributed from residential, commercial, and institutional sources. There are no identified industrial sources with either service area. Design flows for the Hoodsport community and for the two limited service areas were estimated by a combination of several means including: the number of residences, available water use data, type and size of business or institutional use, accepted design flow criteria, and an allocation for infiltration and inflow (I&I).

RESIDENTIAL WASTEWATER FLOWS

A common means of estimating residential wastewater flows is to review water use data, since 70 to 90 percent of potable water typically enters the sewer system. However, many homes on Finch Creek are on private wells and many of the homes on the Mason County PUD No. 1 water system are not metered. Therefore, this method was not feasible for determining residential flows. The Department of Health (DOH) design

criteria recommend that residential design flows for both individual and large onsite sewage systems be based on a peak day design flow per the number of bedrooms in the residence. The peak day design wastewater flow per bedroom is 120 gallons per day (gpd), with a minimum design flow requirement of 240 gpd. The use of 360 gpd per home is recommended if historical water use data is not available. Therefore, for Service Area 1, peak day residential flows were based on 360 gpd per residence.

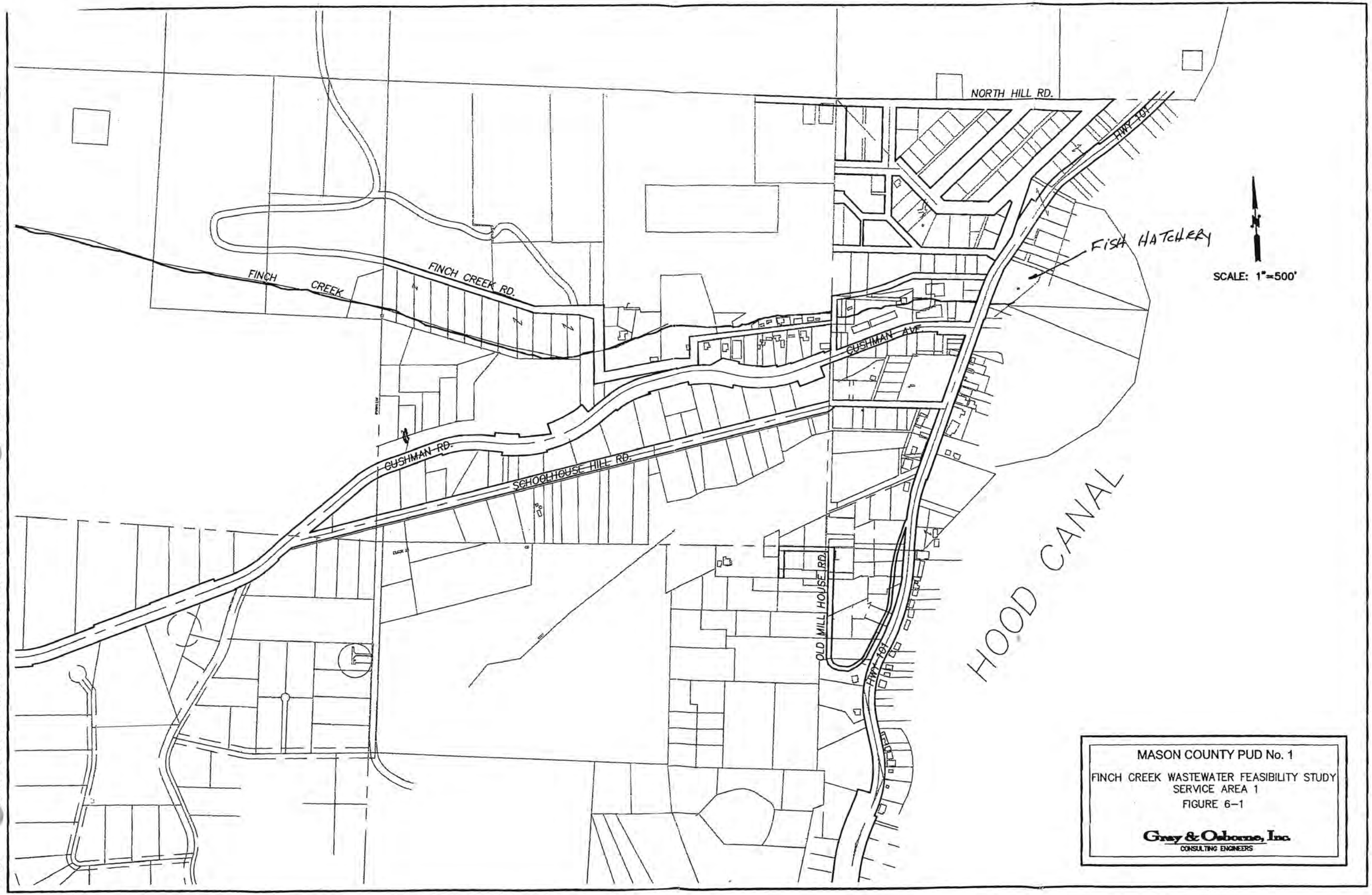
To determine residential flows in Service Area 2, Ecology design criteria were used. The Ecology design manual, *Criteria for Sewage Works Design* (revised December 1998), recommends using an average domestic contribution of 100 gallons per capita per day (gpcd). This includes a normal infiltration and inflow (I&I) allowance of 30 gpcd, so the estimated domestic contribution is 70 gpcd. If an alternative collection system is used, the I&I allowance could be less. Typical peaking factors are then applied to the average flow to establish peak month, peak day, and peak hour flows for the system.

It was anticipated that the shoreline area would need to be served by an alternative collection system such as STEP or grinder pump, so the I&I into the system would be expected to be less than with a gravity system. However, with such a small system, it was desirable to use a conservative estimate to account for variations that could impact the system. Therefore, a per capital flow of 90 gpcd was used, at an average of 2.2 persons per household. If a full gravity system were to be decided upon, then the flows would need to be reevaluated. Peak month residential flows were determined using a peaking factor of 1.5, and peak day residential flows were determined using a peaking factor of 2.

COMMERCIAL AND INSTITUTIONAL FLOWS

For the commercial and institutional flows, water use data were used where it was available. Two years of water data were reviewed, and 85 percent of the maximum month water use was used to estimate the maximum month wastewater flow for those businesses and institutions. Where irrigation was determined to be a factor in maximum month flows, non-irrigation month data were used. Average and peak day flows were determined using peaking factors.

Unfortunately, several businesses and the USDA Forest Service are not currently metered for water use. Where water use records were not available or not considered fully indicative of potential wastewater flows, other methods were used. Commercial flow rates can be based on design units such as type of business, number of employees, square footage of office space, number of restaurant seats, etc. These flows rates were drawn from design information presented in Chapter 4, as well as from Ecology's *Criteria for Sewage Works Design* and EPA's *Design Manual for Onsite Wastewater Treatment and Disposal Systems*. Available septic tank design data were also reviewed as an assistance to estimate the commercial wastewater flows.



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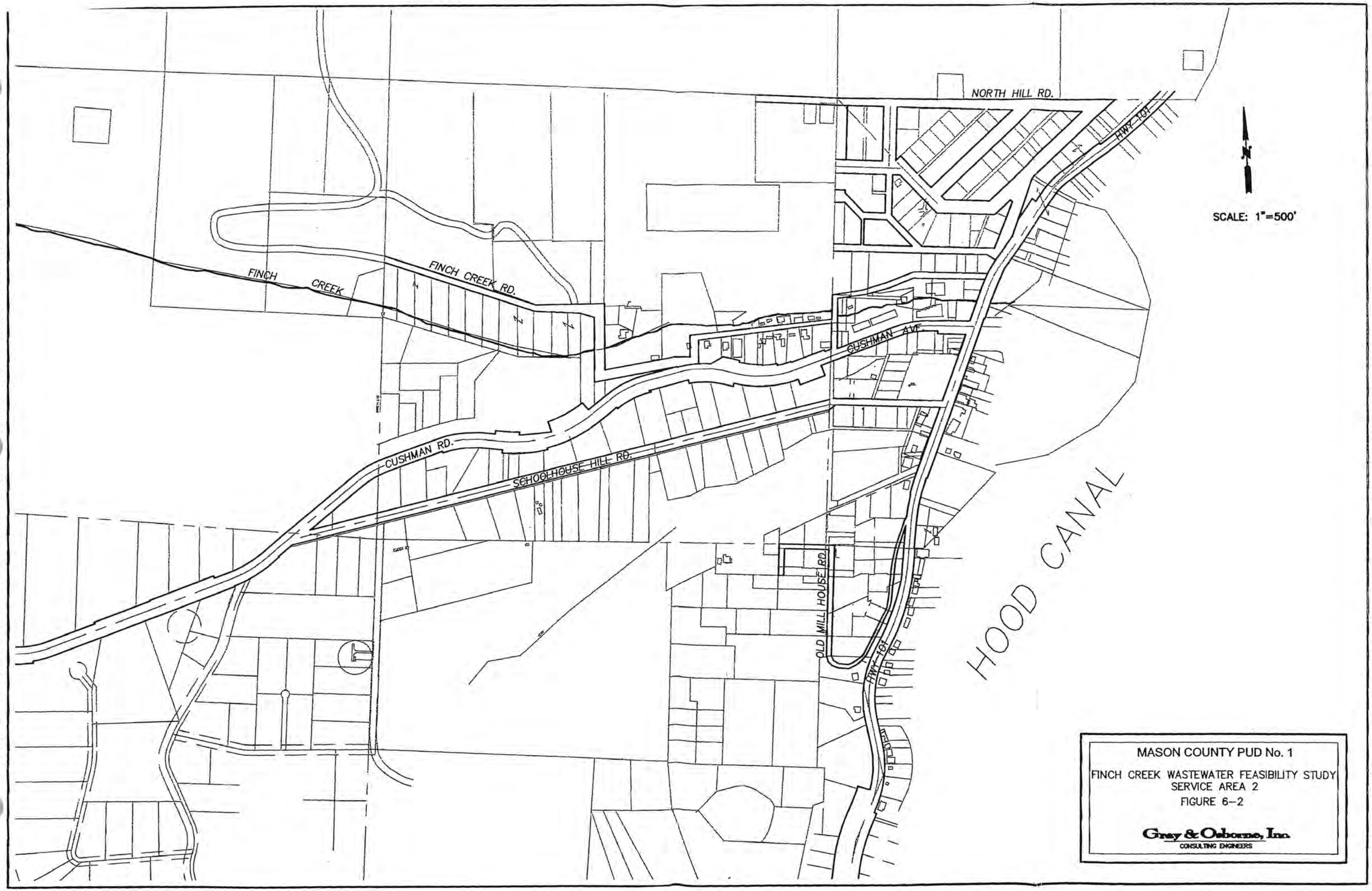

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Once the commercial and institutional flow estimates were developed, a ten percent contingency was added to account for lack of historical flow data, addition of public restrooms at some businesses, changes in operation, etc. Due to the impact of summer tourism and other activities, a peaking factor of 2.0 was used to estimate the maximum month commercial/institutional flows, and a peaking factor of 2.4 was used to estimate the commercial/institutional peak day flows.

INFILTRATION AND INFLOW

Infiltration is defined as the ground water entering a sewer system by means of defective pipes, pipe joints, or manholes. In the past, sewer lines were installed with joints that tended to crack and deteriorate with age. This type of construction contributed significantly to infiltration entering the collection system. Improved construction techniques, including pipe and joint materials, have reduced infiltration entering sewer systems. However, even with new collection system, infiltration continues to be a concern with sewer manholes and in areas of high ground water.

Inflow is defined as surface water runoff entering the sewer system through the holes in manhole covers or direct illegal connections to the sewer system. Illegal connections include yard, basement, or roof drain hookups. Inflow is directly related to rainfall and causes an immediate increase in the peak wastewater flows. Sewer system regulations are usually adopted to prevent stormwater discharges from entering new systems. However, in areas where there are gravity sewers, high ground water, inadequate stormwater systems, and property that might be flooded; property owners have resorted to putting the stormwater into the sewer system. This has caused significant problems for wastewater collection and treatment systems because the systems are not designed to handle the high flows from stormwater. The problem of illicit connections becomes even more significant in small wastewater systems.

Ecology criteria recommend that 30 gpcd be used for gravity collection systems and a lower contribution can be used for low pressure collection systems. For residential flows, an I&I allowance of 20 gpcd was used, because it is anticipated that at least the shoreline portion of the system will be alternative sewer. For commercial and institutional flows, assessment of an I&I allowance is more difficult. The peak flows for commercial and institutional flows will typically occur during the summer season when I&I will be minimal. The I&I will occur at times when the commercial and institutional flows will be at or below average. Also, most of the commercial and institutional entities are in the business district/shoreline area that will likely be served by alternative sewer. Therefore, it was determined to use an I&I allowance of 44 gpd for each commercial and institutional connection, which is the equivalent of one residential connection.

WASTEWATER LOADING CONTRIBUTIONS

In wastewater treatment, the term *loading* refers to the amount of a particular pollutant that is present in the wastewater. The measurement of a pollutant is usually expressed as pounds per day (ppd). In general, loading is determined by multiplying the concentration of the pollutant in the wastewater (weight/volume) and the amount of wastewater flow (volume/time). The loading represented by a given wastewater is typically presented in terms of ppd of biochemical oxygen demand (BOD_5); total suspended solids (TSS); and nitrogen, generally, ammonia (NH_4); and Total Kjeldahl Nitrogen (TKN), which is ammonia plus organic nitrogen.

WASTEWATER CHARACTERISTICS

Wastewater is typically characterized according to its physical, chemical and biological properties. As discussed above, the principal parameters used to characterize wastewater are BOD_5, TSS, and ammonia or TKN, along with pH. The BOD_5 is a measure of the concentration of biodegradable organics in the wastewater as measured over a five-day incubation period. If discharged untreated into the environment, biodegradable organics will deplete natural oxygen concentration and result in the development of septic conditions. A properly designed wastewater treatment process provides sufficient oxygen and detention time for biological stabilization to occur, thereby limiting the effects on the environment. The BOD_5 loading is utilized to size treatment facilities, determine aeration requirements, and to evaluate treatment efficiency. Treatment efficiency is commonly calculated as the percent of BOD_5 removal.

Suspended solids loading is used to determine the amount of organic and inert solids in the wastewater received for treatment. The TSS loading affects the design of primary treatment (i.e., equipment and processes to remove inorganic material and heavier organic material). It also affects the overall production of biosolids and the design of biosolids treatment and disposal processes. The percentage of TSS removal is another measure of the efficiency of the treatment process.

While both BOD_5 and TSS are the most common pollutants of concern in wastewater, other parameters are also important in meeting surface water quality standards or ground water discharge standards. The pH is a measure of the acidity or alkalinity of the wastewater. Excessive acidity or alkalinity can be detrimental to the treatment process, as well as to aquatic life. Nutrient contaminants such as nitrogen and phosphorous can lead to excessive growth of undesirable aquatic life and an upset in the oxygen balance in natural waters. The presence of ammonia will increase the oxygen demand in surface waters and can be toxic to aquatic life in excessive amounts. If applied excessively to land, nitrogen can result in nitrate pollution of ground water. Increasingly, NPDES

permits for surface water discharges and State Waste Discharge Permits for ground water discharges are being written to limit discharge of nitrogen to surface and ground water.

Fats, oils, and grease are other components of wastewater that may cause problems in the collection, treatment, and disposal systems. Oil and grease are relatively stable and are not easily degraded. They tend to coat the surface of equipment, collect on the surface of waters creating an unsightly film, or clog soils when applied to land. Due to residential and commercial sources, oil and grease will be important characteristics of the wastewater in the Hoodsport study area.

Heavy metals such as chrome, cadmium, lead, copper, and zinc can also interfere with the beneficial uses of water due to their toxicity at high concentrations. Typical domestic wastewater, such as that expected in the Hoodsport study area, would be expected to have low concentrations of metals. However, the metals noted tend to concentrate in wastewater biosolids, and could thereby limit the reuse of biosolids. If the treated effluent wastewater is to be discharged to ground water, chlorides and total dissolved solids (TDS) will also be constituents of concern.

Within the study area, there are no data available regarding existing wastewater characteristics and concentrations due to the exclusive use of onsite sewage systems. The wastewater characteristics generated by residential, commercial and institutional connections in the areas are expected to be typical of medium strength wastewater. Table 6-1 presents typical concentrations or measurement levels found in untreated wastewater.

TABLE 6-1

Typical Characteristics of Untreated Wastewater

Constituent	Expected Concentration
BOD_5	220 mg/l
Total Suspended Solids (TSS)	220 mg/l
Inert	55 mg/l
Volatile	165 mg/l
Nitrogen	
Total	40 mg/l
Ammonia	15 mg/l
Phosphorous	8 mg/l
Chlorides	25 mg/l
Total Dissolved Solids (TDS)	250 mg/l
pH	6 to 9
Alkalinity (as Ca CO_3)	100 mg/l
Oil and Grease	100 mg/l
Total Coliform	10^7 to 10^8 / 100 ml
Volatile Organic Compounds	100 ug/l
Heavy Metals	negligible
Source: Metcalf & Eddy, Third Edition	

If septic tanks were to be utilized as part of the wastewater treatment system for primary treatment, the BOD_5 loading and TSS loading would be reduced by as much as 30 percent and 60 percent, respectively. However, the ammonia concentration would be significantly higher due to the additional breakdown of solids and anaerobic conditions in the tank.

ORGANIC LOADING DESIGN CRITERIA

There are no DOH design criteria for organic loading to individual or large onsite systems. Ecology design criteria are taken from the *Criteria for Sewage Works Design* manual. Some typical design loading criteria are presented in Table 6-2.

TABLE 6-2

Unit Wastewater Loading Design Criteria

Source	Design Unit	BOD_5/Unit (ppd)	TSS/Unit (ppd)
Domestic	Per capita	0.2	0.2
Infiltration/Inflow			
Gravity collection	Per capita	0.0	0.0
Pressure collection	Per capita	0.0	0.0
Commercial:			
Motel	Per room	0.26	0.26
Restaurant	Per seat	0.2	0.2
Retail store	Per 1,000 sf	0.01	0.01

Residential organic loadings for BOD_5 and TSS were developed using 0.2 ppd per capita for both parameters and an estimated 2.2 persons per household, as determined in Chapter 3. Based on the estimated average residential flows, this results in both BOD_5 and TSS concentrations of approximately 270 mg/l. This value is a higher concentration than might be considered "typical", however it is appropriate for a small system not significantly impacted by industrial flows and I&I, e.g. partial or total alternative collection system.

Commercial and institutional loadings were developed using two different approaches. Loadings for commercial and institutional entities producing primarily domestic waste were based on design flows and a BOD_5 and TSS concentration of 270 mg/l. Loadings for businesses that produce wastewater containing a significant amount of food waste were based on 300 mg/l. The commercial and institutional loadings were also evaluated using the unit loading criteria presented in Tables 4-10 and 6-2. Nitrogen loadings were based on an estimated TKN concentration of 50 mg/l.

PROJECTED WASTEWATER FLOWS AND LOADINGS

The estimated flows and loadings for Service Area 1 and Service Area 2 are presented in Table 6-3. These were developed based on the design criteria and assumptions presented in this chapter. The number of equivalent residential units (ERU) for Service Area 1 was determined based on the DOH peak day flow criteria of 360 gpd per residential unit. For Service Area 2, the number of ERU's was based on the average flow of 90 gpcd and 2.2 persons per residence, i.e. 198 gpd per residential unit.

Since the peak day design flow for Service Area 1 is not more than 14,500 gpd, a wastewater treatment system for this area would be regulated by the Washington Department of Health. Systems designed for Service Area 2 would be regulated by the

Washington Department of Ecology because the design flows would be greater than 14,500 gpd.

These design flows and loadings have been developed for existing development in the two service areas. If the Mason County PUD No. 1 desires to provide capacity for additional residential, commercial or institutional flows, these design flows and loadings will have to be increased accordingly.

TABLE 6-3

Estimated Flows and Loadings For Service Area 1 and Service Area 2

SERVICE AREA 1				
	Parameter	Average Day	Peak Month	Peak Day
Residential Flows (Estimated Population = 62)	**Flow (gpd)**	5,040	7,560	10,080
	BOD (ppd)	11.3	17	22.7
	TSS (ppd)	11.3	17	22.7
	TKN (ppd)	2	3	4
	ERU's			28
Commercial/ Institutional Flows	**Flow (gpd)**	1,840	3,680	4,420
	BOD (ppd)	4.1	8.3	10
	TSS (ppd)	4.1	8.3	10
	TKN (ppd)	1	1.5	2
	ERU's			12
TOTAL – Service Area 1	**Flow (gpd)**	6,880	11,240	14,500
	BOD (ppd)	15.4	25.3	32.7
	TSS (ppd)	15.4	25.3	32.7
	TKN (ppd)	3	4.5	6
	ERU'S			40

TABLE 6-3 – (continued)

Estimated Flows and Loadings For Service Area 1 and Service Area 2

SERVICE AREA 2				
	Parameter	**Average Day**	**Peak Month**	**Peak Day**
Residential Flows (Estimated Population = 139)	**Flow (gpd)**	12,475	18,715	24,950
	BOD (ppd)	28	42	56
	TSS (ppd)	28	42	56
	TKN (ppd)	5	8	10
	ERU's	63		
Commercial/ Institutional Flows	**Flow (gpd)**	12,940	25,880	31,056
	BOD (ppd)	28	56	67
	TSS (ppd)	28	56	67
	TKN (ppd)	5	10.5	13
	ERU's	65		
TOTAL – Service Area 2	**Flow (gpd)**	25,415	44,595	56,006
	BOD (ppd)	56	98	123
	TSS (ppd)	56	98	123
	TKN (ppd)	10	18.5	23
	ERU's	128		

CHAPTER 7

CHAPTER 7

ALTERNATIVES FOR COMMUNITY TREATMENT SYSTEMS

The evaluation of wastewater treatment alternatives is largely driven by the ultimate means of effluent disposal and the regulatory requirements affecting the disposal. As discussed in Chapter 4, the regulatory requirements differ based on type and volume of discharge. This chapter presents appropriate treatment and disposal alternatives for Service Areas 1 and 2, which were identified in Chapter 6. Each of the alternatives are evaluated based on capital and O&M costs, as well as on non-cost factors such as future expansion potential and environmental concerns. Issues and costs surrounding the continued use of individual onsite systems are also discussed.

SERVICE AREA 1 TREATMENT SYSTEM ALTERNATIVES

The design flows and loadings for Service Area 1 were developed in Chapter 6 and presented in Table 6-3. They are as follows:

(FROM TABLE 6-3)

Service Area 1 Estimated Flows and Loadings

Parameter	Average Day	Peak Month	Peak Day
Flow (gpd)	6,880	11,240	14,500
BOD (ppd)	15.4	25.3	32.7
TSS (ppd)	15.4	25.3	32.7
TKN (ppd)	3	4.5	6

The peak day design flow for Service Area 1, the Finch Creek corridor, is 14,500 gpd. Based on this design flow, the treatment alternatives evaluated must meet design criteria established by the Washington Department of Health (DOH) for large onsite systems (LOSS), unless a mechanical treatment plant is used. The LOSS design criteria are primarily directed toward the protection of public health. As discussed in Chapter 4, the DOH does not evaluate a project's compliance with regard to the Ground Water Standards (Chapter 173-200 WAC). The requirement of such compliance would significantly increase the complexity and the cost of treatment systems for flows under 14,500 gpd.

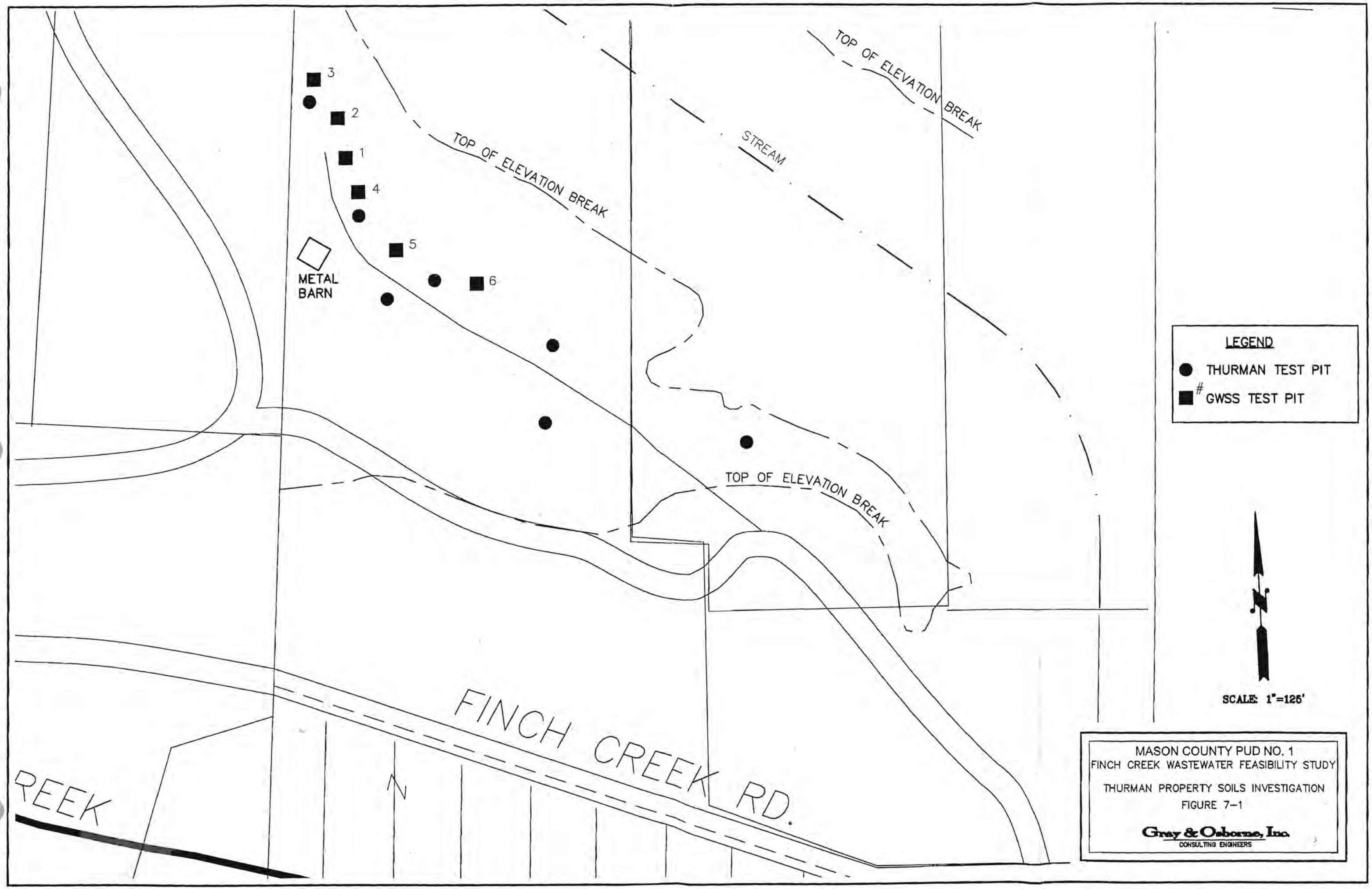
For Service Area 1, it was determined that a LOSS, using a community drainfield for final treatment and disposal, was the most appropriate approach. Prior to the beginning of this feasibility study, there had been some discussion of using land at a local county park for a drainfield site. The site is approximately 400 feet in elevation above Hoodsport and 1.25 miles west via county road. Consequently, the cost of wastewater transmission to the site would be quite expensive. An initial review of this site indicated that the soils had been packed down in areas, raising concern about soil permeability. There was also question as to whether additional land could be made available for future expansion, particularly for any system other than a drainfield. Therefore, it was determined to pursue other possible treatment sites.

A 14-acre parcel of property owned by James Thurman, which lies to the north of upper Finch Creek Road, was evaluated as a potential upland drainfield site. The Thurman property rises steeply from Finch Creek Road and levels off at approximately 240 feet. There are about 6 or 7 acres of level area that would be available for use; the remaining area is too steep or too wet. The site was logged about 40 years ago and is covered with heavy vegetation and alders and evergreens of varying age. Current access to the site is via an unimproved logging road.

In 1996, when the owner sought to short plat the site, six test pits were dug to determine the suitability of the site for onsite systems. The County determined that most of the pits contained Type 1A, i.e. very gravelly, soils that require enhanced onsite treatment. A more recent investigation was performed by Ground Water Science Services (GWSS) in early May 2000. During that investigation, additional test pits were dug that went as deep as 10 feet. With the exception of the western part of the site, where some till was found at about 5 feet deep, soils in the GWSS pits were also found to be sands and gravels. Ground water was not found in any of these pits, however, cemented till was found in some areas at 10 to 11 feet below the ground surface. Based on these evaluations, the Thurman property appears to be a feasible drainfield site for Service Area 1. A site map of the Thurman property is shown in Figure 7-1, including the location of the original and GWSS test pits. The County's soil assessment of the original test pits is found in Appendix 1. The soil logs for the GWSS test pits are included in Appendix 2, along with the GWSS report.

ALTERNATIVE 1: SETTLING TANK WITH PRESSURIZED DRAINFIELD

Two treatment approaches were evaluated to serve Service Area 1, each discharging to a drainfield. The first approach, Alternative 1, provides settling to remove the heavy solids, followed by pressurized distribution to the drainfield. Since the drainfield will discharge to Type 1A soils, the drainfield trenches must be enhanced with sand to provide additional treatment.



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The settling tank, which can be a large septic tank or multiple small tanks, will be sized to provide approximately 2 days of settling time at peak day flow, or about 30,000 gallons. If settling tanks are used in the selected collection system alternative, e.g. with Septic Tank Effluent Pump (STEP) sewers, the large settling tank will not be required. A 1/8" mesh effluent screen will be placed on the outlet pipe to prevent larger solids from passing to the pump tank.

The clarified tank effluent will flow to a 2,000 gallon pumping tank, from where it will be pumped into the drainfield. Two pumps will be provided in the tank, along with the necessary float switches and controls. Each pump will be capable of pumping the required flow necessary to pressurize the drainfield, so that the system can operate at peak flow with one pump out of service. The pumps will be set to alternate pumping into the drainfield.

Since the drainfield would discharge to Type 1A soils, sand-lined trenches are required to provide additional treatment to the wastewater. Type 1A soils are very permeable and provide only limited treatment because the wastewater flows through them very quickly. The sand causes the wastewater to drain more slowly and provides much better treatment than the underlying soils. The drainfield sizing is based on an application rate of 1.0 gallons per day per square foot (gpd/ft^2). The DOH design criteria allow an application rate up to 1.2 gpd/ft^2 in Type 1A soils, where sand-lined trenches are used. However, only primary treatment is being provided to the wastewater prior to its application to the drainfield, and knowledge of wintertime ground water conditions is very limited at this time. Therefore, for planning level estimates, the lower application rate will be used. At a peak day flow of 14,500 gpd, this results in a required drainfield trench area of 14,500 ft^2.

The drainfield trenches would be 4 feet wide and spaced 4.5 feet apart. Standard trenches require drain rock at 3 feet width and 1 foot depth with 2 feet of cover. Because the wastewater would be discharged to Type 1A soil, six inches of sand must be placed on each side of the drain rock and 2 feet of sand must be placed beneath it. This results in a trench 4 feet wide and approximately 5 feet deep. An approved geotextile fabric would be laid along the trench on top of the drain rock to prevent fine material from moving down into the pores and creating clogging problems.

The drainfield area must be laid out in a minimum of two equivalent distribution areas, each capable of handling 50 percent of the peak day flow. A third area, also capable of handling 50 percent of the peak day flow, must be constructed to provide for alternation of the application areas. This allows the soil to fully drain and aerate between applications. A reserve area capable of handling 50 percent of the peak day design flow must also be available for a future replacement drainfield area. In order to reduce the size of the pumps, the required drainfield area will be laid out with four zones, with two additional zones provided for alternation. With the required alternation zones, the total

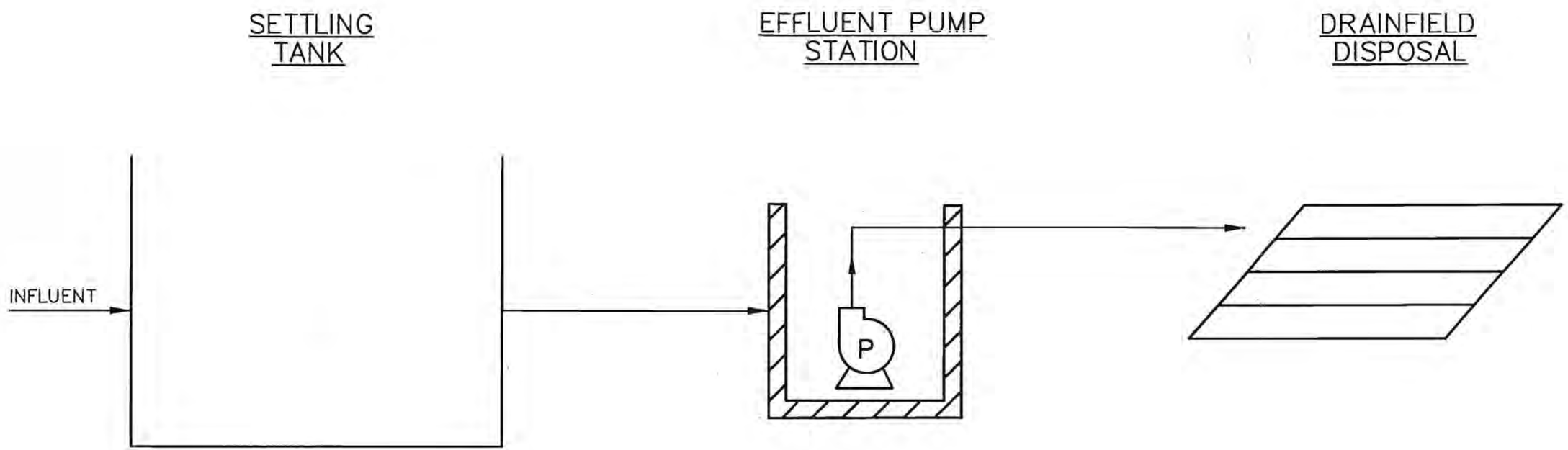
constructed drainfield area, including piping, would be 74,880 ft^2, or approximately 1.8 acres. Including the reserve area, the total drainfield area needed is about 2.4 acres. The design criteria for Alternative 1 are presented in Table 7-1. A process flow diagram for this alternative is shown in Figure 7-2.

TABLE 7-1

Preliminary Design Criteria for Alternative 1 - Settling Tank with Pressurized Drainfield

Component	Value
Settling tank	30,000 gallons
Pump tank	2,000 gallons
Distribution pumps	2 @ 250 gpm, 30 ft TDH
Drainfield	
Drainfield application rate	1.0 gpd/ft^2
Drainfield zones, w/ alternation	6 @ 10,944 ft^2
Drainfield area needed, w/ reserve	2.4 acres
Orifice size	3/16"
Orifice spacing	4 ft
Lateral size	2"

The access road will need to be improved to provide adequate access to the site, and power would need to be provided to operate the distribution pumps. Currently, three phase power is available at the Fire Hall and would be constructed to the site. Occasional power outages could be addressed by installation of an onsite generator, use of a portable generator, or installation of a reserve storage tank. A portable generator could be used at both the treatment site and at a pump station. But in extremely bad weather, it could prove difficult to move the portable unit back and forth to the site. Therefore, installation of a small onsite generator will be assumed for planning purposes. If an alternative collection system such as grinder or STEP pump were proposed, then wastewater would not be able to be pumped to the site. In that event, DOH might not require a generator or emergency storage. The estimated capital cost for construction of this system is provided in Table 7-2, and the O&M costs are presented in Table 7-3.



TREATMENT PROCESS SCHEMATIC

MASON COUNTY PUD No. 1

FINCH CREEK WASTEWATER FEASIBILITY STUDY
SERVICE AREA 1 – ALTERNATIVE 1
SETTLING TANK W/ PRESSURE DRAINFIELD

FIGURE 7–2

Gray & Osborne, Inc.
CONSULTING ENGINEERS

TABLE 7-2

Estimated Construction Costs for Alternative 1 - Settling Tank with Pressurized Drainfield

Item	Estimated Cost
Mobilization/demobilization	$ 35,300
Clearing & grubbing	$ 7,500
Sitework, excavation, waste hauling	$ 81,430
Tanks	$ 30,000
Drainfield pumps, controls	$ 20,000
Drainfield media	$ 53,000
Geotextile	$ 9,760
Site piping, valves	$ 50,000
Topsoil, mulch, seeding	$ 16,270
Electrical incl./ generator	$ 60,000
Road work/security gates	$ 25,000

Subtotal.....$388,260
Sales tax (7.9%).....$ 30,680
Subtotal.....$418,940
Contingency (20%).....$ 83,790
Estimated Construction Cost.....$502,730
Engineering, Legal, Administration (20%).....$100,550
Total Estimated Capital Cost.....$603,280

Labor costs for system O&M were developed based on an estimated cost of $30 per hour for a contracted operator at approximately one hour of oversight per week. Power costs are based on $0.056 per kilowatt-hour ($/kw-hr), and solids hauling costs are based on $0.13 per gallon for hauling and disposal annually.

TABLE 7-3

Estimated O&M Costs for Alternative 1 - Settling Tank with Pressurized Drainfield

Item	Estimated Cost
Labor	$1,560
Supplies	$ 500
Utilities	$1,000
Septic solids hauling	$4,000
Repair and replacement	$2,000
Miscellaneous	$ 500
Total Annual O&M Costs	**$9,560**

Advantages to this treatment approach include low operation and maintenance (O&M) costs and relatively low capital cost. This is due primarily to the simplicity of the facility. The facility would require very little oversight on a day-to-day basis. The primary disadvantage to this approach is the low level of wastewater treatment prior to the drainfield. As a result, the drainfield provides the primary means of treatment before the effluent infiltrates to ground water. If the drainfield treatment should be inadequate or actually fail, then service would be seriously compromised. However, the potential to impact ground water or public health is limited at this site due to its relative isolation. The process is susceptible to impacts from grease or excessive solids loading that can clog drainfield soils, similar to the individual onsite systems that are currently in use.

The potential to expand this type of treatment to treat additional flows is limited, particularly at this location. A larger system would be regulated by the Department of Ecology, and a higher level of treatment would probably be required to accommodate additional flows. Ground water monitoring would also likely be required. Until wet weather monitoring is completed, it is not clear how much additional effluent can be applied at this particular site. The site is limited for drainfield expansion, however, it is feasible that additional land could be purchased from an adjacent landowner. An alternative approach to expansion would be to construct an additional drainfield system at a different location. Under that scenario, each of the systems would be considered a LOSS, governed by DOH design and regulated by DOH.

ALTERNATIVE 2: RECIRCULATING GRAVEL FILTER WITH PRESSURIZED DRAINFIELD

Alternative 2 consists of a primary settling tank, a recirculation tank/pump tank, and a recirculating gravel filter (RGF), followed by disposal to a pressurized drainfield. If settling tanks are used in the collection system, such as with a STEP collection system, the settling tank at the treatment site would not be required. The RGF would produce a relatively well treated effluent, therefore, the drainfield would not be relied on to provide the primary means of treatment. It would provide additional polishing of the effluent and final disposal. For this reason, the maximum allowable drainfield application rate, 1.2 gpd/ft^2, would be used, allowing the drainfield to be significantly smaller than with Alternative 1.

With this alternative, the influent wastewater flows initially into the primary settling tank, where the heavier solids are settled out. The wastewater then flows to the recirculation tank where it mixes with partially treated filter effluent. The mixed wastewater is pumped to the filter at regular intervals, with doses controlled by both a timer and a high level float switch. The wastewater is distributed under pressure into a layer of drain rock lying over the filter media. The filter media consists of a minimum of 36 inches of fine gravel, commonly referred to as pea gravel.

As the wastewater moves downward through the gravel media, biological growth attached to the media removes the organic material in the wastewater. The filtrate is collected at the bottom of the filter and returned to the recirculating/mixing tank. A portion of the filtrate is discharged from a splitter box to the pump tank; the remainder cycles back through the filter. A float-activated valve is frequently used to control recirculation and discharge from the tank. The RGF system is designed for wastewater to pass through the filter about 5 times prior to discharge. When discharged, treated wastewater would be pumped from the treatment system to a drainfield for final treatment and disposal.

The primary settling tank should provide a minimum of 24 hours detention time at peak day flow, or 14,500 gallons. One or more pre-cast tanks or a single cast-in-place tank can be used to provide the required volume. The outlet of the settling tank would be screened with a minimum 1/8 inch plastic mesh screen or a bag filter to prevent the passing of solids on to the recirculation tank.

The recirculation tank volume would be equal to 150 percent of the peak day flow, or 21,750 gallons. The recirculation pumps would operate on a timer, in alternating cycles of 5 minutes on, 25 minutes off. This dosing schedule provides 48 dosing periods per 24 hours, allowing the influent/filtrate mixture to cycle through the filter about 5 times before discharge. Two recirculation pumps capable of pumping the peak day flow over the 48 dosing periods would be provided. These two pumps, plus a third backup pump would be sized at 150 gpm each.

Float switches would be wired in parallel with the timer to control the pumps during periods of excessive wastewater flows, and in the event of timer malfunction. Both timer and float switch controls are required. To protect the pump and the distribution piping from excessive solids, the pumps would be enclosed in a 1/8 inch mesh plastic screen. The tank would be designed to enhance settling and retain solids that might be flushed out of the filter.

Filter sizing is based on the DOH criteria of a maximum filter loading rate of 5 gallons per day per square foot (gpd/ft^2). For the peak day design flow of 14,500 gpd, the total filter surface area required is 2,900 ft^2. Two filter zones would be constructed so that one zone could be taken off line for maintenance. Each zone would be sized at 1,450 ft^2, or approximately 38 feet square. Distribution laterals would be 1-inch piping laid at 2 feet on center, with 1/8-inch orifices spaced at 2 feet on center. Planning level estimates will assume an above-grade concrete filter basin. The filter could also be constructed in an earthen basin lined with a synthetic membrane. The lined earthen basin is frequently less expensive, but does not have the durability and length of life of the concrete basin.

The filtrate would flow to a 2,000 gallon pump tank, from where it would be pumped into the drainfield. Two pumps will be provided in the tank, along with the necessary floats

and controls. Each pump will be capable of pumping the required flow necessary to pressurize the drainfield, so that the system can operate at peak flow with one pump out of service. The pumps will be set to alternate pumping into the drainfield.

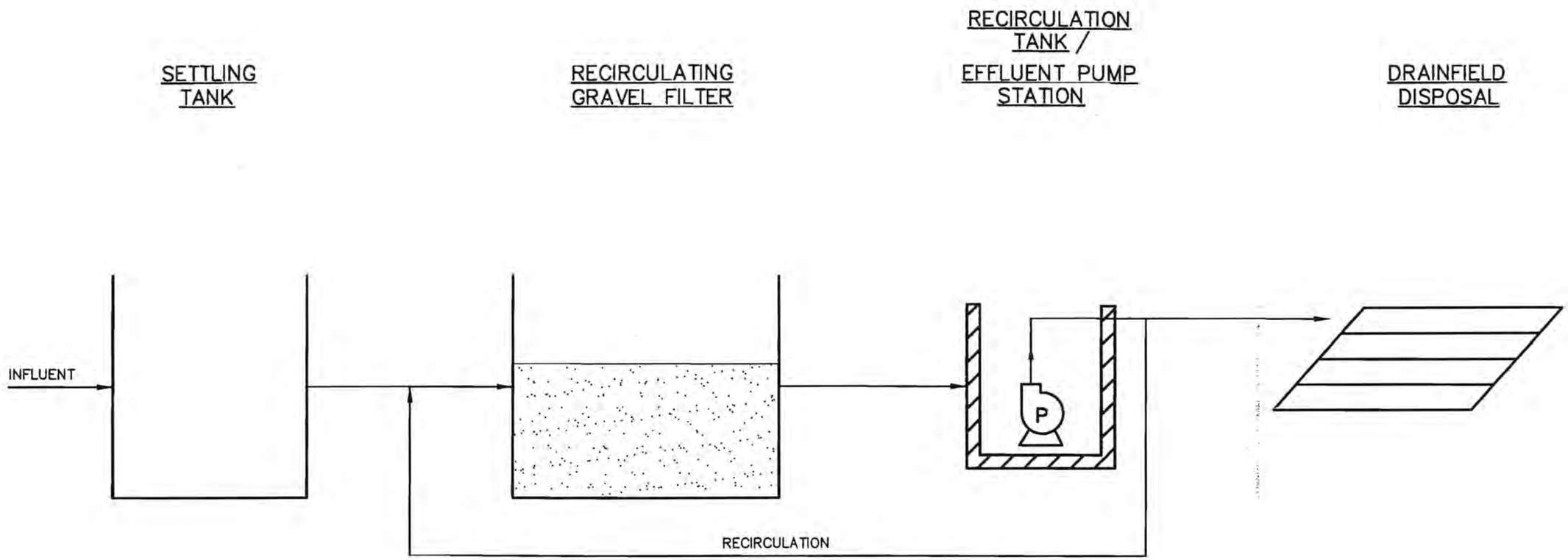
Similar to Alternative 1, the drainfield would discharge to Type 1A soils, therefore, sand-lined trenches are required to provide additional treatment to the wastewater. Based on an application rate of 1.2 gpd/ft^2 and a peak day flow of 14,500 gpd, the required drainfield trench area is 12,083 ft^2. The drainfield area would be laid out in four zones, with two additional zones provided for alternation of the application areas. A reserve area capable of handling 50 percent of the peak day design flow would also be provided for a future replacement drainfield area. With the required alternation zones, the total constructed drainfield area, including piping, would be 67,200 ft^2, or approximately 1.6 acres. Including the reserve area, the total drainfield area needed is about 2.1 acres. A process flow diagram for this alternative is presented in Figure 7-3. The design criteria are presented in Table 7-4.

TABLE 7-4

Preliminary Design Criteria for Alternative 2 - Recirculating Gravel Filter with Pressurized Drainfield

Component	Value
Settling tank	14,500 gallons
Filter	
Filter application rate	5 gpd/ft^2
Filter surface area, 2 zones	2,900 ft^2
Recirculation tank	21,750 gallons
Recirculation pumps w/ 1 redundant	3 @ 150 gpm, 30 ft TDH
Pump tank	2,000 gallons
Distribution pumps	2 @ 210 gpm, 30 ft TDH
Drainfield	
Drainfield application rate	1.2 gpd/ft^2
Drainfield zones, w/ alternation	6 @ 9,120 ft^2
Drainfield area needed, w/ reserve	2.1 acres
Orifice size	3/16"
Orifice spacing	6 ft
Lateral size	2"

Installing properly washed filter media that meets the DOH media specification is critical to the success of the RGF treatment. Past systems have failed due to the use of media that was too fine or media that contained too much fine material because it was not adequately washed. The filter media must meet the DOH criteria for particle size based upon a particle size analysis of the actual gravel material proposed for use. Each load of



TREATMENT PROCESS SCHEMATIC

MASON COUNTY PUD No. 1

FINCH CREEK WASTEWATER FEASIBILITY STUDY
SERVICE AREA 1 – ALTERNATIVE 2
RECIRCULATING GRAVEL FILTER
W/ PRESSURE DRAINFIELD

FIGURE 7–3

Gray & Osborne, Inc.
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media used in construction should be sieve-tested to assure media specification compliance.

The access road will need to be improved to provide adequate access to the site. Three phase power would be provided to the site to operate the recirculation and distribution pumps, and an onsite generator would also be supplied to provide power during outages. The estimated construction costs for constructing the treatment system are presented in Table 7-5, and the estimated O&M costs are presented in Table 7-6.

TABLE 7-5

Estimated Construction Costs for Alternative 2 - Recirculating Gravel Filter with Pressurized Drainfield

Item	Estimated Cost
Mobilization/demobilization	$53,520
Clearing & grubbing	$8,000
Sitework, excavation, waste hauling	$95,500
Filter/drainfield piping	$64,500
Filter/drainfield media	$62,370
Pumps, controls	$45,000
Tanks	$139,250
Drainfield geotextile	$8,540
Topsoil, mulch, seeding	$12,000
Electrical incl./ generator	$75,000
Road work/security	$25,000

Subtotal.....$588,680
Sales tax (7.9%).....$ 46,510
Subtotal.....$635,190
Contingency (20%).....$127,040
Estimated Construction Cost.....$762,230
Engineering, Legal, Administration (20%) $152,450
Total Estimated Capital Cost $914,680

Labor costs for system O&M were developed based on an estimated cost of $30 per hour to the PUD, which includes salary and overhead, and approximately four hours of oversight per week. The PUD may determine to contract for O&M of this facility; this could potentially result in lower labor costs than estimated here. Power costs are based on $0.056 per kilowatt-hour ($/kw-hr), and solids hauling costs are based on $0.13 per gallon for hauling and disposal of solids annually.

TABLE 7-6

Estimated O&M Costs for Alternative 2 - Recirculating Gravel Filter with Pressurized Drainfield

Item	Estimated Cost
Labor	$ 6,240
Supplies	$ 1,000
Utilities	$ 1,600
Solids hauling	$ 1,950
Repair and replacement	$ 2,500
Miscellaneous	$ 500
Total Annual O&M Costs	$13,790

Advantages to this treatment approach include a higher effluent quality than Alternative 1, providing for a smaller drainfield area. Constructing the higher level of treatment would significantly reduce the potential for drainfield problems or failure and would serve to extend its life. The O&M costs are relatively low; though more than Alternative 1 due to the higher complexity of the facility. The facility would require more oversight on a day-to-day basis than Alternative 1, as well as more maintenance.

The potential to expand this type of treatment to treat additional flows is fair. A higher level of treatment would probably be required by Ecology to accommodate additional flows, primarily to reduce nitrogen. Also, ground water monitoring would probably be required. Without wet weather monitoring data, it is not clear how much wastewater can be applied at this particular site, and there is very limited area available for drainfield expansion. However, it is feasible that additional land could be purchased from an adjacent landowner.

SERVICE AREA 2 TREATMENT ALTERNATIVES

The design flows and loadings for Service Area 2 were developed in Chapter 6 and presented in Table 6-3. They are as follows:

(FROM TABLE 6-3)

Service Area 2 Estimated Flows and Loadings

Parameter	Average Day	Peak Month	Peak Day
Flow (gpd)	25,415	44,595	56,005
BOD (ppd)	56	98	123
TSS (ppd)	56	98	123
TKN (ppd)	10	18.5	23

Service Area 2 includes the business district, approximately 3,000 feet of the Hood Canal shoreline, as well as the Finch Creek corridor. The peak day design flow for this service area is 54,680 gpd. Based on this design flow, the treatment alternatives evaluated must meet design criteria established by the Washington Department of Ecology.

Two treatment approaches were evaluated for Service Area 2, hereafter referred to as Alternatives 3 and 4. Alternative 3 would be designed to produce Class A reclaimed water for reuse at the WDFW fish hatchery in Hoodsport. Headquarters and regional staff for WDFW have expressed initial support of this proposal. The other treatment approach, Alternative 4, is an upland mechanical treatment facility that would use a sprayfield for final disposal to ground water. These treatment approaches for Service Area 2 are discussed below.

ALTERNATIVE 3: ACTIVATED SLUDGE/MEMBRANE FILTRATION WITH WATER REUSE

The reclaimed water treatment facility is proposed to be sited on a currently undeveloped piece of property located behind the Hoodsport Inn Restaurant. The required process basins and equipment would be enclosed in a building equipped with odor control equipment. A pipeline would carry the reclaimed water to the fish hatchery for reuse.

As discussed in Chapter 4, treated wastewater must meet Class A or Class B reclaimed water standards in order to be reused at the fish hatchery. The most critical criteria is the disinfection standard for total coliform of 2.2 colonies per 100 ml (TC/100ml) or less, which is the same for Class A or Class B reclaimed water. In order to meet this standard, the water must be highly treated prior to disinfection. Class A standards require that the water be filtered prior to disinfection. Class B standards do not require filtration, however, experience has shown that this disinfection standard can be difficult to meet if the water is not filtered prior to disinfection, therefore treatment to Class A standards is assumed. Treatment facilities that produce reclaimed water must also have a higher level of reliability and redundancy of equipment and alarms in order to ensure continued and consistent treatment of the water. With reclaimed water, protection of public health is

considered the highest priority. Also, for water reuse at the fish hatchery, ammonia and temperature would be parameters of concern.

The WDFW supplied the hatchery's average Finch Creek water intake data for 1997-98, which is presented in Table 7-7. The WDFW also provided its Finch Creek daily water temperature data for 1997 through 1999. Water temperatures were relatively consistent, ranging from a low of 38°F in February to a high of 53°F in August and September.

The peak day flow for the water reclamation facility is estimated at 54,680 gpd. Therefore, the reclaimed water will make up a very small percentage of the total water used at the hatchery. Generally, the ammonia concentrations and temperature of the reclaimed water would not have a measurable effect on the intake water. However, WDFW staff have indicated there may be specific uses at the hatchery particularly suited for the reclaimed water. In that event, the ammonia concentration and temperature of the reclaimed water could determine how the water is used. In general, WDFW believes the overall improvement in water quality in Finch Creek that would be provided by a community treatment facility would significantly benefit the hatchery and the Hoodsport shoreline area.

TABLE 7-7

WDFW Hatchery Finch Creek Water Intake Data - Average Usage for 1997-98

Month	Water Usage (MGD)
January	12.0
February	12.0
March	16.0
April	14.0
May	10.0
June	9.0
July	8.0
August	7.0
September	6.0
October	8.0
November	10.0
December	11.0
Maximum	16.0
Minimum	6.0

Alternative 3 would provide treatment with an extended aeration activated sludge process, a common technology for smaller facilities. However, with this approach, the solids in the aeration basin would be separated from the liquid by an in-basin membrane unit, rather than using conventional settling methods. The use of the in-basin membranes

creates a smaller footprint than conventional clarification and provides a very quality filtrate. Also, because ammonia is a parameter of concern in the wastewater for reuse at the fish hatchery, the treatment process would be designed to include nitrogen removal. Following membrane filtration, the effluent would be disinfected with ultraviolet (UV) radiation, and piped to the fish hatchery for reuse.

Ammonia removal is generally accomplished by acculturating bacteria in the aeration basins that convert the ammonia to nitrate, a form of nitrogen that is not toxic to fish and does not further reduce oxygen in the water. Because this biological conversion uses up a significant amount of alkalinity in the wastewater, it is generally desirable to also create an environment for denitrification, which results in some alkalinity being returned to the wastewater. Denitrification involves creation of a separate anoxic zone at the influent end of the aeration basin. In this anoxic zone, incoming wastewater is mixed with recycle from the downstream aerobic zone of the aeration basin. Within the anoxic zone, the wastewater is mixed but not aerated, creating an environment where nitrate in the recycled aeration basin mixed liquor is biologically converted to nitrogen gas. From the anoxic zone the wastewater flows to the aerobic zone where nitrification takes place. A portion of the liquid is then recycled back to the anoxic basin for denitrification.

The membrane microfilter system that would be used to clarify and filter the treated wastewater is a product produced solely by Zenon Corporation. Modules containing a large number of membrane strands can be placed directly into the aeration basin to provide clarification and filtration. Because the mixed liquor is drawn through membranes placed directly in the aeration basin, settling is not required. As a result, the mixed liquor concentration in the aeration basin can be maintained at three to four times the normally used concentration. This allows for much lower hydraulic detention times, and the necessary aeration basin volume is reduced accordingly. Subsequently, the footprint is significantly smaller when compared to conventional clarification and filtration.

A low pressure vacuum pump draws the mixed liquor through the micropores resulting in a very high quality filtrate. Fouling is controlled by air movement around the membranes and short (15 to 30 seconds), frequent backpulsing with a chlorine solution. Cleaning of the membranes is accomplished by backpulsing with a stronger chlorine solution every few hours. Dip tanks filled with a dilute acid are also provided within the aeration tank for more intensive cleaning of the membranes in the event of serious fouling.

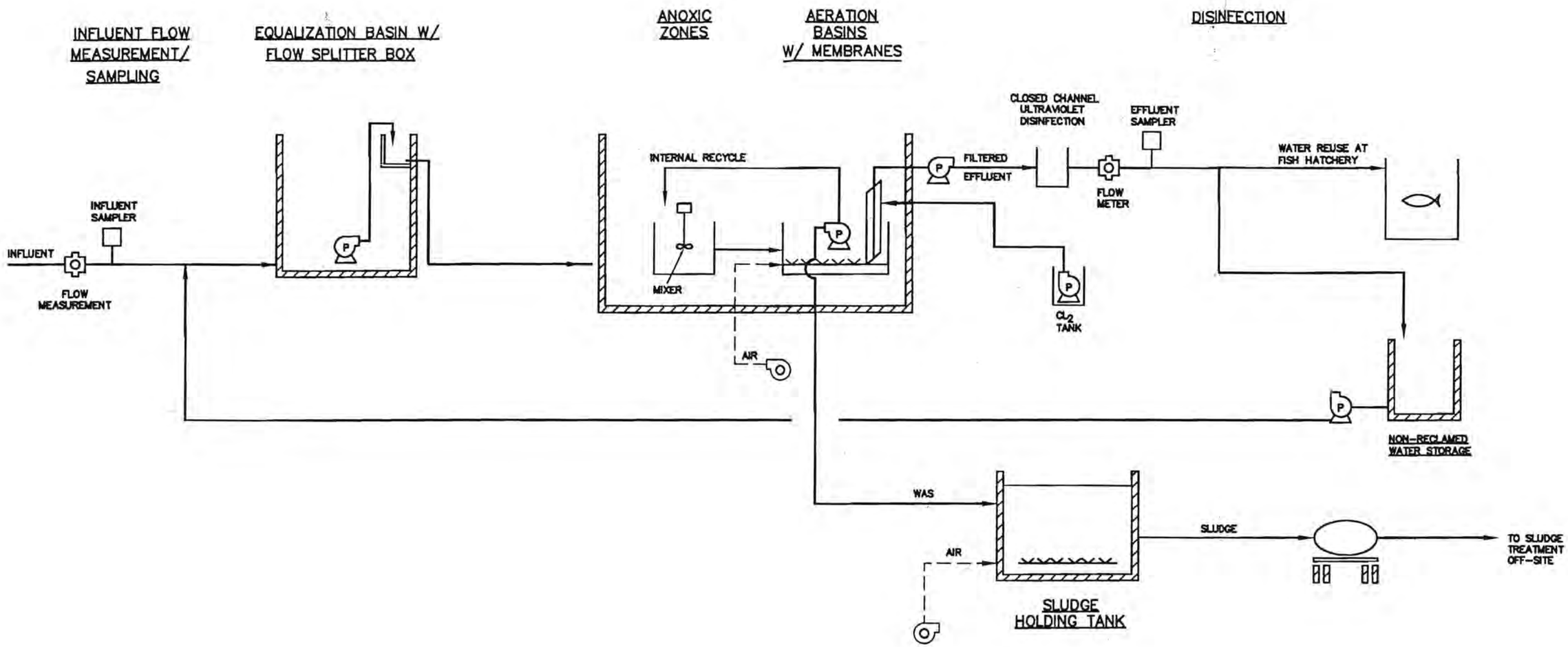
The chlorine solution could be standard bleach or some other concentration of sodium hypochlorite, whatever is readily available. This could then be diluted to the concentrations needed. The frequent backpulsing occurs every 15 to 30 minutes for 15 to 30 seconds; this requires a chlorine concentration of 2 to 10 mg/l. The backpulsing sequence is programmed, and can be adjusted based on cleaning requirements. More intense cleaning is recommended every 4 hours or so; this involves backpulsing for 15 to

30 seconds with a chlorine solution of 200 to 300 mg/l. This backpulsing would be put on a programmable timer with opportunity for adjustment. Using a standard 10 to 12 percent concentration of sodium hypochlorite, annual usage would be relatively small and inexpensive. Zenon will provide all of the necessary treatment process equipment and programmable logic controller (PLC), including aeration equipment and pumps. The company will also provide operator training for the system.

According to the Zenon representative, for this size of facility, pretreatment ahead of the aeration basins should consist of a grinder pump and sump. Screening is not considered necessary and is not recommended due to maintenance concerns. If the collection system provides pretreatment, such as a grinder pump or STEP collection system, the grinder pump would not be required. An equalization basin is recommended ahead of the reactor basins to even out flows to the aeration basins.

The UV radiation equipment will be sized to provide consistent disinfection to meet the total coliform standard of 2.2 TC/100ml. The system will also be designed with redundant process units and alarms, as required by the Reclaimed Water Standards. After the UV disinfection, the reclaimed water would flow by gravity to the fish hatchery for reuse. The treatment facility, including laboratory and equipment storage, would be enclosed in a 2,500 ft^2 building.

The Standards require that reliability measures be installed to ensure that water not meeting reclaimed water standards is not released from the facility. For this facility, it is recommended that 24-hour storage of the maximum month flow, as well as standby equipment for the aeration, membrane filtration, disinfection and pumping systems, be provided. Therefore, a 45,000 gallon storage tank would be located at the facility site. If any of the reclaimed water criteria were not being met, water would automatically be diverted to the tank. Once the facility was again operating within standards, this water would be returned through the treatment process. With alarms to alert operational staff of any problems and standby equipment on hand, such episodes should be infrequent and short-lived. The preliminary design criteria for Alternative 3 are presented in Table 7-8. A process flow diagram is shown in Figure 7-4.



TREATMENT PROCESS SCHEMATIC

MASON COUNTY PUD NO. 1

FINCH CREEK WASTEWATER FEASIBILITY STUDY
SERVICE AREA 2 – ALTERNATIVE 3
ACTIVATED SLUDGE W/ MEMBRANE FILTRATION
PROCESS SCHEMATIC
FIGURE 7–4

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TABLE 7-8

Preliminary Design Criteria for Alternative 3 - Activated Sludge/Membrane Filtration with Water Reuse

Component	Value
Expected effluent quality	
BOD	<5 mg/l
TSS	<5 mg/l
NH_3	<1 mg/l
Total nitrogen	<10 mg/l
Turbidity	<1 NTU
Total coliform	2.2 TC/100 ml
Influent grinder pumps, no.	2
Equalization basin	
Dimensions	12'W x 12'L x 11'WD
Volume	11,000 gallons
Process basins, no.	2
Dimensions, ea.	12'W x 11'L x 11'WD
Working volume, total	16,456 gallons
Design HRT, max. mo. flow	9.2 hrs
Design MLSS	12,000 mg/l
Design anoxic compartment volume	7,640 gallons
Design aerobic compartment volume	8,816 gallons
Membrane modules, no.	6
Aeration	
Type	Fine bubble diffusers
Blower, no.	2
Blower capacity, ea.	240 scfm
UV Disinfection, type	Medium pressure, in-vessel
Reclaimed water pipeline, length	1,600 ft
Non-reclaimed water storage tank	45,000 gallons
Biosolids storage tank	11,000 gallons

The facility must also have an alternative means of reuse, storage or disposal of the reclaimed water in the event the water cannot be reused at the hatchery. Because of the critical importance of continuing operations at the hatchery, such situations are expected to be very rare. Therefore, it is recommended that an outfall be provided off the reuse pipeline to divert the reclaimed water to Finch Creek, where it could be discharged in the event the water could not be used at the hatchery. It is probable that such an event would be known well ahead of time, and affected people and agencies could be notified. An NPDES permit would have to be obtained from Ecology for the potential discharge.

Because the mixed liquor in the aeration basins is maintained at 10,000 mg/l or greater, biosolids are wasted directly out of the aeration basins. The biosolids would be stored onsite and periodically hauled offsite for treatment and disposal at a permitted site in Mason County. Based on an estimated biosolids production of 240 gpd at maximum month flow, approximately 11,000 gallon storage tank would be required to provide 45 days of storage. The tank would be provided with aeration to further stabilize the biosolids and control potential for odors.

The estimated construction costs for Alternative 3 are presented in Table 7-9, and the O&M costs are presented in Table 7-10.

TABLE 7-9

Estimated Construction Costs for Alternative 3 - Activated Sludge/Membrane Filtration with Water Reuse

Item	Estimated Cost
Mobilization/demobilization	$ 94,500
Site work, excavation, waste hauling	$ 50,000
Influent grinder pumps	$ 45,000
Building, basins	$ 120,000
Membrane/sludge eqpt, incl. Blowers, pumps, diffusers	$ 300,000
UV disinfection	$ 35,000
Piping, valves	$ 80,000
Electrical	$ 80,000
Lab eqpt, meters, samplers, etc.	$ 50,000
Odor control	$ 25,000
Generator	$ 50,000
Non-reclaimed water storage tank w/ pumps	$ 50,000
Reclaimed water pipe to hatchery (1,600 ft)	$ 60,000

Subtotal.....$1,039,500
Sales tax (7.9%).....$ 82,120
Subtotal.....$1,121,620
Contingency (20%).....$ 224,330
Estimated Construction Cost $1,345,950
Engineering, Legal, Administration (20%) .....$ 269,190
Total Estimated Capital Cost $1,615,140

Labor costs for system O&M were developed based on 50 percent of a full-time operator contracted at an estimated cost of $34 per hour. Much of the operational control can be handled by programmable control systems, but the operator must be able to understand

the treatment concepts and make adjustments when necessary. A significant portion of the operator's time will be spent performing the monitoring and laboratory tests required by the discharge permit that will be issued for the facility from Ecology. Power costs are based on $0.056 per kilowatt-hour ($/kw-hr), and solids hauling costs are based on $0.13 per gallon for hauling and disposal of solids annually. The membrane modules will need to be replaced every five to seven years, at a cost of about $3,500 per module. The facility will have six modules; therefore, funds must be set aside to fund the $21,000 replacement cost every five to seven years.

TABLE 7-10

Estimated O&M Costs for Alternative 3 - Activated Sludge/Membrane Filtration with Water Reuse

Item	Estimated Cost
Labor	$35,360
Supplies	$ 3,000
Utilities	$12,000
Biosolids hauling and disposal	$ 6,000
Repair and replacement	$ 8,500
Miscellaneous	$ 1,000
Total Annual O&M Costs	**$65,860**

The advantages of this treatment approach are several. The facility has a small footprint and is able to be sited in the area where the wastewater is being generated. This saves a significant amount in transmission costs. The treatment process will produce a high quality of reclaimed water that can be reused in the community, with few limitations. The treatment process can be readily expanded, and the site is adequate to accommodate potential expansion. The primary disadvantage of this alternative is the high capital cost, as well as high O&M costs. As a reclaimed water facility, the process must be consistently meeting all requirements, resulting in a higher level of oversight and monitoring than conventional facilities.

ALTERNATIVE 4: SEQUENCING BATCH REACTOR WITH SPRAYFIELD APPLICATION

Alternative 4 would provide a mechanical treatment facility at an upland site, followed by discharge to a sprayfield. As discussed in Chapter 4, the ground water discharge would have to meet the requirements of the Ground Water Standards (Chapter 173-200 WAC) due to having a peak day design flow greater than 14,500 gpd. In order to meet the Ground Water Standards, the treatment technology proposed for the treatment facility should be capable of producing an effluent that contains a maximum total nitrogen concentration of 10 mg/l or less and 7 mg/l or less as an annual average. The treatment

facility should also have capability to consistently reduce BOD_5 and TSS in the effluent to 20 mg/L or less to enhance disinfection and reduce potential for clogging of the spray heads. A sequencing batch reactor (SBR) process is capable of meeting these design criteria, and is an appropriate technology for smaller systems. The SBR treatment process would be followed by disinfection with ultraviolet radiation, and the effluent would be applied to a sprayfield for final disposal.

The SBR process treats the wastewater in a cyclic batch mode within a single basin. A treatment cycle consists of filling, aeration, and settling, then decanting. Separate secondary clarifiers are not required since the activated sludge is settled in the reactor. Biosolids are wasted from the reactor basins on a periodic basis. Generally, at least two basins are provided so that the operations can be alternated, with one in the fill mode while the other is in treatment mode.

As previously discussed, nitrogen removal is generally achieved by creating an aerobic environment where oxidizing bacteria can convert the ammonia in the raw wastewater to nitrate, then an anoxic environment where denitrifying bacteria convert the nitrate to nitrogen gas. In the SBR, aerated and anoxic periods can be created and manipulated during the basin cycle to achieve nitrification and denitrification. The cycles in an SBR are programmed into a PLC that would be included as part of the SBR equipment. This operation is not difficult, but the operator would need to gain experience and have an understanding of how the treatment process works in order to make the appropriate adjustments. Each of the basins would be fitted with aeration, mixing and decanting equipment. For small systems such as this, the controls, equipment, and training are generally all provided by the manufacturer.

Since the SBR process operates in a batch discharge mode, an equalization basin is required downstream of the reactor basins or the UV system must be sized to handle the maximum batch flow. For this size of facility, it is generally more cost effective to size the UV for the batch flow. The effluent would flow through the UV disinfection channel and on to the sprayfield pump station, where it would be pumped to the sprayfield for final disposal.

The sprayfield is required to have 100 foot buffers, which will significantly increase the land area needed for the sprayfield. Also, since effluent cannot be applied when the ground is saturated or frozen, a storage lagoon would be required. Based on the preliminary hydrogeological analysis, saturated ground conditions would not be expected if the sprayfield is laid out carefully. Therefore sizing of the storage lagoon was estimated based on the number of consecutive days of freezing temperature, with additional storage for rainfall and minimal storage for ground saturation. Daily temperature and rainfall data taken at the Cushman Powerhouse were reviewed for 1990 through April 2000. The longest period of consecutive freezing days during this period was 21 days in December 1992 and January 1993. Rainfall should be pumped out of the

storage lagoon regularly so rainfall storage can be limited to non-application periods. The estimated storage requirement will be based on providing for 21 days of freezing and 5 days of saturation at a maximum month flow of 43,745 gpd, resulting in a volume of 1.14 million gallons. The storage lagoon surface area would be approximately 115 feet by 115 feet, with a water depth of 18 feet. An additional 2 feet (198,000 gallons) will be provided for rainfall storage, with 2 feet of freeboard.

Biosolids would be wasted from the SBR basins to a 20,000 gallon aerated holding tank. Periodically, the sludge would be hauled offsite for final treatment and disposal. The treatment facility would be provided with an emergency generator, as well as the necessary meters, samplers, and laboratory equipment. A process flow diagram for Alternative 4 is shown in Figure 7-5.

Sprayfield Hydrogeological Investigation

Gray & Osborne contracted with Ground Water Science Services (GWSS) to perform a preliminary evaluation of the Thurman property to determine its feasibility as a drainfield and sprayfield application area. The entire site includes approximately 14.4 acres, however, the hydrogeological investigation focused on the approximately 7 acres of land most suitable for effluent application. The scope of work for the evaluation included:

- Test pit soil investigation
- Shallow ground water sampling
- Infiltration testing
- Soil grain size analysis
- Site characterization
- Preliminary nitrogen modeling

Test Pits

On May 3, 2000,GWSS monitored the excavation of 6 test pits on the Thurman property, as well as examination of the 6-test pits already dug onsite. The new pits ranged from about 7 feet to 11 feet below grade. Ground water or signs of moisture were not found in any of the pits. At each pit, the hydrogeologist logged soils and collected samples at two to four foot intervals. Soil classifications presented on test pit logs are based on visual and laboratory observations. Test pits were backfilled with native soil immediately upon completion of logging. As previously noted, the location of these pits is shown on Figure 7-1, and the test pit logs are included in Appendix 2.

The predominant sediments encountered during the field exploration are listed below, in order of increasing depth:

0-1'	-Soft to stiff, organic brown to red fine sandy silts w/ gravel
1-2'	-Medium dense gravelly, sandy silts
2-10'	-Medium dense coarse sandy gravels and gravelly sands
>10'	-Dense gray till/perching layer

However, in a couple of the pits on the western side of the property, a thin layer of clay was found at about 5 feet of depth.

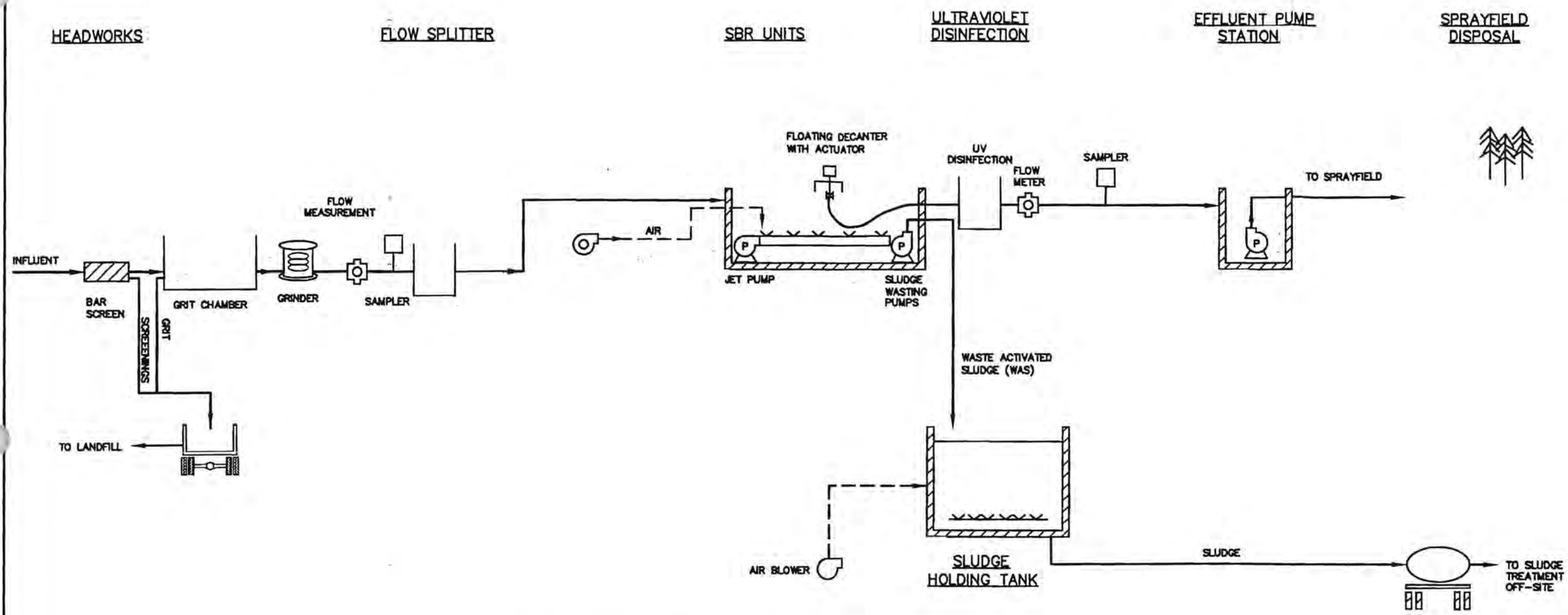
Selected samples were sent to Soil and Plant Laboratory, Inc. in Santa Clara, California and analyzed for the following parameters:

- USDA grain size distribution
- pH
- Salinity
- Organic content
- Half saturation percentage
- Cation exchange capacity

Soil samples submitted for analysis were classified under the USDA soil classification as predominantly loamy sand, very gravelly, or sandy loam, very gravelly, in the upper 1 to 2 feet. This material is underlain by sand, very gravelly or gravels, very sandy, to depths up to 8 to 10 feet below ground surface. There is a predominance of silt size particles in the matrix and greater clay content in the upper 1 foot of the soil column. Based on the USDA classifications, the predicted infiltration rates for these soils would be from 2.4 inches per hour (in/hr) for the sandy loams, very gravelly to 20 in/hr for the deeper sands, very gravelly. Based on the EPA manual, "Land Treatment of Municipal Wastewater (1981)", a recommended application rate for wastewater is 4 to 10 percent of the "clean water" infiltration rate. Using 4 percent of the clean water infiltration results in a recommended hydraulic application rate of 2.3 to 19 inches per day.

Cation exchange capacity is the total amount of exchangeable cations that a soil can adsorb, and is expressed in milliequivalents per 100 grams of soil. Cation exchange capacity was typical for sandy mineral soils with low organic content, ranging from 14 to 27 meq/100g. Typical values range from less than 10 meq/100g for inorganic granular sediments, to over 100 meq/100g for clays or organic soils. Organic content of the soils analyzed was generally low, ranging from 0.2 to 2.6 percent. Site soil pH was generally slightly to moderately acidic, ranging from 4.1 to 6.1.

Based on the grain size, organic content and cation exchange capacity determined for the soils at the Thurman site, the soils will provide a relatively high capacity for water infiltration; however, the site will provide a relatively low level of retention in the soils and subsequent treatment of dissolved constituents. As a result, applied wastewater would be expected to move down to the confining layer. Some would infiltrate, while some would move laterally. Because the site is bounded by steep slopes and a small streambed, some of the effluent would likely migrate offsite before it could infiltrate through the confining layer.



TREATMENT PROCESS SCHEMATIC

MASON COUNTY PUD NO. 1
FINCH CREEK WASTEWATER FEASIBILITY STUDY
SERVICE AREA 2 – ALTERNATIVE 4
SEQUENCING BATCH REACTOR W/ SPRAYFIELD
PROCESS SCHEMATIC
FIGURE 7–5

Gray & Osborne, Inc.
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Permeability/Hydraulic Conductivity

Hydraulic conductivity was estimated from grain size distribution curves using the Hazen method. Using this method, hydraulic conductivity for the soils ranged from 0.45 feet per day (ft/day) to an intermediate value of 45 ft/day and as high as 283 ft/day.

Ground Water Sampling

Since ground water was not encountered in the test pits, a background water sample was obtained from the small stream on the north portion of the property. The stream appears to surface as a spring just upstream of the proposed site. The sample was analyzed for the following parameters: alkalinity, bicarbonate, carbonate, BOD, conductivity, hardness, total and fecal coliform, total dissolved solids, TKN, total phosphorous, turbidity, major anions (fluoride, chloride, nitrite, nitrate, and sulfate) and total metals (barium, beryllium, chromium, copper, iron, manganese, nickel, sodium, and zinc. The results of these analyses indicated very low concentrations for all of the parameters except for a total coliform of 17 TC/100 ml. The area is relatively untouched by human occupation, so impacts are likely to be from wildlife in the area. The area is well above domestic wells located along Finch Creek.

Ground Water Modeling

Preliminary modeling was performed by GWSS using an analytical nitrogen loading model to estimate potential impacts to ground water. The values for model input were derived from ground water and soil data developed during the site investigation and from WWTP design parameters. The model simulated an annual effluent discharge of 25,000 gallons per day (gpd) with an average effluent nitrogen concentration of 7 mg/l. Generally, allowable nitrogen loading is based on an annual average value due to the continual nature and long-term impacts of the discharge. An effluent nitrogen concentration of 7 mg/l equates to a nitrogen loading of 533 pounds per year applied at average design flow of 25,025 gpd.

The model estimates annual nitrogen loads infiltrating over a given area, as well as the nitrogen uptake expected from the vegetation at the site. Plant uptake is expected to be high in the primary growing season and moderate in the fall and spring. Dilution from rainfall will significantly reduce nitrogen concentrations during the non-growing season. At this site, rainfall can average as much as 100 inches per year, most of it falling in the winter months when plant uptake is minimal. Based on the preliminary GWSS modeling, a 4.5 acre sprayfield would be sufficient to keep ground water nitrate levels at about 2 mg/l or less. Based on a sprayfield area of 4.5 acres, the nitrogen loading would be 119 pounds of nitrogen per acre per year (ppa/yr) for an annual flow of 25,025 gpd. With the required buffer area, the total area required for the facility is estimated to be

approximately 10 acres. The preliminary design criteria for Alternative 4 are presented in Table 7-11.

TABLE 7-11

Preliminary Design Criteria for Alternative 4 - Sequencing Batch Reactor with Sprayfield Application

Component	Value
Expected effluent quality	
BOD	≤10 mg/l
TSS	≤10 mg/l
NH_3	≤2 mg/l
Total nitrogen	≤10 mg/l
Effluent fecal coliform	200 FC/100ml
Grit chamber	
Design flow velocity	1 ft/s
Grit removal	Manual
Grinder, no.	1
Size	4"
Batch reactor basins, no.	2
Dimensions, ea.	15'W x 26'L x 12'WD
Volume, ea.	40,500 gallons
Design SRT	27 days
Design MLSS	3,500 mg/l
Design F/M ratio	0.15
Aerators, no.	2
Motor hp	10
Estimated oxygen transfer	18 lb/hr O_2
Decanters	
Type	In-tank weir
Rate	120 gpm
Cycle time	
Fill	3 hrs
React	1.5 hrs
Settle	0.75 hrs
Decant	0.75 hrs
Cycles/day	4
UV Disinfection	Open channel, low pressure
Sprayfield, acres	4.5
Storage lagoon, volume	1.14 MG

TABLE 7-11 – (continued)

Preliminary Design Criteria for Alternative 4 - Sequencing Batch Reactor with Sprayfield Application

Component	Value
Dimensions	115'L x 115'W x 18'WD
Biosolids storage tank	20,000 gallons
Aerator, no.	1
Required oxygen transfer	16.7 lb/hr O_2

As a result of the preliminary GWSS hydrogeological investigation, the suitability of the Thurman property for a sprayfield was determined to be marginal. This concern is primarily due to the area being bounded by steep slopes and a small drainage on the north side of the property. Without wet weather monitoring data and further hydrogeological evaluation, it is not possible to recommend the site for a sprayfield that would receive flows from the larger service area, Service Area 2.

Northwest of the Thurman property is a large amount of property owned by Mount Washington Tree Farm. This property is at least 2,000 feet further up the same logging road than the Thurman property and at an elevation approximately 80 feet higher. Therefore, transmission to the site would be more costly. Power would have to be constructed to the site, and the road would need to be improved. The owner has indicated a willingness to consider having a treatment facility on the property. A preliminary review of the property and available hydrogeological information suggests it is likely of similar subsurface geology as the Thurman property. An in-depth hydrogeological study would need to be performed on the site to develop the final design and layout of the sprayfield. Transmission cost estimates for this alternative will be developed based on use of the Mount Washington site.

Should a sprayfield system be the recommended alternative for implementation, ground water monitoring should be conducted at the chosen site prior to installation of the system. Based on the Ecology guidance document, *Implementation Guidance for the Ground Water Quality Standards* (1996), the collection of ground water samples from a monitoring wells is recommended at least 8 times over a full year, and a statistical evaluation of the data completed to determine ambient ground water quality. Water samples should be analyzed for the following constituents:

- Nitrate
- Total Kjeldahl nitrate (TKN)
- Ammonia
- Phosphate
- Sodium
- Bicarbonate
- Carbonate
- Chloride

- Biological oxygen demand (BOD)
- Total dissolved solids (TDS)
- Total suspended solids (TSS)
- Total coliform bacteria
- Calcium
- Potassium
- Sulfate
- Total iron
- Total manganese
- Dissolved iron
- Dissolved manganese

The following field parameters should also be measured and recorded:

- pH
- Conductivity
- Dissolved oxygen
- Temperature

Water levels should be measured, the data reduced to a common datum and the ground water gradient calculated. This data could be collected in conjunction with the in-depth hydrogeological investigation that would have to be performed prior to design of the facility.

The estimated construction costs for Alternative 4 are presented in Table 7-12, and the O&M costs are presented in Table 7-13.

TABLE 7-12

Estimated Construction Costs for Alternative 4 - Sequencing Batch Reactor with Sprayfield

Item	Estimated Cost
Mobilization/demobilization	$139,350
Sitework, landscaping	$70,000
Headworks	$30,000
SBR basins, equipment (instruments, pumps, blowers, etc.)	$240,000
Piping, valves	$70,000
Electrical incl.generator	$180,000
UV disinfection	$30,000
Samplers, meters, lab equip, etc.	$50,000
Operations building	$70,000
Sludge storage tank, w/pump, aeration	$50,000
Sprayfield pump station	$40,000
Sprayfield	$223,500
Ground water monitoring eqpt.	$40,000

Storage lagoon	$185,000
Hydrogeological study	$40,000
Road work/security	$75,000

Subtotal $1,532,850
Sales tax (7.9%) ..... $ 121,090
Subtotal $1,653,940
Contingency (20%) $330,790
Estimated Construction Cost $1,984,730
Engineering, Legal, Administration (20%) ..... $ 396,950
Total Estimated Capital Cost $2,381,680

Labor costs for system O&M were developed based on 60 percent of a full-time operator contracted at an estimated cost of $32 per hour. As with Alternative 3, much of the operational control can be handled by programmable control systems, but the operator must be able to understand the treatment concepts and make adjustments when necessary. There will also be O&M of the sprayfield to be performed. A significant portion of the operator's time will be spent performing the monitoring and laboratory tests required by the State Waste Discharge Permit that will be issued for the facility from Ecology. Power costs are based on $0.056 per kilowatt-hour ($/kw-hr), and solids hauling costs are based on $0.13 per gallon for hauling and disposal of solids annually.

TABLE 7-13

Preliminary Estimated O&M Costs for Alternative 4 - Sequencing Batch Rector with Sprayfield

Item	Estimated Cost
Labor	$39,940
Supplies	$ 3,000
Utilities	$13,500
Sludge hauling and disposal	$15,000
Repair and replacement	$ 5,000
Ground water monitoring	$20,000
Miscellaneous	$ 1,000
Total Annual O&M Costs	**$97,440**

The advantages of this treatment approach are several. The treatment process will produce a high quality of effluent that during the growing season can be used beneficially for irrigation. The SBR process is similar to others in the area at Harstene Pointe and the new facility currently under construction at North Bay, so O&M could be supported or contracted out to Mason County, who runs the other two facilities. The treatment process can be readily expanded, and the site is adequate to accommodate potential expansion.

The primary disadvantage of this alternative is the high capital cost, as well as the high cost of transmission to the facility. The O&M costs are also higher than Alternative 3, primarily due to the cost of ongoing ground water monitoring.

INDIVIDUAL ONSITE SYSTEM ALTERNATIVES

The alternative to developing a community wastewater system for either of the two designated service areas is to continue use of individual onsite systems. Many of the onsite systems along Finch Creek and Hood Canal are old and constructed in locations that are not favorable for wastewater disposal. They will in time have to be repaired or replaced. The presence of high ground water and proximity to surface water generally preclude the use of conventional onsite systems. County records indicate that replacement systems in these areas over the past several years have been designed as alternative systems such as sand filters, Glendon Biofilters, Biolac treatment units, etc. Local installers have indicated that residential onsite systems to satisfy requirements along Finch Creek and the shoreline generally cost $13,000 to $18,000 to construct. Commercial systems would be more costly, depending on their design flow. However, once the individual system is installed, ongoing O&M costs are relatively small compared to a community system.

With individual onsite systems, some residents and businesses may need to replace their systems, but others may not have to for some time. And the costs for repair or replacement will vary from site to site, depending on how inadequate the site is. Some would view that as an advantage compared to a community system where everyone is affected, but somewhat equally.

The main disadvantage to continuing with the use of individual systems is that the existing water quality impacts will continue or get worse. As a result, the shoreline will continue to be closed to shellfish harvesting due to concerns for public health. This impacts local residents, but also limits the community's ability to attract tourism. Businesses in the community are very limited in their potential to expand due to the limitations of their onsite systems. Some businesses could be put in jeopardy if their systems failed. Therefore, the economic impacts to the community could be significant if the service areas being evaluated continue to be served by onsite systems.

CHAPTER 8

CHAPTER 8

ALTERNATIVES FOR COMMUNITY COLLECTION SYSTEMS

Gravity sewer systems have been the mainstay approach to wastewater collection for hundreds of years. Over the past 25 years or so, however, with the introduction of new technology and improvements to existing technology, several viable alternatives to gravity sewers have developed. These "alternative" systems include low-pressure sewers, septic tank effluent pump (STEP) and grinder pump systems, vacuum sewers, septic tank effluent gravity (STEG) sewers, and variable grade sewers (VGS). In some cases, an alternative system may provide a more cost-effective approach than a conventional gravity system.

COLLECTION SYSTEM CONSIDERATIONS

In laying out a proposed wastewater collection system, there are several factors that must be considered. These include service area topography, geology, ground water, design flows, system layout and potential for future expansion. Elevations within the study area range from near sea level along the shoreline of Hood Canal to approximately 100 feet at the western end of Finch Creek Road.

Surface soils in the area are gravelly with hardpan at shallow depths. The hardpan layer results in a seasonally high ground water table. The high ground water table and proximity to the Hood Canal shoreline are important considerations for the design and construction of a wastewater collection system within the study area.

DESCRIPTIONS OF COLLECTION SYSTEM ALTERNATIVES

The collection system alternatives that will be evaluated for each of the two service area scenarios include conventional gravity sewers, STEP and grinder pump low pressure sewers, and vacuum sewers. Each of these will be described, including design issues and advantages and disadvantages that pertain to their application in Hoodsport. Later in this chapter, system layouts, as well as construction and operation and maintenance costs will be developed.

GRAVITY COLLECTION SYSTEM

Gravity collection systems are constructed with pipes 8-inch in diameter or greater in a continual downward slope great enough to create self-cleaning velocities of 2 feet per second (ft/s) or more when flowing full. Areas of level and rolling topography may

require deep excavations or the installation of sewage lift stations. The primary advantages of gravity systems are their reliability and low operation and maintenance (O&M) costs. Also, improved pipe materials and construction methods have reduced the potential for infiltration and inflow (I&I) into the sewers in recent years.

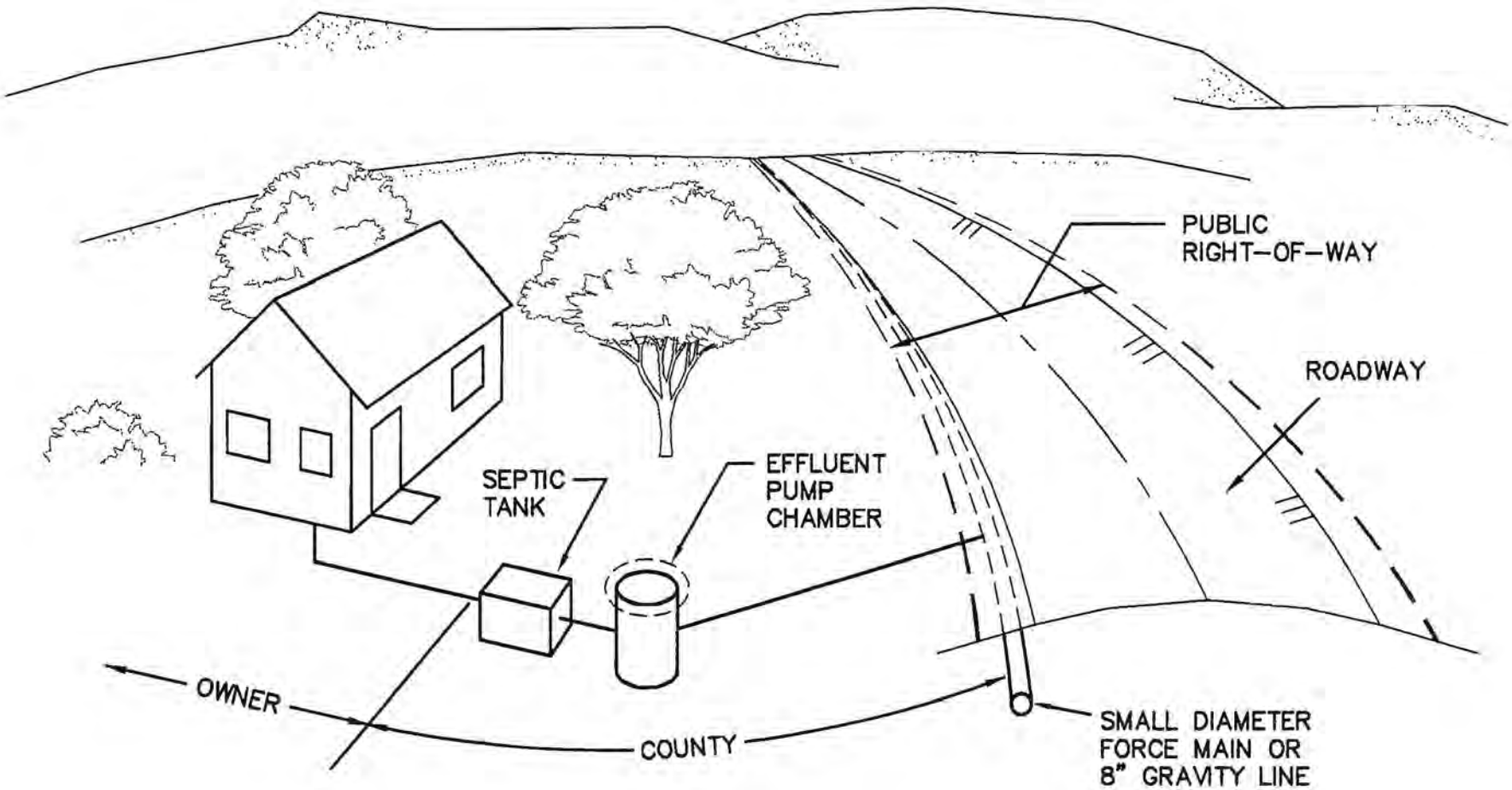
A significant disadvantage of gravity systems is the high construction cost, due primarily to deeper excavation, wider trench widths, and larger pipes than alternative sewers. In addition, dewatering operations are more costly in shoreline areas and areas of high ground water because of the increased excavation requirements.

Along Finch Creek and the Hood Canal shoreline, the location of some residences and businesses below the main collection line would require deep excavation or individual grinder pumps to lift the sewage to the gravity mainline. Also, the gravity system and portions of side sewers would be in ground water for extended periods of time, increasing the potential for infiltration into the system. For construction along Highway 101 and Finch Creek Road, the depth of the pipe and width of the pipe trench would have a significant impact on traffic. The ability to detour traffic around the construction is limited by the lack of alternate routes.

SEPTIC TANK EFFLUENT PUMP (STEP) SYSTEM

The septic tank effluent pump (STEP) system is one of two low pressure sewer systems in use today, the other being the grinder pump system. Each of these systems requires onsite pumping to transport the wastewater into the mainline sewer and to the treatment facility. A STEP sewer system also requires an onsite septic tank to pretreat the wastewater prior to entering the mainline sewer. The wastewater is detained in the septic tank for 2 to 3 days to settle out the heavier solids; solids removal of 50 to 60 percent is common. The low horsepower STEP pump, which is activated by a float located in the septic tank, pumps the settled wastewater into the main sewer line. Because the heavier, bulkier solids have been removed and the wastewater is under pressure, the mainline sewer can be much smaller than conventional gravity sewer, usually 2 to 4-inch diameter PVC piping.

The size of the mainlines also can be reduced because the septic tanks attenuate the peak flows. And, because the system is not dependent on gravity flow, the lines can be installed at about 3 feet below ground and follow the topography, greatly reducing excavation costs. Figure 8-1 shows a typical STEP system configuration. Some communities in western Washington that have STEP collection sewers exclusively or in part of the system are Montesano, Boston Harbor, Yelm, Lacey and Camas.



NOTE:

EFFLUENT PUMP CHAMBER MAY BE IN SEPTIC TANK

MASON COUNTY PUD NO. 1

FINCH CREEK WASTEWATER FEASIBILITY STUDY

TYPICAL STEP PUMP CONFIGURATION

FIGURE 8-1

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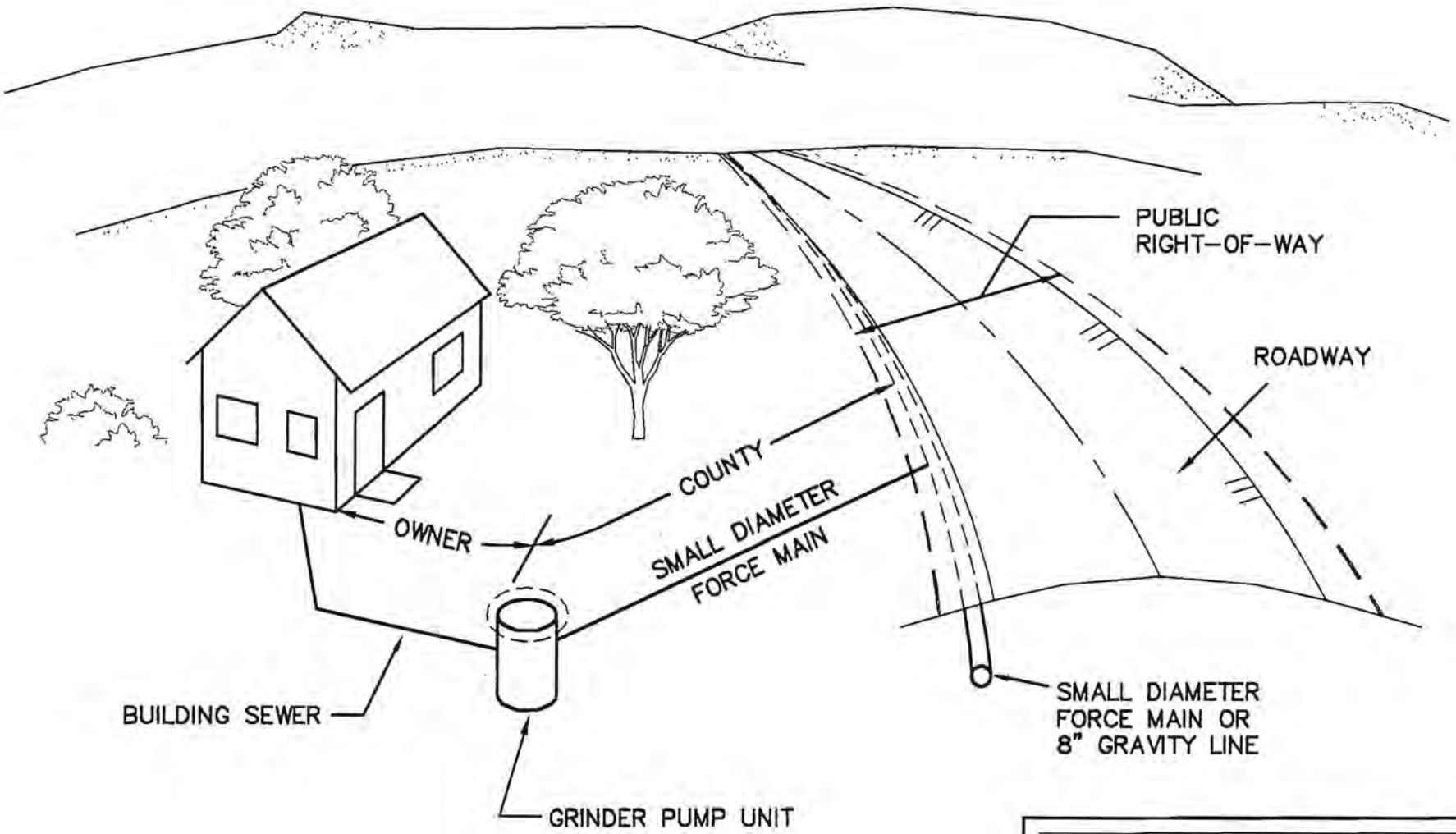

MASON COUNTY PUD NO. 1

FINCH CREEK WASTEWATER FEASIBILITY STUDY

TYPICAL GRINDER PUMP CONFIGURATION

FIGURE 8-2

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Design Issues

Construction of a STEP collection system can incorporate existing septic tanks if the tanks are determined to be suitable. The tanks need to be of correct size, have adequate baffling, and be located in an accessible area. They also should be pumped and evaluated for structural integrity and leakage. If feasible, the tanks should be properly waterproofed to prevent ground water infiltration into the tanks. Alternative sewers are generally not designed to handle extraneous flows such as I&I. Excessive infiltration could result in frequent pumping and overloading to the point some upstream units might not be able to pump into the lines.

On a property-to-property basis, the necessary information on existing tanks can be difficult and costly to obtain. For this reason, the use of new tanks is a common requirement in the design of STEP systems. This ensures the utility a certain amount of control over the flow entering the tanks in terms of a leakage guarantee from the manufacturer, elimination of illicit connections, etc. For the purpose of developing planning cost estimates for this plan, it will be assumed that new septic tanks would be installed on each property.

If the existing tank is used, a pump chamber must be placed downstream of the septic tank. If a new tank is installed, the pump chamber can either be located within the tank or in a separate chamber just downstream from the tank. In either case, a screen must be installed to prevent excessive solids from entering the pump and collection lines. Effective screening is critical because the STEP pumps and collection lines are not designed to handle solids and can become clogged if solids are sizable and excessive. The screens must be maintained to prevent grease buildup and clogging of the screen.

Ecology generally requires that the onsite STEP tanks and pumps be owned, operated, and maintained by the managing utility. Therefore, the utility must obtain easements or rights-of-entry in order to access private property for the installation and maintenance of the onsite equipment. An alarm system is required to signal pumping malfunctions. The septic tanks must be checked and pumped on a regular schedule to ensure excessive solids are not drawn into the pump and discharged to the collection system. Experience with other STEP systems has shown that residential tanks should be pumped every 5 to 7 years. Commercial establishments that produce a large amount of solids and oil and grease would need to be monitored more closely and pumped more frequently. Therefore, the utility must plan for solids pumping and disposal and include this in the O&M costs.

Power must be provided to the tank for operation of the STEP pump, which normally requires 110V power. Some older homes may require a service panel upgrade to accommodate the STEP pump. The homeowner is normally responsible to make any

upgrades necessary to the residential service and for providing a point of connection. Although the utility maintains the alarms and electrical components of the STEP system, the power bill is generally paid by the property owner. In the event of a power failure, wastewater is stored in the septic tank. An emergency generator plug connection can be provided at the tank to run the pump in the event of an extended power outage or the utility could have the tanks pumped during that period.

Advantages and Disadvantages

There are several advantages with installation of a STEP sewer system. Construction of the mainline sewer using small diameter pipes buried at shallow depths is often less costly than the construction of conventional sewers, particularly where the topography makes a gravity sewer expensive to install. Materials and excavation costs are less, and dewatering requirements in areas of high ground water are reduced. Also, the collection piping can be installed on the shoulder of the road, which minimizes road restoration costs and impacts to traffic flow. This would be particularly critical if sewer is installed along Highway 101. Because the STEP pumps do not have to handle solids, higher head pumps can be used, reducing or eliminating the need for additional lift stations. Also, because the septic tanks remove a significant percentage of solids and organic loading, the amount of solids and organic material that must be treated at the treatment facility is reduced. Subsequently, some treatment facility components can be sized smaller, resulting in cost savings. Finally, if they are well constructed, the systems will experience very little I&I.

The primary disadvantage of STEP sewer systems is the cost of the STEP tanks and pumps and ongoing maintenance of the onsite components at each property. The utility must have a program for checking and disposing of the septic tank solids apart from solids handling requirements at the treatment facility. Also, there can be occasional malfunctions with alarms, floats, and pumps, and plugging of the screen and pump. With the increased complexity of the sewer system, there is more potential for problems. Easements and rights-of-entry must be acquired from each property owner for installation and maintenance of the onsite components.

There is also increased potential for odors and corrosion with the STEP system. Because the wastewater is detained in the tanks for 1 to 3 days or more, the waste becomes septic. As the waste begins to break down, the dissolved oxygen (DO) is used up and hydrogen sulfide is generated; hydrogen sulfide is odorous and corrosive. If downstream pumping is required, then oxygen injection, bioxide treatment, air scrubbing, or some other treatment might be needed to control odor and corrosion at lift stations and the treatment facility.

GRINDER PUMP SYSTEM

Like the STEP sewer system, the grinder pump system is a low pressure sewer. The system consists of an onsite grinder pump at each property that pumps into the small diameter mainline collectors. When the wastewater level in the pump tank reaches a given level, the grinder pump is actuated, and the solids are ground as the wastewater is pumped into the collector line. Grinding the solids permits the use of small diameter mainline pipe without creating clogging problems. The mainline sewer may be sized slightly larger than the STEP lines because of the need to transport the solids. Generally, a holding tank is also provided for temporary storage during power outages. A typical configuration of a grinder pump sewer is shown in Figure 8-2. Utilities in western Washington that utilize grinder pumps are Pierce County, Eklund Park (Pacific County), and Kitsap County Sewer District No. 5 (Port Orchard). The North Bay collection system currently under construction in Mason County will also use grinder pumps for much of the system.

Design Issues

The grinder pump and holding tank are typically located on the property they serve. As with the STEP sewer, the utility is expected to own, operate and maintain the grinder pumps, requiring an easement and right-of-entry from the property owner for installation and maintenance. The holding tank should have sufficient storage volume for periods of power loss. If the history of power loss is documented to be infrequent and of short duration, storage requirements may be reduced. Some utilities have installed just the pump tank, which is about 60 gallons. Others have used existing septic tanks for emergency storage. An alarm system is required to signal pump system malfunctions.

There are two types of grinder pumps in use today, the centrifugal type and the positive displacement type. Experience has shown that positive displacement grinder pumps are significantly less expensive to operate and maintain and provide a longer service life. In Pierce County, where grinder pumps have been used in some areas for many years, the mean life of a centrifugal residential grinder pump has been about 6 years. The service life of the positive displacement residential grinder pumps has been 10 years or more. The centrifugal residential grinder pumps have also experienced more problems with clogging and jamming, which increases operation and maintenance costs. For example, a study in Pierce County indicated their O&M costs for centrifugal pumps were $171 to $317 per year while costs for positive displacement pumps were $30 to 74 per year. The grinder pump requires 220V power, and the homeowner is normally responsible to make any upgrades necessary to the residential service and for providing a point of connection. As with the STEP sewer, the utility is responsible for the installation, operation, and maintenance of the electrical connection; the property owner pays the power cost.

Advantages and Disadvantages

The advantages of the grinder pump system over a conventional gravity sewer are similar to those of the STEP system, primarily reduced mainline construction costs. The main advantage of the grinder pump over the STEP system is that the septic tank is not required to remove solids. Grinder pumps have the ability to handle solids, where STEP pumps cannot. As a result, the grinder pumps are less susceptible to clogging and can better handle waste from commercial sites that produce increased levels of solids and grease. Also, without the septic tank, the system is less susceptible to I&I problems.

Similar to STEP, the main disadvantage of the grinder system is the high cost of the onsite grinder pump installation and the ongoing maintenance cost. When compared to the STEP pump, the grinder pump is larger and more expensive, although both types have relatively low power costs.

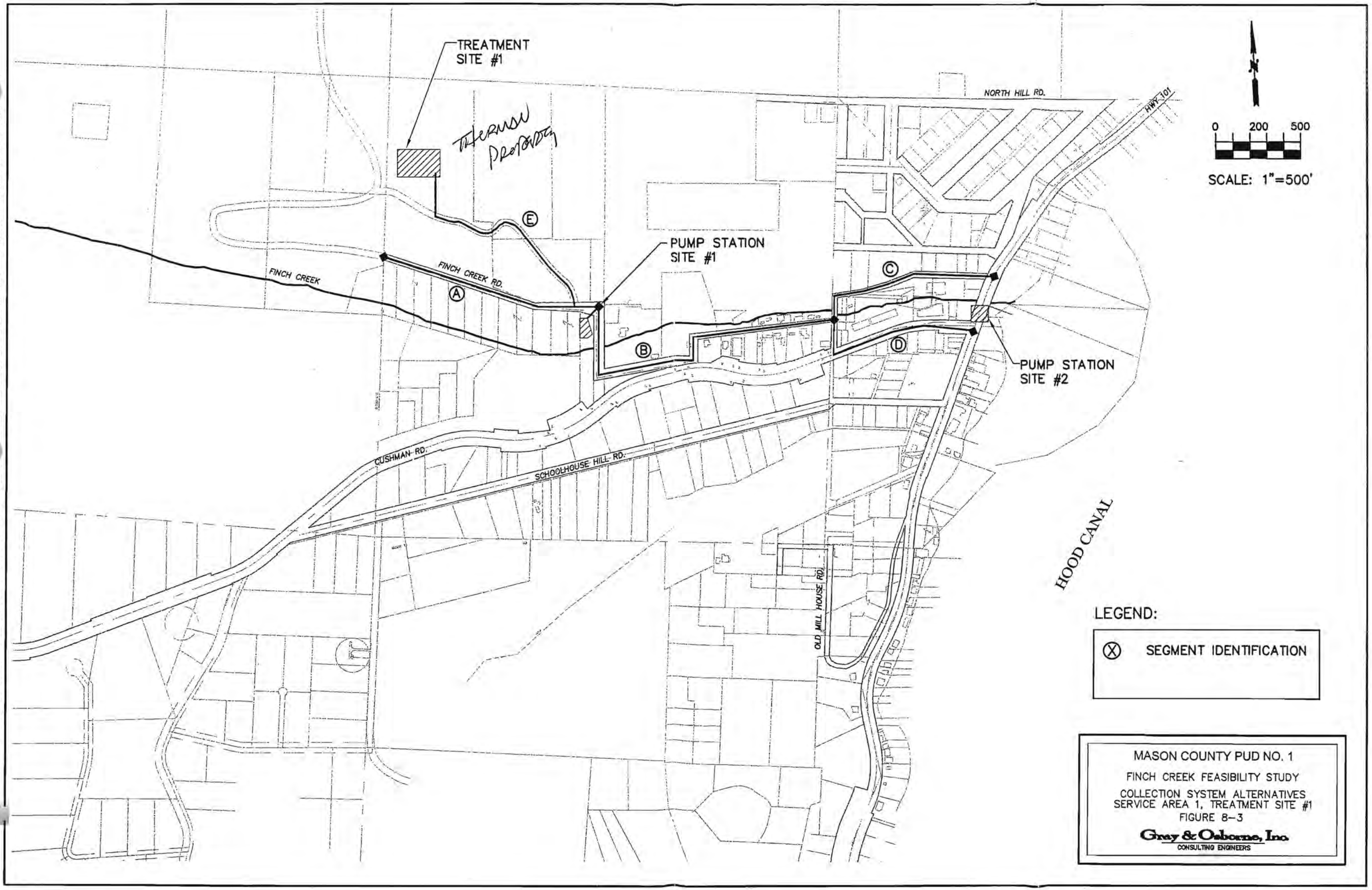
SEPTIC TANK EFFLUENT GRAVITY AND VARIABLE GRADE SEWERS

Septic tank effluent gravity (STEG) and variable grade sewers (VGS) are variations of the same approach. These systems are similar to STEP sewers in that they utilize septic tanks to remove solids. However, the effluent from the tank flows by gravity. Since the mainlines do not have to be designed to carry solids, they can be smaller than conventional gravity sewers. The size of the mainlines also can be reduced because the septic tanks attenuate the peak flows. The slopes of the pipes can be less than with conventional sewers because a flushing velocity, i.e. 2 ft/s or more, does not have to be maintained to move solids along. This results in a shallower depth of excavation.

The grade of the VGS may vary with the topography and actually go uphill as long as the upstream flow will not back up into any of the septic tanks. The STEG sewer will have a constant downward grade. When required by the hydraulic gradeline, each system requires a conventional pump station to transport the flow. Manholes are installed with these sewers, but fewer manholes are needed, typically every 400 to 1,000 feet compared to every 300 to 500 feet with conventional gravity sewers. Similar to the STEP sewer, the wastewater in these systems can be corrosive and odorous. Odor control equipment may be necessary at manholes, lift stations and the treatment facility. Corrosion potential would also have to be addressed. Septic tank effluent gravity sewer was installed in combination with STEP sewer in the community of South Prairie in Pierce County. There are no other known installations of STEG sewer or VGS sewer in Washington.

VACUUM SEWER SYSTEM

The vacuum sewer system is less common in Washington than the low pressure alternative sewer systems. However, they can be more cost-effective in some situations where head requirements are not excessive. The vacuum systems consist of a central



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vacuum station connected to each property by small diameter PVC vacuum piping. A holding tank with a special vacuum valve is installed at each property. Vacuum pumps in the central station maintain vacuum pressure in the system. The vacuum valve at each holding tank seals the line leading to the collection main so that the required vacuum pressure can be maintained in the main. When a given amount of wastewater accumulates behind the valve, the valve opens and allows wastewater to be drawn into the main. The wastewater is collected to the central vacuum station where it is transported by conventional pumps on to the treatment facility. Currently, there are three known installations of vacuum sewers in western Washington. These systems are Salmon Beach, which pumps into the City of Tacoma collection system, and recent installations in the City of Ocean Shores and in the Grand Mound community of Thurston County.

Advantages and Disadvantages

Advantages of the vacuum sewer are similar to those of the low pressure systems. Because the systems are not dependent on gravity flow, they can be installed in areas with level and rolling topography using small diameter PVC lines laid at shallow depth. A significant advantage of vacuum sewers over low pressure systems is that no onsite pumps are required, just the holding tank with a vacuum valve. This significantly reduces the onsite costs for ongoing maintenance. It also eliminates any need for power onsite, so a holding tank is not required in the event of a power outage. As with a conventional system, an emergency generator would be provided at the vacuum station, maintaining full service during an outage.

Disadvantages of the vacuum sewer system include high cost, clogging problems and potential system malfunction should any a vacuum valve remain lodged in the open position. Also, the vacuum in the system is limited in terms of suction head to about 15 feet. As with a lift station, the vacuum station would have to be sited in the system.

EVALUATION OF COLLECTION SYSTEM ALTERNATIVES

As discussed in Chapter 6, providing wastewater collection service to two different service areas was evaluated. Service Area 1 straddles the Finch Creek corridor, taking in residences, businesses and institutional facilities from Highway 101 to the west end of Finch Creek Road. Service Area 2 includes approximately 3,000 feet of Hood Canal shoreline and the business district, in addition to all of Service Area 1. Several collection system alternatives were considered to provide wastewater collection to the two service areas. Gravity, grinder pump, STEP pump, STEG, and vacuum systems were all considered, as well as combinations of gravity and grinder pump sewers and of STEP pump and STEG sewers.

Property maps, survey data and aerial photography were used to establish the length of pipe for laterals, main lines and force mains. The main collector lines and transmission

mains would be located along roads in existing easements. As stated previously, factors within the service area such as shallow ground water, construction in pavement, existing utilities, etc. can impact construction costs significantly, and may drive the selection of the type of sewer constructed. Also, as with Service Area 2, the ultimate destination of the raw wastewater can influence the type of collection system recommended due to transmission costs.

If lift stations are included in the collection system, Ecology criteria require a standby pump and auxiliary power to be used in case of pump malfunction or power outages. An alarm system is required in order to notify operations personnel of a malfunction or lift station failure. Existing drainfields and septic tanks would not be used if a new wastewater collection system were constructed. The Mason County Health Department requires that septic tanks no longer in service have the solids pumped out and the tanks removed, filled in, or broken up. If a public wastewater collection system is built in the study area, the proper pumping and abandonment of existing septic tanks will be required.

Operation and maintenance (O&M) costs for STEP collection systems are based on tank pumping every five years and pump replacement every ten years. Grinder pump system O&M costs are based on pump replacement every fifteen years. Labor costs are based on a rate of $30 per hour, including salary and benefits.

SERVICE AREA 1

For Service Area 1, it is proposed that wastewater be collected and transported to the Thurman property (Treatment Site 1) for treatment and disposal. For this area, five collection system options were evaluated: gravity, grinder pump, and STEP pump systems, as well as a combination gravity/grinder pump system and a combination STEP/STEG system. The proposed layout of a collection system for Service Area 1, with transmission to Treatment Site 1, is presented in Figure 8-3.

Gravity Collection System

A gravity collection system for Service Area 1 would flow down Finch Creek Road to a submersible pump station located in the vicinity of Highway 101. From the pump station, the wastewater would be pumped through a force main back up to a second pump station located in the upper Finch Creek Road area. Ideally, the second pump station could be located near the entrance to the logging road leading to Treatment Site 1. From the second pump station the wastewater would be pumped through a force main to the treatment site.

The gravity sewer line and the return force main would be bored under Finch Creek at the upper bridge crossing. A second creek crossing would be required at the Highway 101

bridge, however, at that location the pipes could be hung on the bridge. Both of the pump stations would be constructed below grade and served by a portable emergency generator. The construction cost estimate for a gravity collection system for Service Area 1, as well as transmission costs to the drainfield site, are presented in Table 8-1. Operation and maintenance costs for the collection and transmission system are also included in the table.

Table 8-1

Construction Cost Estimate for Gravity Collection System with Transmission to Drainfield Site (Treatment Site 1)

Item	Gravity Collection System	Transmission to Treatment Site 1
Mobilization/demobilization	$ 53,420	$ 30,550
Gravity sewer	$438,800	
Force main		$141,000
Grinder pumps	$ 30,000	
Service connections	$ 6,000	
Traffic control	$ 6,000	
Road restoration	$ 35,400	$ 4,500
Pump station(s)		$160,000
Abandon existing septic tanks	$ 18,000	
Subtotal	$587,620	$336,050
Sales tax (7.9%)	$ 46,420	$ 26,550
Subtotal	$634,040	$362,600
Contingency (20%)	$126,810	$ 72,520
Estimated Construction Costs	$760,850	$435,120
Engineering, Legal, Administration (20%)	$152,170	$ 87,020
Total Estimated Capital Costs	$913,020	$522,140
Annual Operation & Maintenance Costs	$ 2,400	$ 6,680

Grinder Pump Collection System

With the grinder pump system, each home or business would have a grinder unit pumping into a 2 or 3-inch force main. The individual grinder pumps do not have sufficient head to pump directly to Treatment Site 1. Therefore, the wastewater would be pumped to a submersible pump station located in the upper Finch Creek area, ideally, near the logging road leading to the treatment site. The pump station would pump the wastewater to the treatment site through a 4" force main laid along the logging road. The pump station would be constructed below grade and served by a portable emergency generator.
The construction cost estimates for a grinder pump collection system for Service Area 1, as well as transmission costs to the drainfield site, are presented in Table 8-2. Operation

and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-2

Construction Cost Estimate for Grinder Pump Collection System with Transmission to Drainfield Site (Treatment Site 1)

Item	Grinder Pump Collection System	Transmission to Treatment Site 1
Mobilization/demobilization	$ 40,100	$ 12,950
Grinder pumps	$180,000	
Service connections	$ 36,000	
Force main	$128,000	$ 45,000
Traffic control	$ 3,600	
Road restoration	$ 35,400	$ 4,500
Pump station		$ 80,000
Abandon existing septic tanks	$ 18,000	
Subtotal	$441,100	$142,450
Sales tax (7.9%)	$ 34,850	$ 11,250
Subtotal	$475,950	$153,700
Contingency (20%)	$ 95,190	$ 30,740
Estimated Construction Costs	$571,140	$184,440
Engineering, Legal, Administration (20%)	$114,230	$ 36,890
Total Estimated Capital Costs	$685,370	$221,330
Annual Operation & Maintenance Costs	$ 7,560	$ 6,680

Combined Gravity/Grinder Pump Collection System

A combined gravity/grinder pump system was also evaluated for Service Area 1. This system would utilize gravity sewer in the upper Finch Creek Road area. The wastewater would flow down to a pump station located near the logging road leading to Treatment Site 1. The remainder of Service Area 1 that lies lower than the proposed pump station would be served by grinder pumps. Each of the homes or businesses in the lower area would have a grinder unit pumping into a 2 or 3-inch force main leading up to the pump station. The pump station would pump the wastewater to the treatment site through a 4-inch force main laid along the logging road. The submersible pump station would be constructed below grade and served by a portable emergency generator.

The construction cost estimates for a combined gravity/grinder pump collection system for Service Area 1, as well as transmission costs to the drainfield site, are presented in

Table 8-3. Operation and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-3

Construction Cost Estimate for Combined Gravity/Grinder Pump Collection System with Transmission to Drainfield Site (Treatment Site 1)

Item	Gravity/Grinder Collection System	Transmission to Treatment Site 1
Mobilization/demobilization	$ 39,110	$ 12,950
Gravity sewer	$ 93,000	
Grinder pumps	$120,000	
Service connections	$ 24,000	
Force main	$ 98,000	$ 45,000
Traffic control	$ 3,600	
Road restoration	$ 34,500	$ 4,500
Pump station		$ 80,000
Abandon existing septic tanks	$ 18,000	
Subtotal	$430,210	$142,450
Sales tax (7.9%)	$ 33,990	$ 11,250
Subtotal	$464,200	$153,700
Contingency (20%)	$ 92,840	$ 30,740
Estimated Construction Costs	$557,040	$184,440
Engineering, Legal, Administration (20%)	$111,410	$ 36,890
Total Estimated Capital Costs	$668,450	$221,330
Annual Operation & Maintenance Costs	$ 6,040	$ 6,680

STEP Pump Collection System

A STEP pump collection system for Service Area 1 would involve siting a STEP tank at each home or business. The STEP pump would discharge the clarified effluent into a 2 or 3-inch force main leading to the treatment site. Because the STEP pumps have sufficient head capacity to pump the effluent to the treatment site, an additional pump station near the logging road is not required. Also, because the heavier solids have already been settled in the STEP tank, a settling tank at the treatment site is also not required.

The construction cost estimate for a STEP pump collection system for Service Area 1, as well as transmission costs to the drainfield site, are presented in Table 8-4. Operation and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-4

Construction Cost Estimate for STEP Collection System with Transmission to Drainfield Site (Treatment Site 1)

Item	STEP Collection System	Transmission to Treatment Site 1
Mobilization/demobilization	$ 39,380	$ 4,950
Septic tanks/STEP equipment	$172,800	
Service connections	$ 36,000	
Force main	$128,000	$45,000
Traffic control	$ 3,600	
Road restoration	$ 35,400	$ 4,500
Abandon existing septic tanks	$ 18,000	
Subtotal	$433,180	$54,450
Sales tax (7.9%)	$ 34,220	$ 4,300
Subtotal	$467,400	$58,750
Contingency (20%)	$ 93,480	$11,750
Estimated Construction Costs	$560,880	$70,500
Engineering, Legal, Administration (20%)	$112,180	$14,100
Total Estimated Capital Costs	$673,060	$84,600
Annual Operation & Maintenance Costs	$ 9,000	$ 0

COMBINED STEP/STEG COLLECTION

A combination STEP pump/STEG collection system for Service Area 1 would involve siting a STEP tank or a STEG tank at each home or business. With this system, the upper Finch Creek Road area would be served by STEG sewer. The STEG sewer would flow down to a pump station located near the logging road that leads to the treatment site. The area below the pump station would be served by STEP pump sewer, with a STEP tank at each home or business. The STEP pump would discharge the clarified effluent into a 2 or 3-inch force main leading to the submersible pump station. The pump station would pump the wastewater to the treatment site through a 4-inch force main laid along the logging road. Because the heavier solids have already been settled in the STEP or STEG tanks, a settling tank at the treatment site would not be required.

The pump station would be constructed below grade and served by a portable emergency generator. Because the clarified effluent is septic and corrosive in nature, the pump station equipment would have to be corrosion resistant, and odor control would need to be provided. The construction cost estimates for a combined STEP pump and STEG collection system for Service Area 1, as well as transmission costs to the drainfield site, are presented in Table 8-5. Operation and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-5

Construction Cost Estimate for Combined STEP/STEG Collection System with Transmission to Drainfield Site (Treatment Site 1)

Item	STEP/STEG Collection System	Transmission to Treatment Site 1
Mobilization/demobilization	$ 33,820	$ 14,950
Septic tanks	$ 72,000	
STEP equipment	$ 67,200	
STEG equipment	$ 6,000	
Service connections	$ 18,000	
Force main	$118,000	$ 45,000
Traffic control	$ 3,600	
Road restoration	$ 35,400	$ 4,500
Odor Control		$ 20,000
Pump station		$ 80,000
Abandon existing septic tanks	$ 18,000	
Subtotal	$372,020	$164,450
Sales tax (7.9%)	$ 29,390	$ 12,990
Subtotal	$401,410	$177,440
Contingency (20%)	$ 80,280	$ 35,490
Estimated Construction Costs	$481,690	$212,930
Engineering, Legal, Administration (20%)	$ 96,340	$ 42,590
Total Estimated Capital Costs	$578,030	$255,520
Annual Operation & Maintenance Costs	$ 7,500	$ 6,680

SERVICE AREA 2

For Service Area 2, it is proposed that wastewater be collected and transported to either of two sites: a water reclamation facility in Hoodsport (Treatment Site 2) or an upland SBR/sprayfield facility (Treatment Site 3). Gravity, grinder pump, and STEP pump systems, as well as a combination gravity/grinder pump system, were evaluated for collection and transport to each of the two treatment sites. Also, a vacuum system was evaluated for collection and transport to Treatment Site 2, and a combination STEP/STEG system was evaluated for collection and transport to Treatment Site 3.

Wastewater Collection and Transmission to Treatment Site 2

The proposed layout of a collection system for Service Area 2 that would transport wastewater to Treatment Site 2 is presented in Figure 8-4. As discussed in Chapter 7, the proposed treatment site is located behind the Hoodsport Inn Restaurant. Since the

treatment site is located within the area where the wastewater is being collected, transmission costs will be significantly less than for the upland site, Treatment Site 3.

Gravity Collection System

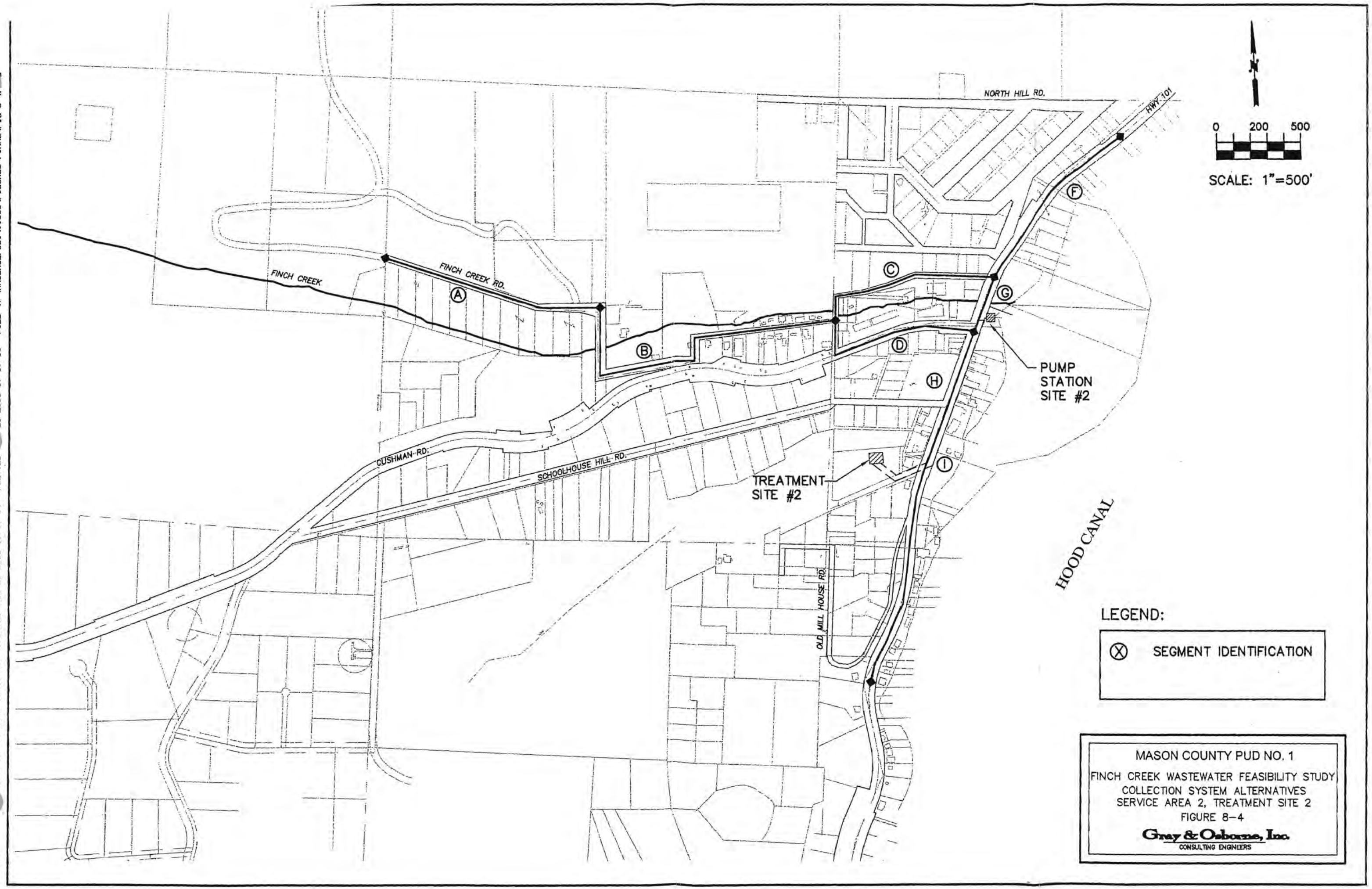
With a gravity collection system and transmission to Treatment Site 2, wastewater would flow from the Finch Creek area and from the north and south along Highway 101 to a submersible pump station located in the vicinity of the WDFW hatchery. The gravity system layout would require a bored crossing under Finch Creek at the upper bridge. There would also be an additional crossing under the Hwy 101 bridge.

From the pump station, the wastewater would be pumped through a 4-inch force main to the treatment site. Pumping the wastewater to Treatment Site 2 would require a relatively short force main of approximately 1,500 feet, with about 30 feet of elevation gain. The pump station would be constructed below grade and be served by a portable emergency generator. The construction cost estimates for a gravity collection system and transmission to the treatment site are presented in Tables 8-6. Operation and maintenance costs for the collection and transmission systems are also included in the tables.

TABLE 8-6

Construction Cost Estimate for Gravity Collection System with Transmission to Water Reuse Facility (Treatment Site 2)

Item	Gravity Collection System	Transmission to Treatment Site 2
Mobilization/demobilization	$ 108,700	$ 12,800
Gravity sewer	$ 837,000	
Traffic control	$ 64,800	
Road restoration	$ 136,200	
Pump station		$ 80,000
Force main		$ 48,000
Abandon existing septic tanks	$ 49,000	
Subtotal	$1,195,700	$140,800
Sales tax (7.9%)	$ 94,460	$ 11,120
Subtotal	$1,290,160	$151,920
Contingency (20%)	$ 258,030	$ 30,380
Estimated Construction Costs	$1,548,190	$182,300
Engineering, Legal, Administration (20%)	$ 309,640	$ 36,460
Total Estimated Capital Costs	$1,857,830	$218,760
Annual Operation & Maintenance Costs	$ 4,800	$ 6,680



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Grinder Pump Collection System

With the grinder pump system, each home or business would have a grinder unit pumping into a 2 or 3-inch force main. For the system to transport wastewater to Treatment Site 2, the water reuse facility, a 2-inch line is sufficient for the whole system, and the grinder pumps are adequate to pump the wastewater to the treatment site. A transmission main or pump station is not required since the site is located within the collection system.
The construction cost estimates for a grinder pump collection system and transmission to Treatment Site 2, the water reuse facility, are presented in Table 8-7. Operation and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-7

Construction Cost Estimate for Grinder Pump Collection System with Transmission to Water Reuse Facility (Treatment Site 2)

Item	Grinder Collection System	Transmission to Treatment Site 2
Mobilization/demobilization	$ 100,220	
Grinder pumps	$ 490,000	
Service connections	$ 98,000	
Force main	$ 202,000	
Traffic control	$ 27,000	
Road restoration	$ 136,200	
Abandon existing septic tanks	$ 49,000	
Subtotal	$1,102,420	$0
Sales tax (7.9%)	$ 87,090	
Subtotal	$1,189,510	$0
Contingency (20%)	$ 237,900	
Estimated Construction Costs	$1,427,410	$0
Engineering, Legal, Administration (20%)	$ 285,480	
Total Estimated Capital Costs	$1,712,890	$0
Annual Operation & Maintenance Costs	$ 20,580	$0

Combined Gravity/Grinder Collection System

The combined gravity/grinder pump system proposed to collect and transport wastewater to Treatment Site 2 would use grinder pumps in the upper Finch Creek Road area and along Highway 101. The grinder pumps would pump across the upper Finch Creek bridge, then discharge to gravity sewer below the bridge. The gravity sewer would provide service along Finch Creek Road down to Highway 101, where it would discharge

to a pump station in the vicinity of the WDFW hatchery. The low pressure mainline sewer along the highway would also discharge to the pump station. The pump station would then pump the wastewater to the treatment site.

The pump station would be constructed below grade and served by a portable emergency generator. The construction cost estimates for a combination gravity/grinder pump collection system and transmission to Treatment Site 2, the water reuse facility, are presented in Table 8-8. Operation and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-8

Construction Cost Estimate for Combined Gravity/Grinder Pump Collection System with Transmission to Water Reuse Facility (Treatment Site 2)

Item	Gravity/Grinder Collection System	Transmission to Treatment Site 2
Mobilization/demobilization	$ 98,360	
Gravity sewer	$ 244,000	
Grinder pumps	$ 370,000	
Service connections	$ 74,000	
Force main	$ 60,000	$ 48,000
Traffic control	$ 60,000	
Road restoration	$ 126,600	
Pump station		$ 80,000
Abandon existing septic tanks	$ 49,000	
Subtotal	$1,081,960	$128,000
Sales tax (7.9%)	$ 85,470	$ 10,110
Subtotal	$1,167,430	$138,110
Contingency (20%)	$ 233,490	$ 27,620
Estimated Construction Costs	$1,400,920	$165,730
Engineering, Legal, Administration (20%)	$ 280,180	$ 33,150
Total Estimated Capital Costs	$1,681,100	$198,880
Annual Operation & Maintenance Costs	$ 17,540	$ 6,680

STEP Pump Collection System

Constructing a STEP pump collection system for Service Area 2 would involve siting a STEP tank at each home or business. A 2-inch collection line is sufficient throughout the system to transport wastewater to Treatment Site 2, the water reuse facility. Also, the STEP pumps are adequate to pump the wastewater to the treatment site. Therefore, a transmission main or pump station is not required, since the site is located within the collection system.

The construction cost estimates for a STEP pump collection system to Treatment Site 2, the water reuse facility, are presented in Table 8-9. Operation and maintenance costs for the collection system are also included in the table.

TABLE 8-9

Construction Cost Estimate for STEP Pump Collection System with Transmission to Water Reuse Facility (Treatment Site 2)

Item	STEP Pump Collection System	Transmission to Treatment Site 2
Mobilization/demobilization	$ 98,260	
Septic tanks/STEP equipment	$ 470,400	
Service connections	$ 98,000	
Force main	$ 202,000	
Traffic control	$ 27,000	
Road restoration	$ 136,200	
Abandon existing septic tanks	$ 49,000	
Subtotal	$1,080,860	$0
Sales tax (7.9%)	$ 85,390	
Subtotal	$1,166,250	$0
Contingency (20%)	$ 233,250	
Estimated Construction Costs	$1,399,500	$0
Engineering, Legal, Administration (20%)	$ 279,900	
Total Estimated Capital Costs	$1,679,400	$0
Annual Operation & Maintenance Costs	$ 24,500	$0

Vacuum Collection System.

Installation of a vacuum collection system in Service Area 2 would involve siting a vacuum unit at each residence or business. A vacuum/pump station would be located at Treatment Site 2. The wastewater would be drawn by vacuum pressure through a 4-inch main to the treatment site, where it would be pumped into the treatment facility. The vacuum/pump station would be served by the same onsite generator as the treatment facility.

The construction cost estimates for a vacuum collection system to Treatment Site 2, the water reuse facility, are presented in Table 8-10. Operation and maintenance costs for the collection system are also included in the table.

TABLE 8-10

Construction Cost Estimates for Vacuum Collection System with Transmission to Water Reuse Facility (Treatment Site 2)

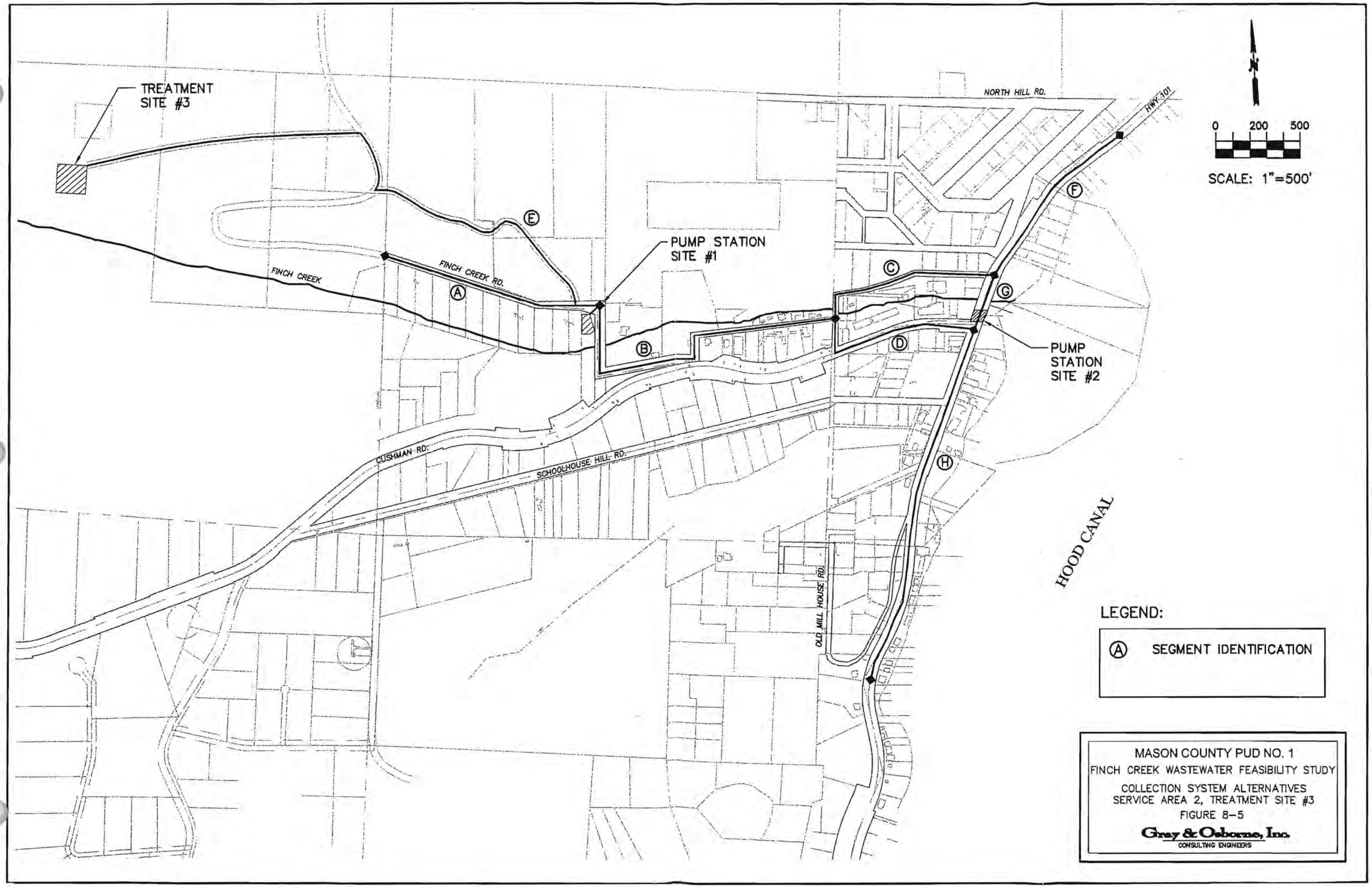
Item	Vacuum Collection System	Transmission to Treatment Site 2
Mobilization/demobilization	$ 102,430	
Vacuum units	$ 245,000	
Service connections	$ 98,000	
Vacuum main	$ 308,050	
Traffic control	$ 27,000	
Road restoration	$ 136,200	
Vacuum/pump station	$ 210,000	
Subtotal	$1,126,680	$0
Sales tax (7.9%)	$ 89,010	
Subtotal	$1,215,690	
Contingency (20%)	$ 243,140	
Estimated Construction Costs	$1,458,830	$0
Engineering, Legal, Administration (20%)	$ 291,770	
Total Estimated Capital Costs	$1,750,600	$0
Annual Operation & Maintenance Costs	$ 29,720	$0

Wastewater Collection and Transmission to Treatment Site 3

Collection of the wastewater for transmission to Treatment Site 3, the upland SBR/sprayfield facility, would be much the same as described for Treatment Site 2. However, transmission to Treatment Site 3 will be significantly more difficult. The site is approximately 300 feet above the area where the wastewater would be collected, so all of the collection system alternatives will require at least one pump station to lift the wastewater to the site. Also, the distance to the site will require that several thousand feet of force main be constructed to transport wastewater to the facility. The proposed collection system layout and transmission line to Treatment Site 3 is shown in Figure 8-5.

Gravity Collection System

Similar to the system previously described for Treatment Site 2, the gravity collection system would be constructed down through the Finch Creek area and from the north and south along Highway 101. The wastewater would drain to a submersible pump station located in the vicinity of the WDFW hatchery. The gravity system layout would require a bored crossing under Finch Creek at the upper bridge and an additional crossing under the Hwy 101 bridge.



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From the pump station, the wastewater would be pumped through a 4-inch force main to the treatment facility. Two additional pump stations would be required to lift the wastewater to the treatment facility, one in the upper Finch Creek Road area, and another at 100 to 120 feet in elevation up the logging road. The pump stations would all be constructed below grade and be served by a portable emergency generator.

The construction cost estimates for a gravity collection system and transmission to Treatment Site 3 are presented in Table 8-11. Operation and maintenance costs for the collection and transmission systems are also presented in the table.

TABLE 8-11

Construction Cost Estimates for Gravity Collection System with Transmission to SBR/Sprayfield Facility (Treatment Site 3)

Item	Gravity Collection System	Transmission to Treatment Site 3
Mobilization/demobilization	$ 103,700	$ 45,480
Gravity sewer	$ 787,000	
Traffic control	$ 64,800	
Road restoration	$ 136,200	$ 10,800
Pump station(s)		$240,000
Force main		$204,000
Abandon existing septic tanks	$ 49,000	
Subtotal	$1,140,700	$500,280
Sales tax (7.9%)	$ 90,120	$ 39,520
Subtotal	$1,230,820	$539,800
Contingency (20%)	$ 246,160	$107,960
Estimated Construction Costs	$1,476,980	$647,760
Engineering, Legal, Administration (20%)	$ 295,400	$129,550
Total Estimated Capital Costs	$1,772,380	$777,310
Annual Operation & Maintenance	$ 4,800	$ 20,040

Grinder Pump Collection System.

With the grinder pump system, each home or business would have a grinder unit pumping into a 2 or 3-inch force main. In order to transport wastewater to Treatment Site 3, the upland SBR/sprayfield facility, the grinder pumps would pump all the wastewater to a submersible pump station located in the upper Finch Creek area. Ideally, the pump station would be located near the logging road leading to the treatment site. The wastewater would be pumped to the treatment site through a 4" force main laid along the logging road. A second pump station would be required at approximately 100 to 120 feet

in elevation up the logging road to lift the wastewater to the treatment facility. The pump stations would be constructed below grade and served by a portable emergency generator. The construction cost estimates for a grinder pump collection system and transmission to Treatment Site 3, the upland SBR/sprayfield facility, are presented in Table 8-12. Operation and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-12

Construction Cost Estimates for Grinder Pump Collection System with Transmission to SBR/Sprayfield Facility (Treatment Site 3)

Item	Grinder Collection System	Transmission to Treatment Site 3
Mobilization/demobilization	$ 103,040	$ 27,880
Grinder pumps	$ 490,000	
Service connections	$ 98,000	
Force main	$ 202,000	$108,000
Traffic control	$ 64,800	
Road restoration	$ 126,600	$ 10,800
Pump station(s)		$160,000
Abandon existing septic tanks	$ 49,000	
Subtotal	$1,133,440	$306,680
Sales tax (7.9%)	$ 89,540	$ 24,230
Subtotal	$1,222,980	$330,910
Contingency (20%)	$ 244,600	$ 66,180
Estimated Construction Costs	$1,467,580	$397,090
Engineering, Legal, Administration (20%)	$ 293,520	$ 79,420
Total Estimated Capital Costs	$1,761,100	$476,510
Annual Operation & Maintenance Costs	$ 20,580	$ 13,360

Combination Gravity/Grinder Pump Collection System

The combination gravity/ grinder pump collection system to collect and transport wastewater to Treatment Site 3, the upland SBR/sprayfield facility, would be similar to the previous grinder pump system. The main difference is that the upper Finch Creek area would be served by gravity sewer. The remainder of the service area would be served by grinder pumps. The grinder pumps would discharge into a 2 or 3-inch force main line leading to a submersible pump station located in the upper Finch Creek area, ideally located near the logging road leading to the treatment site. The pump station would pump the wastewater to the treatment site through a 4" force main laid along the logging road. A second pump station would be required approximately 100 to 120 feet in

elevation up the logging road. The pump stations would be constructed below grade and served by a portable emergency generator.

The construction cost estimates for a combination gravity/grinder pump collection system and transmission to Treatment Site 3, the upland SBR/sprayfield facility, are presented in Table 8-13. Operation and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-13

Construction Cost Estimates for Combined Gravity/Grinder Pump Collection System with Transmission to SBR/Sprayfield Facility (Treatment Site 3)

Item	Gravity/Grinder Collection System	Transmission to Treatment Site 3
Mobilization/demobilization	$ 102,140	$ 27,880
Gravity sewer	$ 93,000	
Grinder pumps	$ 430,000	
Service connections	$ 86,000	
Force main	$ 172,000	$108,000
Traffic control	$ 64,800	
Road restoration	$ 126,600	$ 10,800
Pump station(s)		$160,000
Abandon existing septic tanks	$ 49,000	
Subtotal	$1,123,540	$306,680
Sales tax (7.9%)	$ 88,760	$ 24,230
Subtotal	$1,212,300	$330,910
Contingency (20%)	$ 242,460	$ 66,180
Estimated Construction Costs	$1,454,760	$397,090
Engineering, Legal, Administration (20%)	$ 290,950	$ 79,420
Total Estimated Capital Costs	$1,745,710	$476,510
Annual Operation & Maintenance Costs	$ 20,060	$ 13,360

STEP Pump Collection System

A STEP pump collection system to collect and transport wastewater to Treatment Site 3, the upland SBR/sprayfield facility, would have 2, 3 and 4-inch force main leading to the treatment site. The STEP pumps do not have sufficient head capacity to pump the clarified effluent to the treatment site elevation. Therefore, an additional pump station approximately 200 to 220 feet in elevation up the logging road would be required. The pump station would pump the wastewater to the treatment site through a 4" force main

laid along the logging road. The pump station would be constructed below grade and served by a portable emergency generator.

The construction cost estimates for a STEP pump collection system and transmission to Treatment Site 3, the upland SBR/sprayfield facility, are presented in Table 8-14. Operation and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-14

Construction Cost Estimates for STEP Pump Collection System with Transmission to SBR/Sprayfield Facility (Treatment Site 3)

Item	STEP Pump Collection System	Transmission to Treatment Site 3
Mobilization/demobilization	$ 101,080	$ 19,880
Septic tanks/STEP equipment	$ 470,400	
Service connections	$ 98,000	
Force main	$ 202,000	$108,000
Traffic control	$ 64,800	
Road restoration	$ 126,600	$ 10,800
Pump station		$ 80,000
Abandon existing septic tanks	$ 49,000	
Subtotal	$1,111,880	$218,680
Sales tax (7.9%)	$ 87,840	$ 17,280
Subtotal	$1,199,720	$235,960
Contingency (20%)	$ 239,940	$ 47,190
Estimated Construction Costs	$1,439,660	$283,150
Engineering, Legal, Administration (20%)	$ 287,930	$ 56,630
Total Estimated Capital Costs	$1,727,590	$339,780
Annual Operation & Maintenance Costs	$ 24,500	$ 6,680

Combination STEP/STEG Collection System

The combined STEP pump/STEG collection system would be very similar to the STEP system previously discussed. However, the upper Finch Creek area would be served by STEG sewer. Since this area would be served by gravity, a pump station would be required near the entrance to the logging road leading to the treatment facility. The wastewater entering this pump station would be septic, therefore, odor control would be provided. The wastewater would be pumped through a 4-inch force main to the treatment site. A second pump station would be required at about 100 to 120 feet in elevation up the logging road to lift the wastewater to the treatment facility. The pump stations would be constructed below grade and served by a portable emergency generator.

The construction cost estimates for the combined STEP pump/STEG collection system and transmission to Treatment Site 3, the upland SBR/sprayfield facility, are presented in Table 8-15. Operation and maintenance costs for the collection and transmission system are also included in the table.

TABLE 8-15

Construction Cost Estimates for STEP/STEG Collection System with Transmission to SBR/Sprayfield Facility (Treatment Site 3)

Item	STEP/STEG Collection System	Transmission to Treatment Site 3
Mobilization/demobilization	$ 98,320	$ 29,880
Septic tanks	$ 196,000	
STEP/STEG Equipment	$ 246,800	
Service connections	$ 98,000	
Force main	$ 202,000	$108,000
Traffic control	$ 64,800	
Road restoration	$ 126,600	$ 10,800
Odor Control		$ 20,000
Pump stations		$160,000
Abandon existing septic tanks	$ 49,000	
Subtotal	$1,081,520	$328,680
Sales tax (7.9%)	$ 85,440	$ 25,970
Subtotal	$1,166,960	$354,650
Contingency (20%)	$ 233,390	$ 70,930
Estimated Construction Costs	$1,400,350	$425,580
Engineering, Legal, Administration (20%)	$ 280,070	$ 85,120
Total Estimated Capital Costs	$1,680,420	$510,700
Annual Operation & Maintenance Costs	$ 23,000	$ 13,360

Collection system costs for both Service Area 1 and Service Area 2 are presented in Chapter 9, along with treatment costs for each of the treatment alternatives. The resulting total project costs are evaluated, as well as non-cost factors for each of the alternatives.

CHAPTER 9

CHAPTER 9

PLAN SUMMARY AND RECOMMENDATIONS

This chapter summarizes the previous chapters and provides comparisons for both capital and O&M costs for alternative wastewater collection, transmission, and treatment for the respective service areas. Based on these comparisons, a recommended approach is developed.

PLAN SUMMARY

In Chapter 6, two possible service areas were identified, Service Area 1 and Service Area 2, and flows and loadings were developed for each area. Chapters 7 and 8 presented alternatives for providing wastewater collection and treatment to the two service areas. Construction cost estimates and operation and maintenance (O&M) costs for each alternative were also presented. This section compares the possible alternatives based on capital, O&M, and net present worth costs, and the least costly alternatives for providing wastewater collection and treatment are identified.

SERVICE AREA 1

Service Area 1 includes the Finch Creek corridor from the west end down to Highway 101. Five collection system alternatives were evaluated to provide wastewater collection to this area: gravity, grinder pump, combination gravity/grinder pump, STEP pump, and combination STEP pump/STEG sewer. Two treatment approaches were also evaluated, both discharging to a pressurized drainfield. The first approach, Alternative 1, would provide preliminary settling of solids and rely on the drainfield to provide much of the wastewater treatment. Alternative 2 would provide treatment in a recirculating gravel filter (RGF) and rely on the drainfield primarily for disposal. A summary of the project costs for each of the alternatives for Service Area 1 is provided in Table 9-1.

The STEP collection system is the least costly collection alternative for Service Area 1. The lower cost is due primarily to the fact that the STEP pumps are capable of pumping to the treatment site without the need for an additional pump station. This also results in lower O&M costs for the collection and transmission system.

Alternative 1 is the least costly treatment approach for Service Area 1. Treatment Alternative 2 would provide much better treatment of the wastewater prior to discharge to the drainfield than would Alternative 1, which provides only settling. This additional treatment would reduce the potential of drainfield problems or failure. However, the DOH design criteria do not allow any credit, i.e. downsizing of the drainfield, for providing the higher level of treatment. Therefore, Alternative 2 is significantly more costly than Alternative 1. The annual O&M costs are also higher for Alternative 2, due to its higher level of complexity and need for more maintenance and oversight.

TABLE 9-1

Summary of Project Costs for Service Area 1

Alternatives	Collection System	Transmission	Process Treatment	Total Capital Cost	Annual O&M Cost	Present Worth		
						5 years	10 years	20 years
Gravity Collection Septic Tank w/DF	$913,020	$522,140	$603,280	$2,038,440	$18,640	$2,121,420	$2,189,630	$2,291,763
Grinder Collection Septic Tank w/DF	$685,370	$221,330	$603,280	$1,509,980	$23,800	$1,615,930	$1,703,020	$1,833,430
Gravity/Grinder Collection Septic Tank w/DF	$668,450	$221,330	$603,280	$1,493,060	$22,280	$1,592,250	$1,673,770	$1,795,850
STEP Pump Collection Septic Tank w/DF	$673,060	$84,600	$603,280	$1,360,940	$18,560	$1,443,570	$1,511,480	$1,613,180
STEP/STEG Collection Septic Tank w/DF	$578,030	$255,520	$603,280	$1,436,830	$23,740	$1,542,520	$1,629,380	$1,759,460
Gravity Collection Recirculating Gravel Filter w/DF	$913,020	$522,140	$914,680	$2,349,840	$22,870	$2,451,650	$2,535,340	$2,660,650
Grinder Collection Recirculating Gravel Filter w/DF	$685,370	$221,330	$914,680	$1,821,380	$28,030	$1,946,160	$2,048,730	$2,202,320
Gravity/Grinder Collection Recirculating Gravel Filter w/DF	$668,450	$221,330	$914,680	$1,804,460	$26,510	$1,922,480	$2,019,480	$2,164,740
STEP Collection Recirculating Gravel Filter w/DF	$673,060	$84,600	$914,680	$1,672,340	$22,790	$1,773,800	$1,857,190	$1,982,060
STEP/STEG Collection Recirculating Gravel Filter w/DF	$578,030	$255,520	$914,680	$1,748,230	$27,970	$1,872,750	$1,975,090	$2,128,350

CHAPTER 9

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SERVICE AREA 1

Service Area 1 includes the Finch Creek corridor from the west end down to Highway 101. Five collection system alternatives were evaluated to provide wastewater collection to this area: gravity, grinder pump, combination gravity/grinder pump, STEP pump, and combination STEP pump/STEG sewer. Two treatment approaches were also evaluated, both discharging to a pressurized drainfield. The first approach, Alternative 1, would provide preliminary settling of solids and rely on the drainfield to provide much of the wastewater treatment. Alternative 2 would provide treatment in a recirculating gravel filter (RGF) and rely on the drainfield primarily for disposal. A summary of the project costs for each of the alternatives for Service Area 1 is provided in Table 9-1.

The STEP collection system is the least costly collection alternative for Service Area 1. The lower cost is due primarily to the fact that the STEP pumps are capable of pumping to the treatment site without the need for an additional pump station. This also results in lower O&M costs for the collection and transmission system.

Alternative 1 is the least costly treatment approach for Service Area 1. Treatment Alternative 2 would provide much better treatment of the wastewater prior to discharge to the drainfield than would Alternative 1, which provides only settling. This additional treatment would reduce the potential of drainfield problems or failure. However, the DOH design criteria do not allow any credit, i.e. downsizing of the drainfield, for providing the higher level of treatment. Therefore, Alternative 2 is significantly more costly than Alternative 1. The annual O&M costs are also higher for Alternative 2, due to its higher level of complexity and need for more maintenance and oversight.

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Grinder Collection Septic Tank w/DF	$685,370	$221,330	$603,280	$1,509,980	$23,800	$1,615,930	$1,703,020	$1,833,430
Gravity/Grinder Collection Septic Tank w/DF	$668,450	$221,330	$603,280	$1,493,060	$22,280	$1,592,250	$1,673,770	$1,795,850
STEP Pump Collection Septic Tank w/DF	$673,060	$84,600	$603,280	$1,360,940	$18,560	$1,443,570	$1,511,480	$1,613,180
STEP/STEG Collection Septic Tank w/DF	$578,030	$255,520	$603,280	$1,436,830	$23,740	$1,542,520	$1,629,380	$1,759,460
Gravity Collection Recirculating Gravel Filter w/DF	$913,020	$522,140	$914,680	$2,349,840	$22,870	$2,451,650	$2,535,340	$2,660,650
Grinder Collection Recirculating Gravel Filter w/DF	$685,370	$221,330	$914,680	$1,821,380	$28,030	$1,946,160	$2,048,730	$2,202,320
Gravity/Grinder Collection Recirculating Gravel Filter w/DF	$668,450	$221,330	$914,680	$1,804,460	$26,510	$1,922,480	$2,019,480	$2,164,740
STEP Collection Recirculating Gravel Filter w/DF	$673,060	$84,600	$914,680	$1,672,340	$22,790	$1,773,800	$1,857,190	$1,982,060
STEP/STEG Collection Recirculating Gravel Filter w/DF	$578,030	$255,520	$914,680	$1,748,230	$27,970	$1,872,750	$1,975,090	$2,128,350

SERVICE AREA 2

Service Area 2 includes the business district and much of the shoreline area, as well as Service Area 1, the Finch Creek corridor. Two treatment approaches were evaluated, a water reclamation facility sited within the community of Hoodsport and an upland treatment and land application facility. Five collection system alternatives were evaluated to provide wastewater collection and transmission to each of the two treatment sites. For collection and transmission to the water reclamation facility, gravity, grinder pump, combination gravity/grinder pump, STEP pump, and vacuum sewer were considered. For collection and transmission to the upland treatment facility, gravity, grinder pump, combination gravity/grinder pump, STEP pump, and combination STEP pump/STEG sewer were considered.

The water reclamation facility, Treatment Alternative 3, would utilize activated sludge treatment with clarification and filtration provided by membrane units. The reclaimed water would be reused at the WDFW fish hatchery. The other treatment approach, Treatment Alternative 4, would utilize a sequencing batch reactor (SBR) process followed by application of the effluent to a sprayfield at an upland location. A summary of the project costs for each of the alternatives for Service Area 2 is provided in Table 9-2.

Treatment Alternative 3 is the least costly treatment approach for Service Area 2. The water reclamation facility would provide a higher level of treatment than the upland SBR facility. However, the upland facility is more costly to construct and operate, primarily because the end use of the water, land application via sprayfield, is more expensive than the end use of the reclaimed water, i.e. reuse at the hatchery. The annual O&M costs are also less for Alternative 3, the water reclamation facility. Costs for operation and maintenance of the two facilities are similar. However, the need for ground water monitoring at the upland sprayfield site, as well as maintenance of the sprayfield itself, causes O&M costs for Alternative 4 to be higher than Alternative 3.

For collection and transmission to the upland treatment site, the STEP collection system is least costly. This is due mainly to the ability of the STEP pumps to lift the wastewater to a higher elevation, resulting in the need for fewer additional pump stations. Transmissions costs to the water reclamation facility are much less than to the upland treatment site since the reclamation facility is located within the area where the wastewater is being produced. For collection and transmission to the water reclamation facility, the STEP pump and grinder pump systems have similar costs and are less costly than the other alternatives. Construction costs for the STEP pump collection system are less than the grinder pump system; however, the grinder pump system has lower annual O&M costs. The present worth analysis indicates that, based on 20-year costs, the two are too close to differentiate.

Table 9-2
Summary of Project Costs
for Service Area 2

Alternatives	Collection System	Transmission	Process Treatment	Total Capital Cost	Annual O&M Cost	Present Worth		
						5 years	10 years	20 years
Gravity Collection Water Reclamation Facility	$1,857,830	$218,760	$1,615,140	$3,691,730	$77,340	$4,036,030	$4,319,030	$4,742,800
Grinder Collection Water Reclamation Facility	$1,712,890	$0	$1,615,140	$3,328,030	$86,440	$3,712,840	$4,029,140	$4,502,775
Gravity/Grinder Collection Water Reclamation Facility	$1,681,100	$198,880	$1,615,140	$3,495,120	$90,080	$3,896,140	$4,225,750	$4,719,330
STEP Pump Collection Water Reclamation Facility	$1,679,400	$0	$1,615,140	$3,294,540	$90,360	$3,696,800	$4,027,440	$4,522,560
Vacuum Collection Water Reclamation Facility	$1,750,600	$0	$1,615,140	$3,365,740	$95,580	$3,791,240	$4,140,980	$4,664,700
Gravity Collection Sequencing Batch Reactor w/Sprayfield	$1,772,380	$777,310	$2,381,680	$4,931,370	$122,280	$5,475,740	$5,923,170	$6,593,190
Grinder Collection Sequencing Batch Reactor w/Sprayfield	$1,761,100	$476,510	$2,381,680	$4,619,290	$131,380	$5,204,170	$5,684,900	$6,404,780
Gravity/Grinder Collection Sequencing Batch Reactor w/Sprayfield	$1,745,710	$476,510	$2,381,680	$4,603,900	$130,860	$5,186,460	$5,665,290	$6,382,330
STEP Collection Sequencing Batch Reactor w/Sprayfield	$1,727,590	$339,780	$2,381,680	$4,449,050	$128,620	$5,021,640	$5,492,270	$6,197,030
STEP/STEG Collection Sequencing Batch Reactor w/Sprayfield	$1,680,420	$510,700	$2,381,680	$4,572,800	$133,800	$5,168,450	$5,658,040	$6,391,180

PLAN RECOMMENDATIONS

The least cost alternative for providing wastewater collection to Service Area 1 is a STEP collection system, with a settling tank and pressurized drainfield located at the Thurman property. The estimated capital cost for this alternative is $1,360,940; however, this cost does not include land costs, which have not yet been determined. Annual O&M costs for the collection and treatment systems are estimated to be $18,560 per year. Based on 40 equivalent residential units (ERU) in Service Area 1, the capital cost per ERU is $34,020, and the O&M costs are $38.70 per ERU per month. A site layout for this alternative is presented in Figure 9-1.

The capital costs per ERU are very high, and are clearly not affordable unless a significant amount of funding assistance is secured. There may also be other means of reducing the capital costs. Some of the construction could possibly be completed through self-help, i.e. volunteer construction. One eastern Washington community was able to secure free prison inmate labor for construction of its collection and treatment system. If it is feasible to reuse some of the existing septic tanks, that could also reduce construction costs.

The least cost alternative for providing wastewater collection and treatment to Service Area 2 is a water reclamation facility with either a STEP or grinder pump collection system. A site layout of the water reclamation facility is presented in Figure 9-2, and an hydraulic profile is presented in Figure 9-3. The grinder pump collection system would be recommended over the STEP pump system for several reasons. The Madrona Beach collection system, which is owned and operated by Mason County PUD No. 1 (PUD No. 1), has grinder pumps in the collection system. Currently, the O&M of that system is contracted to a private contractor. Also, much of the nearby North Bay collection system, which is currently under construction by Mason County, is a grinder pump system. Installation of a grinder pump system would be consistent with these other systems and make it more feasible for PUD No.1 staff to seek technical support or for PUD No. 1 to contract O&M to the County or a private contractor. Grinder pumps would require less area for installation than STEP units on small lots and would eliminate the need for periodic pumping of the STEP tanks. At the treatment facility, the septic STEP influent would result in increased potential for odor and corrosion problems. Therefore, with the costs for STEP and grinder pump collection very comparable, the grinder pump system is recommended for Service Area 2.

The estimated capital cost for this alternative is $3,328,030. This does not include land costs, which are not yet determined. Annual O&M costs for the collection and treatment systems are estimated to be $86,440 per year. Based on 128 equivalent residential units (ERU) in Service Area 2, the capital cost per ERU is $26,000, and the O&M costs are $56.30 per ERU per month.

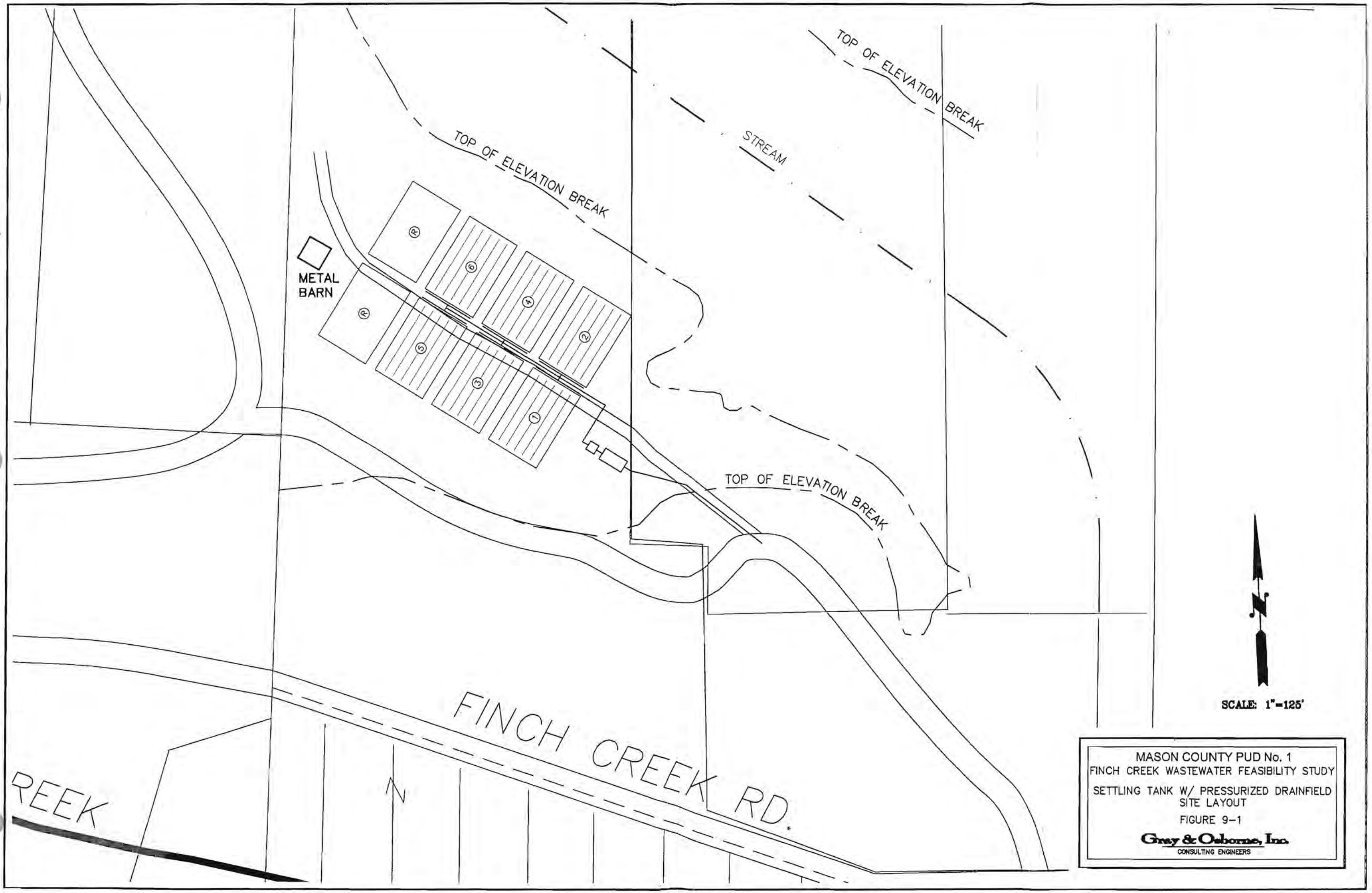
The capital costs per ERU are high, and are not affordable unless a significant amount of funding assistance is secured. As previously discussed, there may be means of reducing the capital costs, such as with self-help or prison inmate labor. The O&M costs are particularly high, due to the limited number of connections to support the ongoing costs of the facility.

RECOMMENDED APPROACH

The Mason County PUD No. 1 has several possible courses of action based on the findings of this feasibility study. The PUD No. 1 can pursue funding for a project to serve Service Area 1, pursue funding for a project to serve Service Area 2, or choose to not pursue either of the two projects, at least at this time. Both of the proposed project alternatives are expensive, particularly for the number of connections that would be served. During the development of the feasibility study, members of the community were generally supportive of the possibility of a community wastewater system for the Finch Creek area and for the business district and shoreline area. However, it is unclear how supportive homeowners and businesses would be when faced with the costs of these projects.

The Service Area 1 project would provide a solution to ongoing septic system problems along Finch Creek, which resulted in the development of this study. If a large portion of the project can be funded by grants, the O&M and debt service costs might be affordable to the homeowners and business owners served by the system. Construction of the Service Area 1 project would result in improved water quality in Finch Creek; however, it would probably have limited impact on improving conditions along the Hood Canal shoreline and in allowing shellfish harvest areas to be reopened. The project would be beneficial to businesses along Finch Creek, but would not provide help for the other businesses in the community. The type of treatment proposed for Service Area 1 is not readily expandable to serve additional flows, if future expansion were to be desired. However, a second and even third drainfield system could be constructed elsewhere to serve the shoreline and business district.

The Service Area 2 project would provide a solution to septic system problems, not just along Finch Creek, but also along much of the Hoodsport shoreline and the business district. The project could feasibly result in much of the Hoodsport shoreline being opened to recreational shellfish harvesting. The project would provide a means for local business owners to maintain and even expand their businesses. These issues could make the project attractive to funding agencies such as Ecology and the Department of Community Trade and Economic Development. The treatment facility is amenable to expansion to serve additional flows, if this were desired in the future. However, even with a large amount of the project funded by grants, the monthly O&M costs to support the system would be quite high. If these cost cannon be reduced, the project would be difficult for the community to afford.



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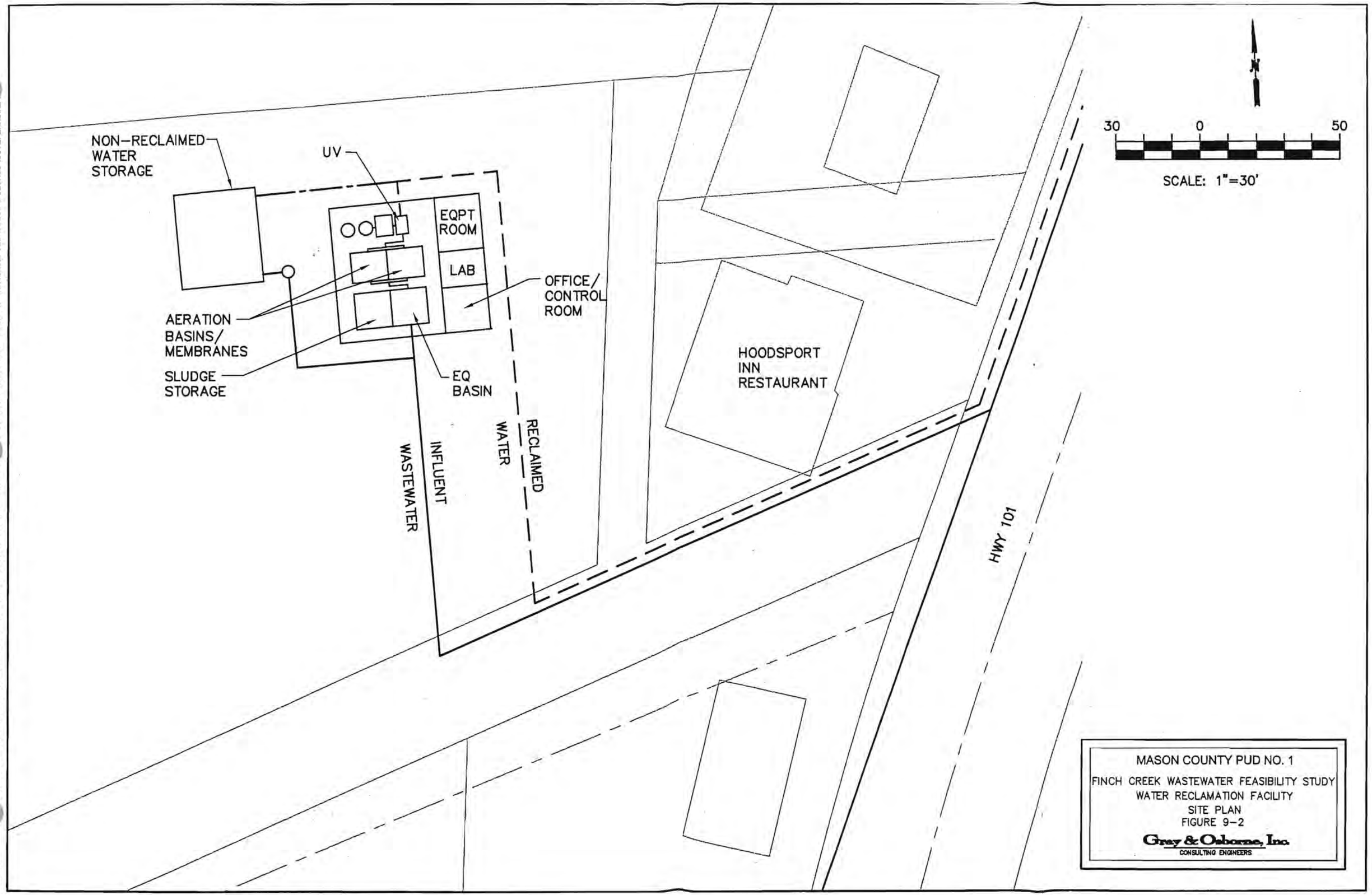

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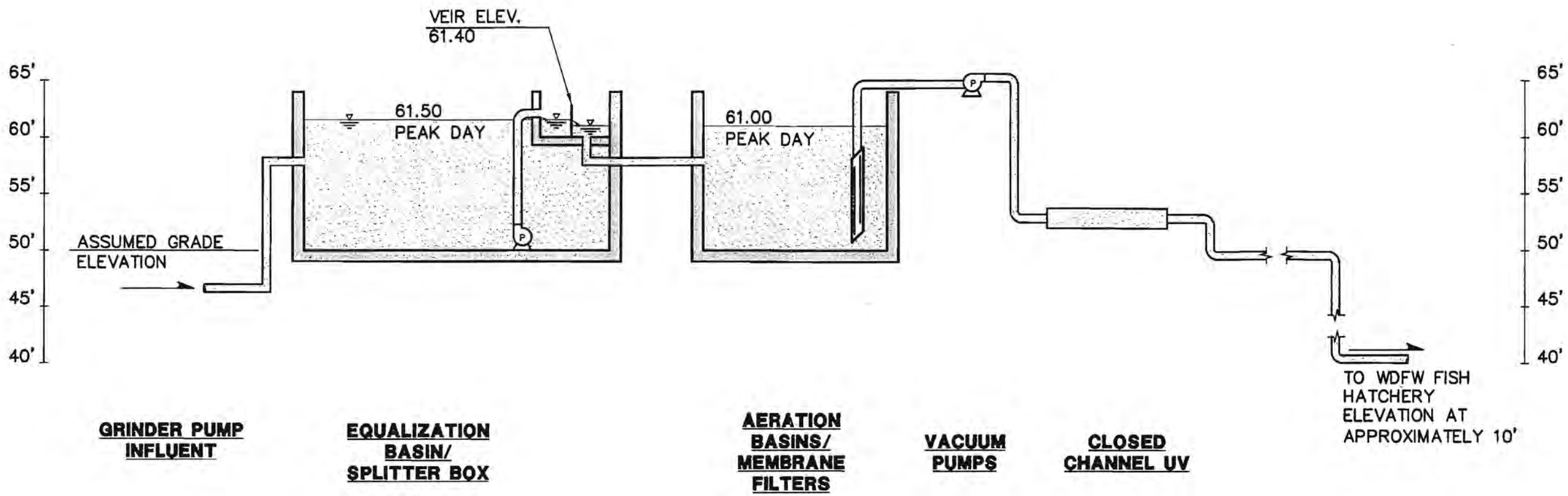

HYDRAULIC PROFILE

SCALE: HORIZONTAL NOT TO SCALE
VERTICAL ELEVATION SCALE 1"=10'

MASON COUNTY PUD NO. 1

FINCH CREEK WASTEWATER FEASIBILITY STUDY
WATER RECLAMATION FACILITY

HYDRAULIC PROFILE
FIGURE 9-3

Gray & Osborne, Inc.
CONSULTING ENGINEERS

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Chapter 10 will present information on potential funding the Mason County PUD No. 1 may want to pursue. It is recommended that the PUD No. 1 investigate the potential for receiving funding for each of the two projects and also evaluate means of reducing capital costs, e.g. self-help, prison labor, etc. Following these efforts and meetings with the community, the PUD No. 1 would be positioned to determine a course of action.

CHAPTER 10

CHAPTER 10

FINANCING

Based on the estimated total project costs presented in Chapter 9, a significant amount of funding support will have to be secured in order to implement either of the proposed projects. There are several state and federal agencies that administer funding programs for wastewater infrastructure projects, and these would all be potential sources of funding. Other potential sources of funding are general facilities charges and formation of an improvement district.

AGENCY FUNDING SOURCES

Agency funding sources include two state agencies, the Washington Departments of Ecology and Community Trade and Economic Development, and two federal programs under the U.S. Department of Agriculture, the Rural Development Program and the Forest Service Community Assistance Program. A summary of the available programs is presented in Table 10-1, and each is discussed below.

TABLE 10-1

Potential Sources of Grant and Loan Project Funding

Agency	Grants	Loans
Department of Ecology	Centennial Clean Water Fund	Centennial Clean Water Fund
		State Revolving Fund
Department of Community Trade and Economic Development	Community Development Block Grant • General Purpose Grant • Community Investment Fund (CIF) Grant	Public Works Trust Fund
USDA Rural Development	Rural Development Grant	Rural Development Loan
USDA Forest Service	Community Assistance Program	

DEPARTMENT OF ECOLOGY

The Department of Ecology (Ecology) provides both grant and loan funding for water pollution control and improvement projects. The Centennial Clean Water Fund is a state-funded program that offers grants and loans for infrastructure and activity projects. Low-interest loans are also available for the same types of projects through the State Revolving Fund, a federally seeded loan program.

Centennial Clean Water Fund

The Centennial Clean Water Fund (CCWF) provides funding for infrastructure projects such as wastewater collection and treatment facilities, as well as activity projects such as watershed planning or stream restoration. The Legislature recently issued provisos to dedicate grant funding to facility projects in small towns and to projects that benefit depressed salmonid stocks.

The CCWF will fund all phases of wastewater systems development, including planning, design, and construction. However, only comprehensive sewer planning and treatment facility construction are considered eligible for grants. Facilities planning and design costs are not eligible for grants; they can only be funded with loans.

Collection sewers, pump stations, and transmission mains are eligible for loans only. Land acquisition for the treatment facility may also be funded through a loan. For the construction of wastewater treatment facilities, a maximum grant of $2.5 million per project per funding cycle may be awarded. Grant awards for facility projects are generally limited to 50 percent of the eligible project costs. If commercial, industrial or institutional flows contribute greater than 20 percent of the total design flows to the treatment facility, the eligibility of the facility may be decreased. Also, CCWF funding for excess capacity at the facility is limited to 10 percent and is only loan eligible. Supplemental financial assistance may be available when project costs result in financial hardship to the community. The hardship level is determined as a total sewer rate, including debt service and operation and maintenance costs, in excess of 1.5 percent of the median household income. If an applicant qualifies for hardship status, addition grant funding may be offered.

Grant funds from the CCWF program are allocated annually on a competitive basis. The application period is in February of each year and offers are normally made by July of the same year. Ranking criteria for CCWF grants and loans include the potential for water quality protection or improvement for the affected water body, the presence or absence of a health emergency based on the existing conditions, or the need to assist an existing facility to meet an enforcement or compliance order. Projects that address public health

concerns, have a high local priority, or address existing compliance issues are considered high priority and receive additional points in the ranking process.

State Revolving Fund Loan Program

Ecology also administers the State Revolving Fund (SRF) loan program, which provides low interest loans for wastewater collection and treatment facility projects, as well as some activity projects. Loans can be made for up to 100 percent of project costs, including planning, design and construction. Loans may be made for up to 20 years, at rates generally ranging from 0 percent up to 75 percent of market rate. However, this past year, rates ranged from 0 to 1.5 percent, depending on the length of the loan. Also, if a project creates a financial hardship for the community, interest rates may be lowered or the length of the loan extended.

The SRF can be used to fund excess capacity for up to 20 years of future growth. Land acquisition is not eligible for SRF funding. Also, if commercial, industrial or institutional flows contribute greater than 20 percent of the total design flows, the eligibility of the facility may be decreased.

Applications for SRF are received in February at the same time as the CCWF applications. A single application may be made for both programs. The rating and ranking criteria are also essentially the same. Funding of projects frequently involves a package that includes both CCWF grant and SRF loan funds.

DEPARTMENT OF COMMUNITY TRADE AND ECONOMIC DEVELOPMENT

The Department of Community Trade and Economic Development (CTED) administers the federal Community Development Block Grant program, which is funded through the U.S. Department of Housing and Urban Development. The CTED also administers the Public Works Trust Fund, a state revolving loan program. Both of these programs offer funding for infrastructure projects in counties and local communities.

Community Development Block Grant

The Community Development Block Grant (CDBG) program provides grants for a wide range of community development projects, including public utilities. Eligible utility projects include the planning, design and construction of water and wastewater systems. The CDBG funding is available through two different means, the General Purpose grant and the Community Investment Fund grant.

Under the General Purpose program, an applicant can apply for up to $750,000 in grant funds per funding cycle. There are no matching funding requirements. All projects must principally benefit low- and moderate-income (LMI) persons, as defined through CDBG

criteria. Public utility districts are not eligible to apply directly to the CDBG program, however, a utility can request that the county apply on their behalf. Applications are received in November, with notification of award by February or March of the following year.
The Community Investment Fund (CIF) is a CDBG program offered to a very limited number of communities and counties each year. Funds are designated for infrastructure projects that have the potential to promote economic development and provide new jobs or retain existing jobs. Grants may be offered for up to $1 million, with no matching funding required. To be eligible to receive funding from CIF, a community or county must be ranked in the top three on the County's priority list developed through the WA-CERT (Washington Community Economic Revitalization Team) process. Funding agencies use the local WA-CERT rankings to evaluate local priorities and support of projects.

Application for a CIF grant is not open to any project that may be eligible. The project's sponsor must be requested by CTED to apply for CIF funding. The criteria CTED uses for selecting projects include potential for job creation or retention, benefit to low and moderate income populations, and readiness to proceed.

Public Works Trust Fund

The Public Works Trust Fund (PWTF) is a state revolving loan fund program that provides funding for utility infrastructure projects. Eligible projects include planning, design and construction of wastewater collection and treatment systems. Project costs due primarily to projected growth are considered ineligible. Such costs do not make a project ineligible, but must be excluded from the scope of the PWTF proposal.

The PWTF offers loans substantially below market rates, payable over periods ranging up to 20 years. Recent interest rates have ranged from 0.5 to 2.0 percent, depending on the matching funds supplied by the applicant. The lowest interest rate is obtained by contributing a higher local share. The applicant must provide a minimum of 10 percent local matching funds. This requirement can be met by other loan funds if community funds are utilized to pay back the loan. With 10 percent matching funding, a 2 percent interest rate is available. A 20 percent local share qualifies the applicant for a 1.0 percent, and a 30 percent local share qualifies for a 0.5 percent rate. The useful life of the project determines the loan term, with a maximum of 20 years. Loans may be offered up to a maximum of $3.5 million.

Application may be made by a local government such as a city or county, or a special purpose utility district. The application must also have a long-term plan for financing its public works needs. If the applicant is a county or city, it must adopt a 0.25 percent real estate excise tax. Applications are received in May; however, funding offers are not made until approximately one year later. Once the proposed funding list is developed, it must

be approved by the State Legislature, which then appropriates the funding for the projects. It is not until after the appropriation bill is signed that actual loan offers are made.

UNITED STATES DEPARTMENT OF AGRICULTURE

The United States Department of Agriculture (USDA) administers funding for small communities through the Rural Development Agency. The USDA Forest Service also provides a limited amount of grant funding in timber affected communities.

Rural Development

The USDA Rural Development (RD) Agency offers funding for small community infrastructure projects through both a grant and loan program. Planning and design costs are eligible for RD funding, but only through reimbursement after the project is bid for construction. Therefore, the community must fund planning and design by other means until construction begins.

Loans are offered for terms up to 40 years; the current interest rate is at 4.5 percent. Grants may be awarded when the costs of a project result in a financial hardship for the community. Hardship is determined as the annual debt service portion of the utility rate equal to or exceeding 1.0 percent of the community's median household income. Application may be made to RD at any time, and offers are made as funds become available each year.

Forest Service Community Assistance Program

The USDA Forest Service (Forest Service) administers a grant program that offers a limited amount of funds each year. The funds are generally directed to timber affected areas for a variety of community development or assistance projects. Grants may be available for up to $50,000, with a 20 percent match requirement. The Forest Service does not fund construction, but will fund many components that support planning or design of projects. Eligible projects include planning documents, e.g. this feasibility study, environmental reports, biological assessments, and geotechnical evaluations. Applications may be submitted at any time, with awards made as funds become available each year.

OTHER POTENTIAL FUNDING SOURCES

In combination with agency grants and loans, the PUD No. 1 could secure funding for construction through general facilities charges. The PUD No. 1 could also seek to form a utility local improvement district to provide some funding for a project. Each of these funding sources is discussed below.

GENERAL FACILITIES CHARGES

A general facilities charge (GFC), frequently referred to as a connection fee or charge, is a one-time charge assessed against new sewer customers as a way to recover a part or all of the capital cost of a system constructed for their use. The intent is that all system customers pay an equitable share of the cost of the system. General facilities charges may include the pro-rata share of the original cost of the system and the pro-rata share of any future improvements anticipated over the next several years. Therefore, for general facilities charges to work effectively, they must periodically be reviewed to ensure that they accurately reflect the true costs associated with planned improvements.

UTILITY LOCAL IMPROVEMENT DISTRICTS

Another potential source of funds for improvements is through the formation of Utility Local Improvement Districts (ULID) involving an assessment made against properties benefited by the improvements. ULID bonds are further guaranteed by revenues and are financed by issuance of revenue bonds.

ULID financing is frequently applied to wastewater system construction and expansion. Typically, ULIDs are formed by a city or county at the written request (by petition) of the property owners within a specific area. Upon the receipt of a sufficient number of signatures or petitions, the local improvement area is defined, and the improvement is designed and constructed. Each separate property in the ULID is assessed in accordance with the special benefits the property receives from the wastewater system improvements. The assessment places a lien on the property and must be paid in full upon sale of the property. Further, property owners may pay the assessment immediately upon receipt, reducing the costs financed by the ULID.

If the ULID assessment is paid in installments, it may be eligible to be deducted from federal income taxes. Low-income senior citizens may be able to defer assessment payments until the property is sold. Also, CDBG funds may be available to property owners with incomes near or below poverty level to help reduce assessments. The major disadvantage to the ULID process is that it may be politically difficult to approve formation. The ULID process can be stopped if owners of 40 percent of the property area within the ULID boundary protest its formation. Also, utilization of a ULID increases total project costs by a factor of about 1.3.

APPENDIX A

PART 3: HEALTH DEPARTMENT REVIEW (OFFICIAL USE ONLY)

SOIL LOGS AND SITE CHARACTERISTICS

LOT # A	LOT # B	LOT # ___	LOT # ___
Test Pit A 0-25 very sandy loamy sand 25-56 (PR)	Test Pit A (PR) 0-20 loamy sand ext. gravelly 20-60 (PR) compacted coarse sand & gravel	Test Pit A	Test Pit A
Depth of root pen.: 50 Depth of mottling: ___ Depth to rest. layer: 25 Soil type (USDA): PR	Depth of root pen.: 60" Depth of mottling: ___ Depth to rest. layer: 0 Soil type (USDA): PR	Depth of root pen.: ___ Depth of mottling: ___ Depth to rest. layer: ___ Soil type (USDA): ___	Depth of root pen.: ___ Depth of mottling: ___ Depth to rest. layer: ___ Soil type (USDA): ___
Test Pit B	Test Pit B	Test Pit B	Test Pit B
Depth of root pen.: ___ Depth of mottling: ___ Depth to rest. layer: ___ Soil type (USDA): ___	Depth of root pen.: ___ Depth of mottling: ___ Depth to rest. layer: ___ Soil type (USDA): ___	Depth of root pen.: ___ Depth of mottling: ___ Depth to rest. layer: ___ Soil type (USDA): ___	Depth of root pen.: ___ Depth of mottling: ___ Depth to rest. layer: ___ Soil type (USDA): ___
Curtain drain needed? N Slope (%): 1 Shoreline? (Y/N): N Minimum lot size:[A] 1AC	Curtain drain needed? ___ Slope (%): 1 Shoreline? (Y/N): N Minimum lot size:[A] 1AC	Curtain drain needed? ___ Slope (%): ___ Shoreline? (Y/N): ___ Minimum lot size:[A] ___	Curtain drain needed? ___ Slope (%): ___ Shoreline? (Y/N): ___ Minimum lot size:[A] ___

[A] Minimum lot size applies to new subdivisions and is defined as the minimum allowable land area per residence or residential equivalent (450 gallons per day).

COMMENTS

PART 3: HEALTH DEPARTMENT REVIEW (OFFICIAL USE ONLY)

SOIL LOGS AND SITE CHARACTERISTICS

LOT # 1	LOT # 2	LOT # 3	LOT # 4
Test Pit A	Test Pit A	Test Pit A	Test Pit A
0-20 sandy loam 20-31 (1A) coarse sand & gravel 31-52 loamy sand-	0-16 gravelly sandy loam 16-30 very gravelly loamy sand 30-37 (1A) coarse sand & gravel	0-27 gravelly loamy sand 27-37 mottled ~~weathered~~ till cemented sand & gravel	0-12 gravelly sandy loam 12-59 (1A) coarse sand & gravel
Depth of root pen.: 50	Depth of root pen.: 50	Depth of root pen.: 27	Depth of root pen.: 56
Depth of mottling: —	Depth of mottling: —	Depth of mottling: 27	Depth of mottling: —
Depth to rest. layer: 20	Depth to rest. layer: 30	Depth to rest. layer: 27	Depth to rest. layer: 12
Soil type (USDA): 1A	Soil type (USDA): 1A	Soil type (USDA): 3	Soil type (USDA): 1A
Test Pit B	Test Pit B	Test Pit B	Test Pit B
		0-16 gravelly sandy loam 16-44 (1A) coarse sand & gravel 44-59 loamy to med. sand.	
Depth of root pen.:	Depth of root pen.:	Depth of root pen.: 55	Depth of root pen.:
Depth of mottling:	Depth of mottling:	Depth of mottling: —	Depth of mottling:
Depth to rest. layer:	Depth to rest. layer:	Depth to rest. layer: 16	Depth to rest. layer:
Soil type (USDA):	Soil type (USDA):	Soil type (USDA): 1A	Soil type (USDA):
Curtain drain needed? N	Curtain drain needed? N	Curtain drain needed? N	Curtain drain needed? N
Slope (%): 2	Slope (%): 3	Slope (%): 2	Slope (%): 2
Shoreline? (Y/N): N	Shoreline? (Y/N): N	Shoreline? (Y/N): N	Shoreline? (Y/N): N
Minimum lot size:[A] 1AC	Minimum lot size:[A] 1AC	Minimum lot size:[A] 1AC	Minimum lot size:[A] 1AC

[A] Minimum lot size applies to new subdivisions and is defined as the minimum allowable land area per residence or residential equivalent (450 gallons per day).

COMMENTS

APPENDIX B

THURMAN PROPERTY
ASSESSMENT FOR WASTEWATER APPLICATION

PRELIMINARY HYDROGEOLOGIC EVALUATION
HOODSPORT, WASHINGTON

July 5, 2000

Prepared for:

Gray & Osborne, Inc.

GROUND WATER SCIENCE SERVICES, LLC

THURMAN PROPERTY
WASTEWATER APPLICATION
PRELIMINARY HYDROGEOLOGIC EVALUATION
HOODSPORT, WASHINGTON

1.0 INTRODUCTION

PROJECT OBJECTIVES

Ground Water Science Services (GWSS) was contracted by Gray and Osborne, Inc. to perform a preliminary hydrogeologic evaluation of a 14.37 acre land parcel designated as the James Thurman property. This parcel is located on a flat topographic area located approximately 200 feet above the town of Hoodsport on the north side of Finch Creek County Road.

The key purpose of the preliminary evaluation was to very quickly and cost effectively assess whether the Thurman property has "potential" for the application of treated wastewater for either spray or drain field technology. The scope of work did not include ground water modeling or placement of ground water monitoring wells but was intended as a first examination of property to assess types of soils and hydrogeologic conditions encountered.

SCOPE OF WORK

To complete this investigation Ground Water Science Services completed the following tasks:

- Logged and sampled soils from 6 newly excavated test pits and 4 old excavated test pits
- Conducted 6 field reconnaissance visits to investigate drainages, vegetation, topography and other hydrologic features including the examination of springs, seeps and saturated soil conditions;
- Examined and mapped geologic outcrops and hydrostratigraphy;
- Conducted 3 falling head infiltration tests;

- Sampled and analyzed ground water from a spring for background water quality parameters;

- Prepared a series of site maps showing test pits and major topographic and hydrogeologic features;

- Reviewed regional literature and studies including:

  1) "Soil Properties and Genesis of Pans in Till-Derived Andisols, Olympic Peninsula" by M.A. Wilson, 1996, Soil Science Society of America Journal, v. 60, no.1, p.206-208.

  2) "Geologic Map of North Central Mason County" by R.J. Carson, 1976, DNR Library Files.

  3) "Application of Paleomagnetism, fission-track dating, and tephra correlation to Lower Pleistocene sediments in the Puget Lowland, Washington" by D.J. Easterbrook, J.L. Roland, R.J. Carson and N.D. Naeser, 1988, Geological Society of America Special Paper, 227.

  4) Department of Ecology Well Logs.

- Analyzed soils for grain size, cation exchange capacity, salinity, moisture content and other key soil properties;

- Graphed and classified soils to USCS soils classification;

- Conducted preliminary analyses on nitrate and hydrologic infiltration impacts;

- Prepared a geologic cross section;

- Prepared this final report with recommendations.

FINDINGS

The results of our investigation show the following:

- ***Site soils are capable of infiltrating treated wastewater***. Infiltration tests and test pit logs show the site is underlain by 8 to 10 feet of gravelly sands and sandy loamy gravels above the first major till unit. Observed infiltration rates are 6 inches to greater than 20 inches per hour for topsoils and underlying gravelly sands and sandy gravels. *There were areas on the site however, on the west end*

of the property where some of sandy gravels appeared to be highly compacted and/or cemented which would reduce infiltration rates.

- ***Cation exchange capacities (CEC) of soils range from 14 to 27 meq/l.*** CEC's are generally low values indicating low to moderate sorption of ionic compounds will occur onto negatively charged clay particles. Local soil studies show a predominance of kaolinite and gibbsite clays which have generally lower CEC's than smectite clays for example, and are typically formed in high leaching conditions or in rapid water movement areas. Hoodsport soils are typically low in silica.

- ***Ground water was not encountered during the excavation of any of the test pits.*** Sandy loamy gravels in all test pits were dry and had some iron oxide staining but no moisture was observed. Some of the lower elevation till exposures, however, had small seeps and moist soils were observed on the road traverses the Jennifer Thurman-Williams property. A seasonal water table may develop on the Thurman property, however, during the rainy season between October and April. *Water table development should be investigated further through the placement of shallow monitoring wells.*

- ***Background water samples taken from the stream located on the north side of the James Thurman property exhibit low conductivity, low turbidity and total dissolved solids.*** Alkalinity was predominantly bicarbonate alkalinity representing rainfall type water versus ground water. Total coliform was exhibited on the second round of sampling at low levels, however there was no fecal coliform, as anomalously observed during the first sampling round. The low coliform reading may indicate deer or other animal waste in near the stream.

- ***Very low concentrations of nitrate and phosphate were also observed at .25 mg/l and .02 mg/l respectively.*** It is strongly suggested that another round of stream sampling for these parameters as well be conducted to confirm presence of coliform, nitrate and phosphate. Nitrate and phosphate are not typically lab or sampling artifacts or are as sensitive as coliform analyses, so the extremely low levels are worth observing further.

- ***Soft moist soils and some slope instability were observed on the bluff and road which traverses the Jennifer Thurman-Williams property.*** The presence of soft moist soils (fine sandy loams) and slumping may indicate some seasonal ground water movement through the sandy loams and gravels and even through the lower till units at the top of the bluff. There also appears to be ground water movement towards the un-named stream to the north on the James Thurman property, as the creek steadily gains water as it enters Finch Creek (as observed on PUD commissioners property). *It will be critical to prevent excessive groundwater movement into these areas and application of spray field wastewater would have to be carefully monitored.*

- ***Preliminary estimates for applying wastewater from the town of Hoodsport show that there is capacity for infiltrating wastewater (either sprayfield or drainfield) at average rates of 5800 gpd and no greater than 14,500 gallons per day on a peak day.*** Key areas on the site are underlain by highly permeable sandy gravels which are capable of transmitting water very rapidly and laterally towards the stream above the till unit. Because ground water was not encountered in early May (and deeper clayey soils had no moisture content) it is not known what net site infiltration behavior is and whether a significant water table does form in winter months. We would expect that significant volumes of rainfall would infiltrate into surface soils and flow horizontally towards the stream face at 11 to 283 feet per day. Some areas of the site has clayey surface soils which may inhibit effective infiltration. It is not recommended that higher flow volumes be applied to this land parcel due to lack of significant buffer space on property and increase of surface water flow to downgradient properties and/or slope failure concerns.

- ***The most ideal area for a drainfield is in the highly permeable sandy gravels east of the storage shed in the central portion of the site.*** Lateral permeabilities are likely to be high and minimize bio-clogging which may be a problem in sediments with significant finer particle sizes. Breakthrough above ground surface or mounding is not expected for these sands and gravels as they exhibit high hydraulic conductivities. Some gravels in the area exhibit some cementation which would reduce permeability and could create mounding problems. Typical trench lengths should be 10 to 100 feet in length based on hydraulic conductivities of 11 to 283 feet per day.

- ***Winter hydrologic studies are highly recommended to determine if a water table forms in the upper permeable soils of the site and to observe general surface and ground water flow conditions towards the slope/bluff faces and properties located at the base of the stream.***

- ***Preliminary ground water storage calculations in the sandy gravels could be as much as 215,000 cubic feet or 1.6 million gallons in a 4.0 acre area.*** Assuming that no significant water table exists below the Thurman property breakthrough or oversaturation of soils above ground surface is not likely to occur. Highly transmissive (high hydraulic conductivities) sandy gravels will transport infiltrated water laterally towards stream or bluff face. There are areas on the James Thurman property which should be avoided for direct application/infiltration of treated wastewater because the till and/or clay is only within 4 feet of ground surface (such as the area forming the head of the un-named stream on the west end of the James Thurman property).

SITE DESCRIPTION

TOPOGRAPHY

The James Thurman property is located on a relatively flat topographic area approximately 800 feet upslope from Finch Creek County Road #44880 (Figure 1 and 2). The property encompasses around 14.37 acres of which roughly half is topographically flat land that is not incised by a stream or steep side slopes. The site can be accessed by an un-maintained dirt road from the Finch Creek County Road.

The eastern boundary of the Thurman property is bordered by the Jennifer Thurman-Williams parcel which consists of 7.98 acres of which only half is useable or consists of topographically 'flat' property. The remaining portion of the property is very steep with greater than 60% grade side slopes.

The topographic elevations on the James Thurman parcel range from 220 to 240 feet above mean sea level (msl). A small deeply incised stream (unnamed) crosses the site from the southwest to the southeast. As the stream crosses the western Thurman property border it is a dry flat creek bed (as observed on all site visits even during the rainy days) which very quickly becomes a steeply incised flowing stream. GWSS obtained background water quality samples from a seep on the lower sidewalls of the streambed where the water first 'daylights' into the stream. However, this water may not be actual ground water but surface water flow within the streambed itself.

VEGETATION

Most of the property is covered by thick brush and forest which had to be cleared to dig test pits. The forested areas include firs, Western Cedars, large flowering dogwood trees and host of other trees including some extremely large vine maples with considerable undergrowth and a large epiphyte canopy (ferns, mosses, lichens). Ferns, mosses, vines, lichens and decomposed tree and brush litter compose a soft organic rich layer above the topsoil which is likely to have a high organic carbon content capable of sorbing contaminants and ionic compounds. There are also a variety of flowering plants which are excellent at nitrate uptake.

GEOLOGY AND HYDROGEOLOGY

The site is underlain by Quaternary glacial sediments of the Vashon Glaciation (Vashon Recessional Outwash and Till) and deeper sections in the vicinity have been mapped as sediments from the Olympic Alpine glaciers and the Cordilleran Ice Sheet sediments which include the Clark Creek and Anna's Bay Drift respectively (Easterbrook and others, 1988). Extensive units of gray, stratified lucustrine silts have been documented overlying the Clark Creek till (Olympic Alpine). During our investigation of some of the

test pits we encountered similar gray, stratified lucustrine sediments at depths of 8 feet below ground surface (bgs) which likely represent the damming of melt waters from alpine and the Vashon glaciers (Figure 3).

The general site stratigraphy is as follows (depths in feet below ground surface):

0-1' soft to stiff organic brown to red fine sandy silts with gravel
0-2' medium dense gravelly sandy silts (Hoodsport Gravelly Sandy Loams)
2-10' medium dense coarse sandy gravels and gravelly sands
>10' dense gray till/perching layer

The till was encountered at less than 6 feet below ground surface in test pits TP-3 on the western end of the James Thurman property. This area is likely to become saturated during heavy rainfall.

Test Pits

GWSS logged and monitored the excavation of 6 test pits on May 3, 2000 and re-examined open older test pits. Representative soil samples were taken and submitted to Soil and Plant Laboratory, Inc. in Santa Clara, California for grains size analyses (USDA), pH, salinity, organic content, half saturation percentage and cation exchange capacity (CEC). Test pits were backfilled with native soil immediately upon completion of logging. Test pit logs are provided in the appendices.

Soils

Grain size analyses show that the site is underlain predominantly by "loamy sand, very gravelly" or "sandy loam, very gravelly" in the upper 1 to 2 feet below ground surface followed by "sand, very gravelly" or "gravels, very sandy" to depths up to 8 to 10 feet bgs, according to the USDA classification. Under the USCS classification these sediments would be classified as GM-GC and GW-GP respectively. There is a predominance of silt size particles in the matrix and greater clay content in the upper 1 foot of the soil column. Predicted infiltration rates for these soils would be around 2.4 inches per hour for "sandy loams, very gravelly" and upward to 20 inches per hour for the deeper "sands, very gravelly".

Nearby soil studies (Wilson, 1996) show the Hoodsport pans (also identified as the Hoodsport gravelly sandy loams) can have a high degree of cementation which is not simply a function of glacial compaction but of chemical weathering processes. We observed some of the gravel and sands to be fairly cemented and compacted (TP-4) which would have lower permeability than the more loosely packed gravels observed in the test pit TP-6 area.

Studies also indicate smectite clays are absent in the Hoodsport soils and secondary mineral phases are largely gibbsite and kaolinite which are indicative of a low-silica environment or high leaching and rapid ground water flow conditions. Slow ground water flow conditions are more likely to produce 2:1 layered clays with higher cation exchange capacities (CEC) such as smectites.

Cation exchange capacities for sandy loamy gravels range from 14.1 to 27.0 meq/100 grams in the topsoil and organic content in soils ranges from .2 to 2.6 percent. The upper one to two feet of top soil has good to moderate potential for sorption of contaminants and ionic compounds.

Ground Water

Ground water was not encountered in any of the test pits. All sediments were dry in early May with the exception of very moist soils encountered on the road traversing the Jennifer Thurman-Williams parcel just below the bluff. These silty fine sands appear to be picking up moisture from possible seeps emanating from the bluff. There were several areas which also exhibited slumping due to saturation of soils. *It becomes very important that excessive ground water flow conditions do not occur off or through the bluff face.* Therefore, identifying the location of sprayfield application will become very important. It maybe more optimal to have recharge-ground water flow towards the un-named stream at the north end of the James Thurman property which has year round flow and appears to have more stable side slopes.

Ambient Water Quality

Since ground water was not encountered on site we obtained background water quality samples from the un-named stream on the north portion of the James Thurman Property. We sampled and analyzed water for the following parameters: alkalinity, bicarbonate, carbonate, BOD, conductivity, hardness, total and fecal coliform, total dissolved solids, Total kjeldahl nitrogen, total phosphorous, turbidity, major anions (fluoride, chloride, nitrite, nitrate and sulfate) and total metals (barium, beryllium, chromium, copper, iron, manganese, nickel, sodium and zinc).

The waters exhibited very low turbidity, low conductivity and low total dissolved solids concentrations. Anion concentrations in water chemistry were sulfate, chloride and very low levels of nitrate. It is not clear why nitrate is in the stream sample as well as low concentrations of total phosphorus at .25 and .02 mg/l, respectively. These parameters are not as sensitive to false positive readings as total and fecal coliform which were anomalously high during the first sampling. During the second sampling only total coliform was present at 17 CFU/100 ml. Because the stream was sampled where a small amount of water first "daylights" into the stream bed this area may be prone to animal activity and therefore presence of some coliform may occur. Also, on several visits to the

site, nearby human activity was heard on the property bordering the west boundary of the site (children playing, dogs and wood chopping) and it is not known whether a home exists just west of the site. Ammonia nitrogen, TKN and metals were non-detect. It is recommended that another round of sampling for coliform, nitrate and total phosphorous be conducted. Also, background sampling of the stream at lower elevations before it enters Finch Creek may also provide useful information. Laboratory analytical results are provided in the Appendix.

Nearby Domestic and Water Supply Wells

All domestic and water supply wells for the vicinity appear to be located along Finch Creek and below successive till units which are representative of older glaciations. There is no immediate hydraulic connection between the receptor unit in the Vashon Recessional Outwash located at 220 feet above mean sea level above Hoodsport and domestic wells installed in lower outwash sediments under a series of two or three successive till layers. Surface water however, flowing down Finch Creek and the un-named site creek may eventually recharge lower systems.

Field Infiltration Tests

Three falling head infiltration tests were conducted in the upper 1 foot of soil (in the test pit TP-6 vicinity) which ranged from 6 to 20 inches per hour. Although falling head tests have limited predictive capability they are still useful in preliminary site assessments for observing soil infiltration behaviour. The test pit TP-6 area was identified as the best zone for infiltration of treated wastewater because there was a larger receptor unit in the Vashon Recessional Outwash which lacked any clayey-silty layers above the till which could create perched or mounded ground water conditions and gravels were non-cemented. In addition, higher cation exchange capacities were observed for topsoils in this area and root zones were much deeper. Old test pits observed in this vicinity also confirmed loosely packed non-cemented sandy gravels with a 1-foot thick sandy gravelly loam topsoil. The area to the east of the existing storage shed appears to be primarily loosely packed sandy gravels in the upper surface with deeper root zones than those encountered to the north or west of storage shed.

Hydraulic Conductivity Estimates

Hydraulic conductivity of sandy sediments can be estimated from grain size distribution curves using the Hazen method. This method uses the effective grain size (D10) as a measure of the particle that will most affect/inhibit ground water movement. By plotting grain size distributions in millimeters on semi-log paper the D10 value is estimated in centimeters and used in the Hazen approximation: $K=C(D10)^2$. Hydraulic conductivities ranged from .45 ft/day to an intermediate value of 45 ft/day and as high as 283 feet per day even with silty and clays size particles.

Preliminary Analyses on Infiltration Potential and Nitrate Loading

Spray field High Design Loadings versus Low Design Loadings -- Based on precipitation and estimates for evapotranspiration we conservatively estimated that the site was capable of infiltrating 2" per hour (more likely to average 6"). Minimum areas for a peak flow of 14,500 gpd would be 1.22 acres and average daily flows of 5800 gpd would require around .49 acres. Available useable acreage is around 4 to 6 acres. Higher flow volumes would require 4 or more acres.

Drainfield -- Small drainfields of minimal size would range from 10 to 100 feet based on our calculuations using estimated hydraulic conductivities. These could be designed in the most permeable sandy gravelly areas east and towards the interior of the site which would infiltrate average flow volumes of 5800 gpd and occasional peak volumes of 14,500 gpd. Concern for drainfields are bio-clogging of organics onto finer grain sizes of sediments. Test pits, however, show the upper soils are relatively cleans coarse sands and gravels which have hydraulic conductivities of greater than 50 feet per day. A 5 to 10 gpm application of waste water into a trench is not likely to produce mounding effects. Water would enter trench and leave trench towards stream at equivalent rates.

Because ground water was not encountered in early May and all sediments including silty clay perching layers were very dry (this was surprising) we are assuming that only a small water table actually develops during the winter and that there is a high degree of evapotranspiration from dense vegetation and deep root zones (>48" in TP-6 area).

Preliminary calculations for the potential storage volumes in the receptor unit (roughly half of the total acreage of the James Thurman property) can be as much as 2.6 million gallons or 352,800 cubic feet. Break through or oversaturation of soils (and consequently full use of hydraulic storage in the receptor unit) is not likely to occur on the site as the soils appear to be very well drained and water may move laterally on top of the till perching layer towards the stream and/or bluff. *It will be important to minimize severe hydraulic loading towards the bluff face, but this has to be closely examined and winter hydrologic monitoring should be conducted.*

Preliminary nitrate loading calculations show that with application of wastewater with average concentrations of 7 mg/l of nitrate, could result in ground water concentrations ranging from .10 mg/l (with dilution) to 3.8 mg/l (for no dilution). However this figure takes into account only conservative estimates for denitrification at 10% loss. In addition, both the organic carbon content of soils and cation exchange capacities of topsoils will promote sorption and breakdown of contaminants (for sprayfield application).

Conclusions and Recommendations

Assuming that a high water table does not form in the Vashon recessional sands and gravels (receptor unit), the James Thurman Property could potentially infiltrate treated wastewater at an average daily rate of 7000 gpd (or occasional peak rate of 14,500 gpd). Greater volumes of water may create slope stability problems in surrounding areas as water is likely to move laterally through the gravels on top of the first major till unit and outward towards the Jennifer Thurman-Williams parcel (bluff area) and/or towards the stream at the north end of the James Thurman parcel.

Cation exchange capacities for upper soils are low to moderate and play a more critical role in sprayfield application but will also help in the drainfield scenario as well. The higher percentage of organic carbon in the topsoil may allow good sorption of contaminants and ionic compounds.

Application rates of treated wastewater should be no more than 1 or 2" per day and more optimally around .5" inch per day over a 3 to 4 acre area. *It is highly recommended that winter hydrologic and ground water conditions be observed over the course of a winter to ascertain if a water table forms in the receptor unit above the till and to observe other hydrologic features such as flow volumes in the stream, formation of any wetlands and seep activity along bluff faces.* This would entail installing some piezometers (such as small diameter piezometers with a fairly less expensive and more portable direct-push drilling technology versus costlier traditional monitoring wells).

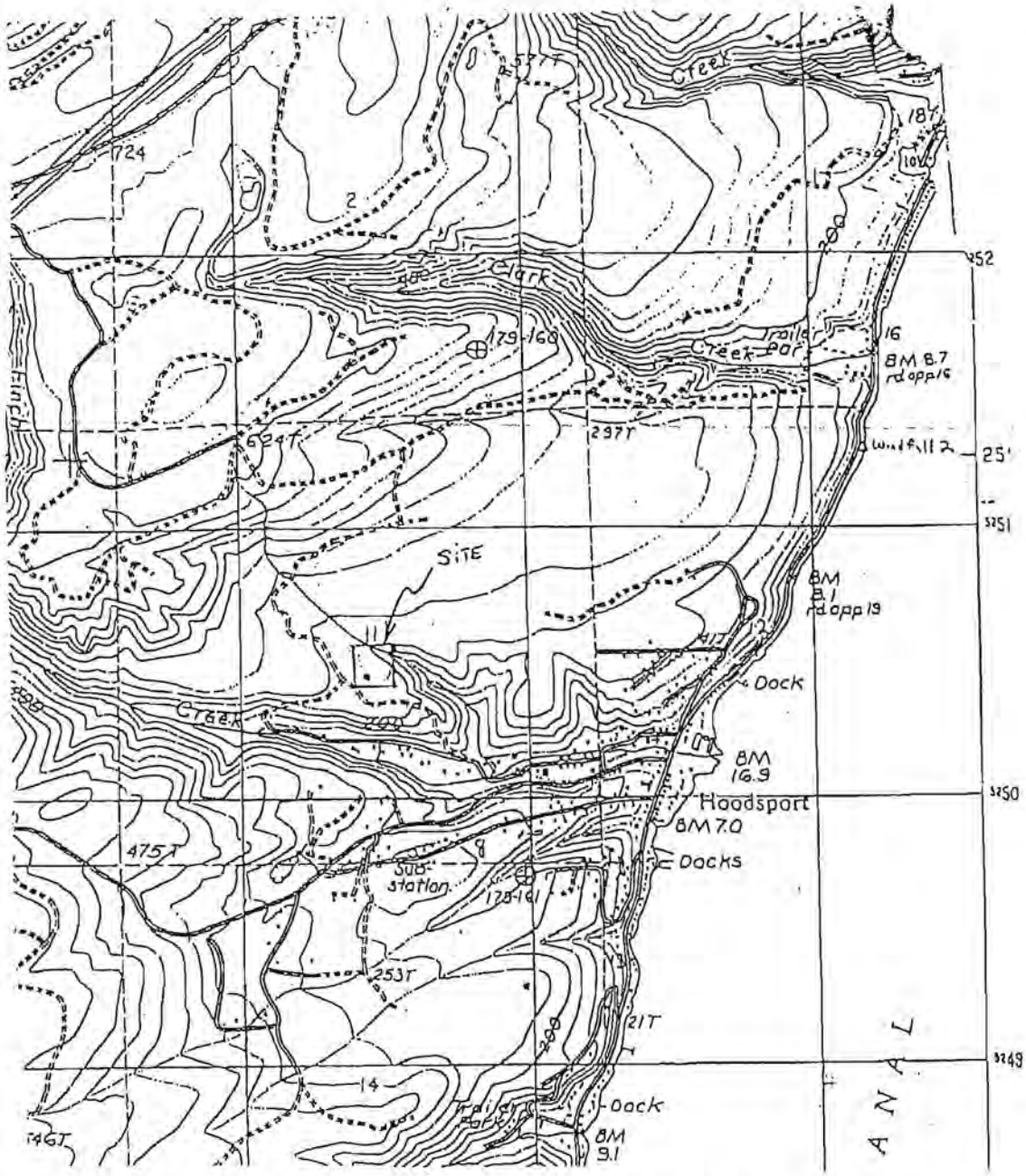


FIGURE 1 : SITE LOCATION MAP

Hydrogeologic Investigation
Hoodsport, Washington

Ground Water Science Services

James Thurman Property
(NW 1/4, SE 1/4, Section 11
TWN 22 N, R4W)

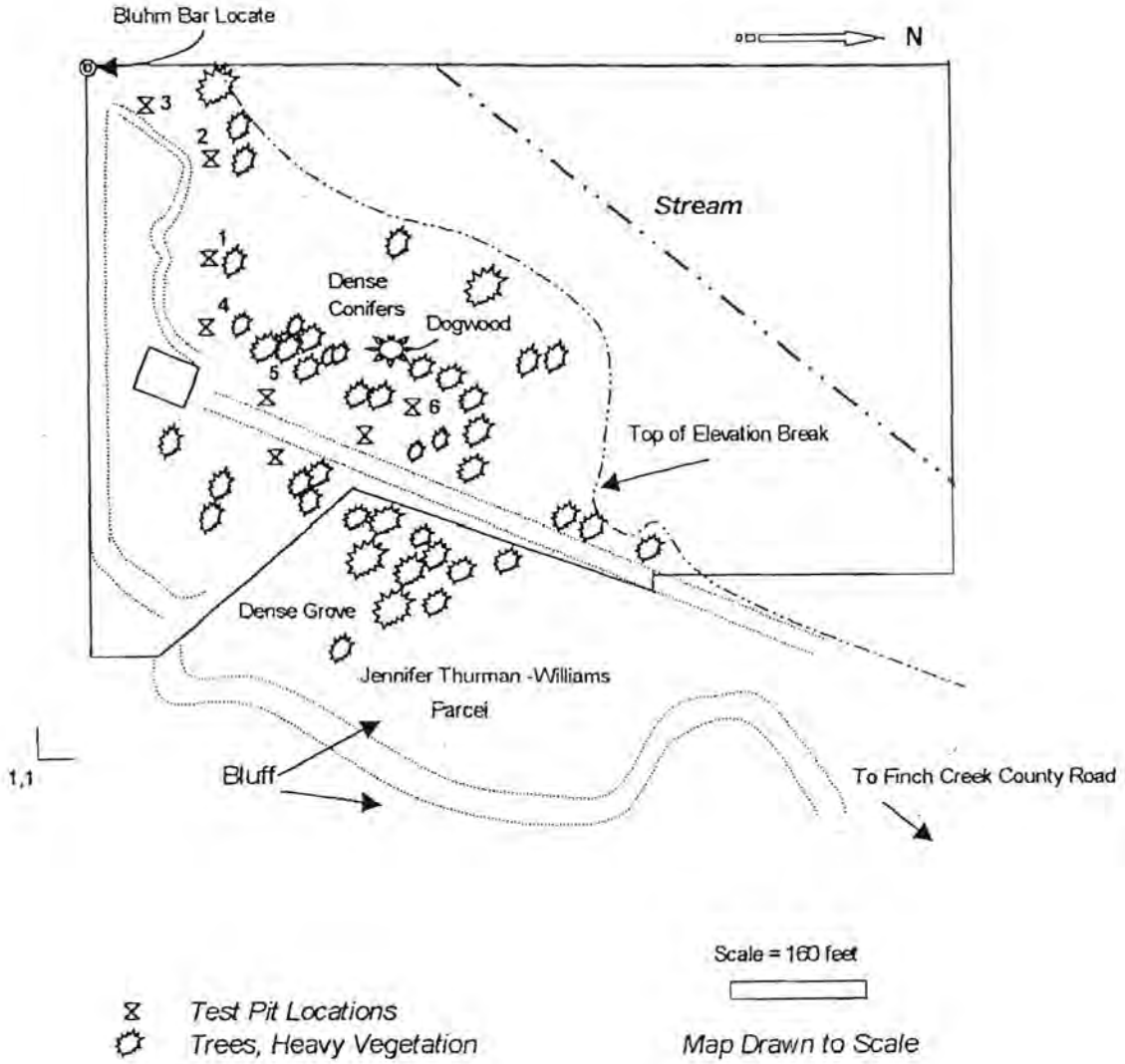


FIGURE 2 : Test Pit Location Map

Hydrogeologic Investigation
Hoodsport, Washington

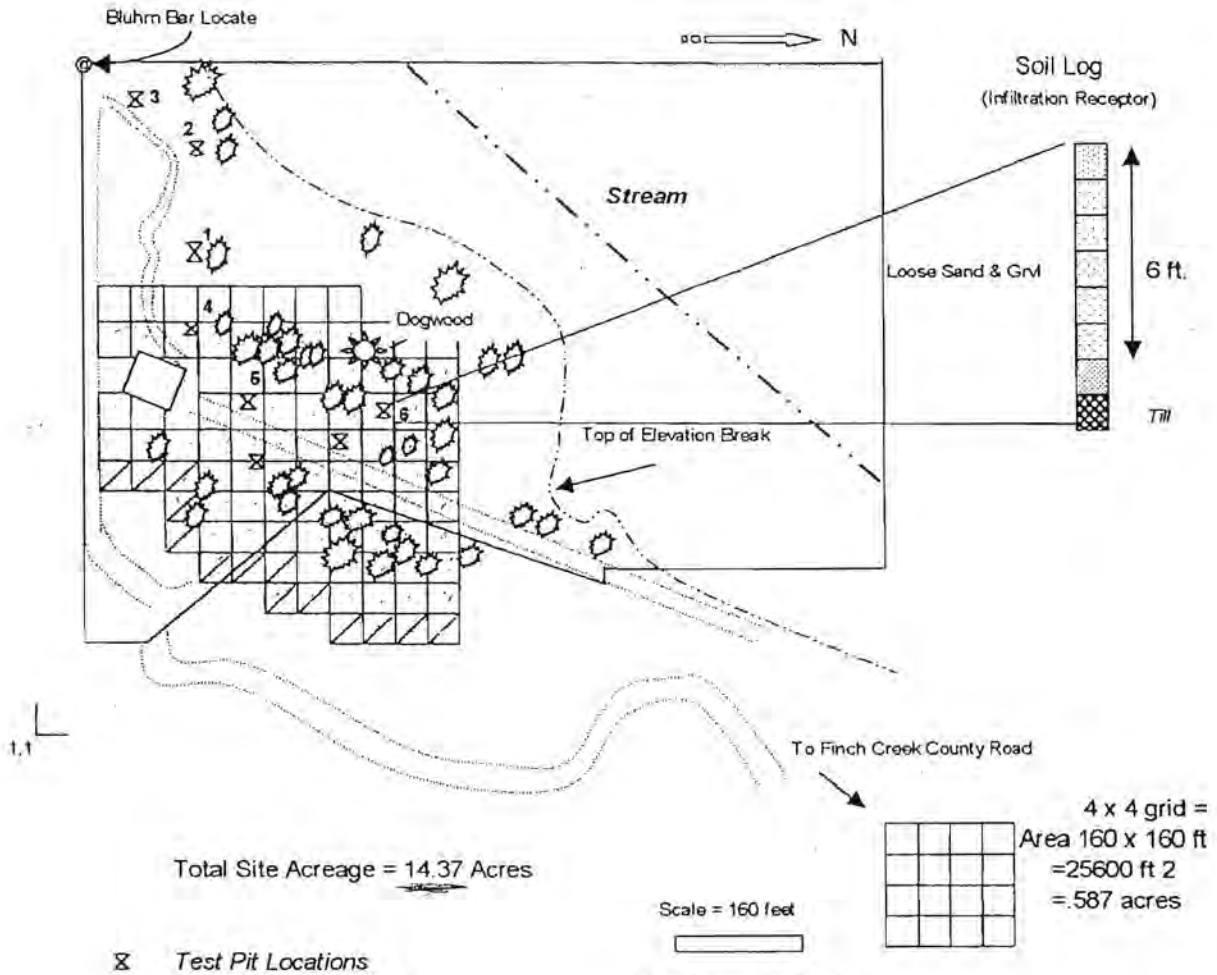
Ground Water Science Services

$100,000

30+ acres

91/16 = 5.69
+.587
3.5 ac

Thurman Property
(NW 1/4, SE 1/4, Section 11
TWN 22 N, R4W)

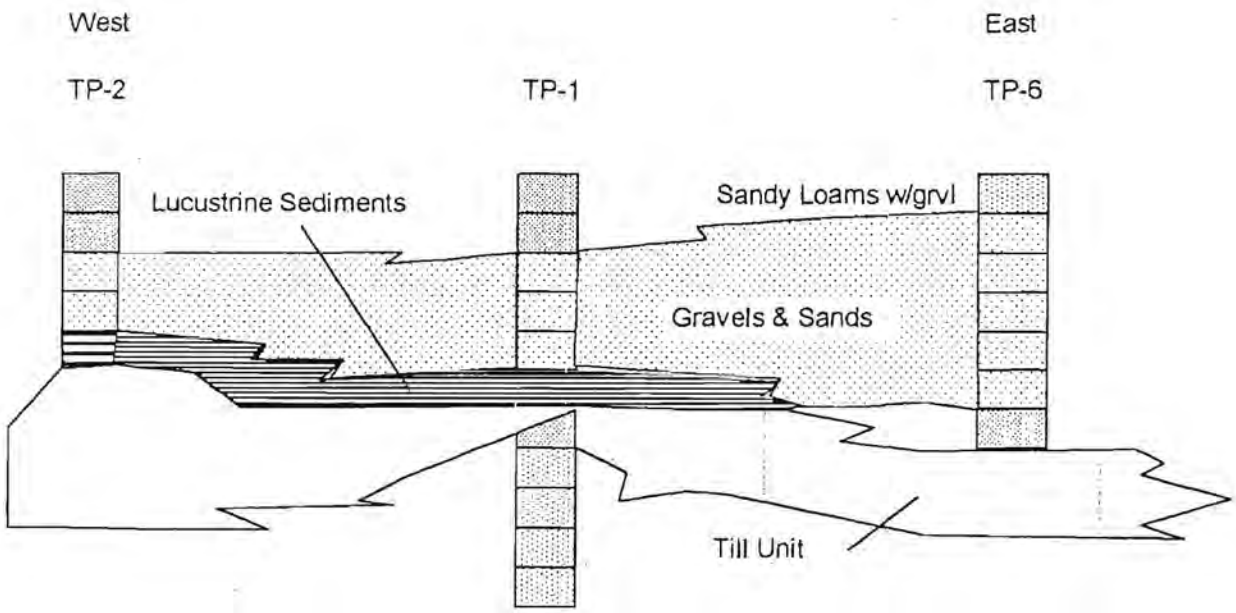


OPTIMAL AREA FOR INFILTRATION
(GRID)

FIGURE 3 : AREA IDENTIFIED FOR INFILTRATION OF TREATED WASTEWATER

Hydrogeologic Investigation
Hoodsport, Washington

Ground Water Science Services

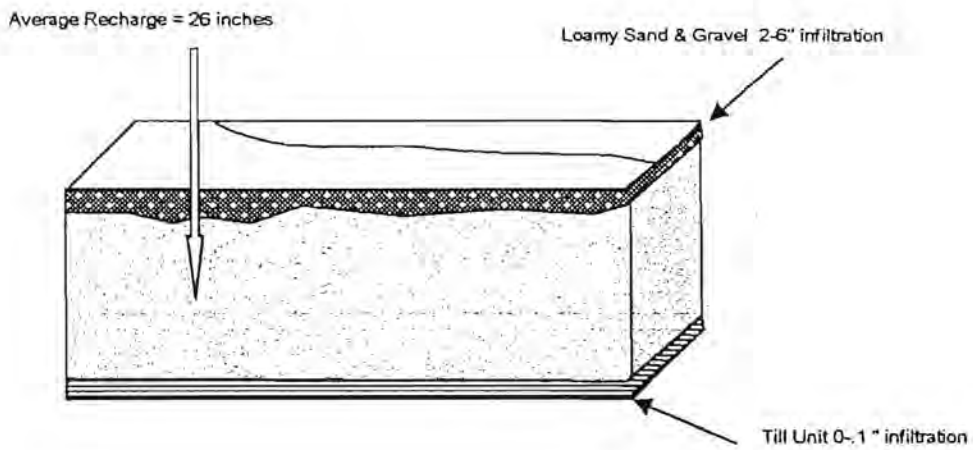


Ground Water Science Services

Thurman Property
Hoodsport, Washington

Geologic Cross-Secttion

FIGURE: 4



24 hr 100 yr

Site Runoff	Type	Hydro Gr	Curve SCS No.	Inches Runoff	Acres	sf/acre	Volume Runoff ft3
Wood/Forest	2nd Growth	A	55	1.87	4	43560	27152.4
Wood/Forest	2nd Growth	A	55	1.87	8	43560	54304.8

Precip Volumes

	Inches Precip	Acres	sf/acre	Volume Precip ft3
	1	4	43560	14520
	2	4	43560	29040
	3	4	43560	43560
yearly	90	4	43560	1306800
	1	8	43560	29040
	2	8	43560	58080
	3	8	43560	87120
	90	8	43560	2613600

Aquifer Storage

Porosity	Inches	Acres	sf/acre	Volume Storage ft3
0.2	96	4	43560	278784
0.2	96	8	43560	557568
0.3	96	8	43560	836352
0.45	96	8	43560	1254528

Cp= 5.5 mg/l
Cn= 7.83 mg/l nitrate concentration from the water reclamation facility

Inches per year to apply to site = 60

Design Flow gpd	Monthy gallons	Monthy ft3	Yearly ft3	Yearly Acre-ft	Acres w/o Rain Pd
21153	655743	87666.18	1051994	24.15046	4.83

	PET (inches) (1)	Mean Precip (inches) (2)	Adjusted Mean Precip (inches) (2a)	10 Yr. Return Period Precip (inches) (3)	Site Operation % (4)	Perc Rt Pw (in/month) (5)	Hydraulic Loading Rate Lw(p) (inches) (6)	Hydraulic Minimum Area Req'd (acres) (7)	U Nitrogen Uptake (kg-ha-mo) (8)	Loading Rate Based on Nitrogen Lw(n) (inches) (9)	Nitrogen Minimum Area Req'd (acres) (10)	Loading Rate (inches) (11)	Minimum Area Required (acres) (12)
January	0.14	13.42	13.42	15.433	50	28.8	13.507	1.787914	0.639659	65.39017	0.369329	13.507	1.787914
February	0.73	12.37	12.37	14.2255	50	28.8	15.3045	1.577925	3.335364	66.76881	0.361703	15.3045	1.577925
March	1.58	9.51	9.51	10.9365	50	28.8	19.4435	1.242027	7.219007	62.34209	0.387386	19.4435	1.242027
April	3.08	5.81	5.81	6.6815	50	28.8	25.1985	0.958365	14.07249	60.94204	0.396286	25.1985	0.958365
May	4.68	3.33	3.33	3.8295	50	28.8	29.6505	0.814467	21.38288	66.42975	0.363549	29.6505	0.814467
June	5.2	1.82	1.82	2.093	50	28.8	31.907	0.756867	23.75876	64.86232	0.372334	31.907	0.756867
July	6.38	1.19	1.19	1.3685	50	28.8	33.8115	0.714235	29.15017	74.61656	0.323661	33.8115	0.714235
August	5.29	1.54	1.54	1.771	50	28.8	32.319	0.747218	24.16997	64.50225	0.374413	32.319	0.747218
September	3.57	2.88	2.88	3.312	50	28.8	29.058	0.831074	16.3113	52.29141	0.461844	29.058	0.831074
October	1.62	7.62	7.62	8.763	50	28.8	21.657	1.115083	7.401767	53.77829	0.449075	21.657	1.115083
November	0.46	15.69	15.69	18.0435	50	28.8	11.2165	2.15302	2.101736	79.65344	0.303194	11.2165	2.15302
December	0.1	15.74	15.74	18.101	50	28.8	10.799	2.236258	0.456899	76.00071	0.317766	10.799	2.236258
	32.83	90.92	90.92	104.558			273.872						

Lw(p) - (Pr-ET) + Pw

Cp= 5.5 mg/l
Cn= 7.83 mg/l nitrate concentration from the water reclamation facility

Inches per year to apply to site = 60

Design Flow gpd	Monthy gallons	Monthy ft3	Yearly ft3	Yearly Acre-ft	Acres w/o Rain Pd
42305	1311455	175328.2	2103939	48.29978	9.66

	PET (inches) (1)	Mean Precip (inches) (2)	Adjusted Mean Precip (inches) (2a)	10 Yr. Return Period Precip (inches) (3)	Site Operation % (4)	Perc Rt Pw (in/month) (5)	Hydraulic Loading Rate Lw(p) (inches) (6)	Hydraulic Minimum Area Req'd (acres) (7)	U Nitrogen Uptake (kg-ha-mo) (8)	Loading Rate Based on Nitrogen Lw(n) (inches) (9)	Nitrogen Minimum Area Req'd (acres) (10)	Loading Rate (inches) (11)	Minimum Area Required (acres) (12)
January	0.14	13.42	13.42	15.433	50	28.8	13.507	3.575743	0.639659	65.39017	0.73864	13.507	3.575743
February	0.73	12.37	12.37	14.2255	50	28.8	15.3045	3.155775	3.335364	66.76881	0.723388	15.3045	3.155775
March	1.58	9.51	9.51	10.9365	50	28.8	19.4435	2.483995	7.219007	62.34209	0.774754	19.4435	2.483995
April	3.08	5.81	5.81	6.6815	50	28.8	25.1985	1.916684	14.07249	60.94204	0.792553	25.1985	1.916684
May	4.68	3.33	3.33	3.8295	50	28.8	29.6505	1.628895	21.38288	66.42975	0.727081	29.6505	1.628895
June	5.2	1.82	1.82	2.093	50	28.8	31.907	1.513698	23.75876	64.86232	0.744651	31.907	1.513698
July	6.38	1.19	1.19	1.3685	50	28.8	33.8115	1.428436	29.15017	74.61656	0.647306	33.8115	1.428436
August	5.29	1.54	1.54	1.771	50	28.8	32.319	1.494402	24.16997	64.50225	0.748808	32.319	1.494402
September	3.57	2.88	2.88	3.312	50	28.8	29.058	1.662109	16.3113	52.29141	0.923666	29.058	1.662109
October	1.62	7.62	7.62	8.763	50	28.8	21.657	2.230113	7.401767	53.77829	0.898128	21.657	2.230113
November	0.46	15.69	15.69	18.0435	50	28.8	11.2165	4.305939	2.101736	79.65344	0.606374	11.2165	4.305939
December	0.1	15.74	15.74	18.101	50	28.8	10.799	4.472411	0.456899	76.00071	0.635518	10.799	4.472411
	32.83	90.92	90.92	104.558			273.872						

Lw(p) - (Pr-ET) + Pw

Cp= 5.5 mg/l
Cn= 7.83 mg/l nitrate concentration from the water reclamation facility

Inches per year to apply to site = 60

Design Flow	Monthy	Monthy	Yearly	Yearly	Acres w/o Rain Pd
gpd	gallons	ft3	ft3	Acre-ft	
52881	1639311	219159.2	2629911	60.37444	12.07

	PET (inches) (1)	Mean Precip (inches) (2)	Adjusted Mean Precip (inches) (2a)	10 Yr. Return Period Precip (inches) (3)	Site Operation % (4)	Perc Rt Pw (in/month) (5)	Hydraulic Loading Rate Lw(p) (inches) (6)	Hydraulic Minimum Area Req'd (acres) (7)	U Nitrogen Uptake (kg-ha-mo) (8)	Loading Rate Based on Nitrogen Lw(n) (inches) (9)	Nitrogen Minimum Area Req'd (acres) (10)	Loading Rate (inches) (11)	Minimum Area Required (acres) (12)
January	0.14	13.42	13.42	15.433	50	28.8	13.507	4.469658	0.639659	65.39017	0.923295	13.507	4.469658
February	0.73	12.37	12.37	14.2255	50	28.8	15.3045	3.944701	3.335364	66.76881	0.904231	15.3045	3.944701
March	1.58	9.51	9.51	10.9365	50	28.8	19.4435	3.10498	7.219007	62.34209	0.968438	19.4435	3.10498
April	3.08	5.81	5.81	6.6815	50	28.8	25.1985	2.395844	14.07249	60.94204	0.990686	25.1985	2.395844
May	4.68	3.33	3.33	3.8295	50	28.8	29.6505	2.03611	21.38288	66.42975	0.908846	29.6505	2.03611
June	5.2	1.82	1.82	2.093	50	28.8	31.907	1.892114	23.75876	64.86232	0.930809	31.907	1.892114
July	6.38	1.19	1.19	1.3685	50	28.8	33.8115	1.785537	29.15017	74.61656	0.809129	33.8115	1.785537
August	5.29	1.54	1.54	1.771	50	28.8	32.319	1.867993	24.16997	64.50225	0.936005	32.319	1.867993
September	3.57	2.88	2.88	3.312	50	28.8	29.058	2.077626	16.3113	52.29141	1.154577	29.058	2.077626
October	1.62	7.62	7.62	8.763	50	28.8	21.657	2.787628	7.401767	53.77829	1.122655	21.657	2.787628
November	0.46	15.69	15.69	18.0435	50	28.8	11.2165	5.382398	2.101736	79.65344	0.757964	11.2165	5.382398
December	0.1	15.74	15.74	18.101	50	28.8	10.799	5.590487	0.456899	76.00071	0.794393	10.799	5.590487
	32.83	90.92	90.92	104.558			273.872						

Lw(p) - (Pr-ET) + Pw

Ground Water Science Services, LLC	TEST PIT LOG	TP-1
Project: Hoodsport Thurman Property	Sheet: 1 of: 6	
Client: Gray & Osborne Engineering		
Logged By: Nadine Romero	Equipment: Backhoe	
Operator: Brian Sund; Libero Castro	Area: North Above Finch Creek	
LOG OF TEST PIT No: 1	Date: 5/3/00	Elevation: Aprox. 240'

PIT CONSTRUCTION	DEPTH FEET	SAMPLE No. DEPTH	CLASSIFICATION	COMMENTS
			Med stiff,redbrwn clayey SAND w/GRVL (GC)	
			Compact to Loose Sand & GRAVEL (GW)	Moderate K-permeability
	5		Stiff, Dense CLAY .5 Feet Thick w/ gravel (CL)	Low K-permeability Unit
			Stiff, red clayey GRAVEL (GC)	
			Compact, gray Fine SAND (SP)	Bedding Structures
			(No Moisture Observed	
	10		*In Test Pit)*	End of Test Pit @ 10.2 ft bgs
	15			
	20			

PHOTOGRAPH OF TEST PIT

Ground Water Science Services

Thurman Property
Hoodsport, Washington

Log of Test Pit
TP-1

FIGURE: 1

Ground Water Science Services, LLC		TEST PIT LOG	TP-2
Project:	Hoodsport Thurman Property	Sheet:	2 of: 6
Client:	Gray & Osborne Engineering		
Logged By:	Nadine Romero	Equipment:	Backhoe
Operator:	Brian Sund; Libero Castro	Area:	North Above Finch Creek
LOG OF TEST PIT No:	2	Date:	5/3/00 Elevation: Aprox. 240'

PIT CONSTRUCTION	DEPTH FEET	SAMPLE No. DEPTH	CLASSIFICATION	COMMENTS
			Med stiff,redbrwn clayey SAND w/GRVL (GC)	
			Loose to Compact, Red, Fine SAND (SP)	Moderate K-permeability
			Stiff, Dense CLAY .5 Feet Thick w/ gravel (CL)	Bedding Structures; Laminated
	5		HARD PAN (Till) – Dense sandy CLAY (GC)	LowK-permeability Unit
			(No Moisture Observed In Test Pit)	End of Test Pit @7.3 ft bgs
	10			
	15			
	20			

PHOTOGRAPH OF TEST PIT

Ground Water Science Services

Thurman Property
Hoodsport, Washington

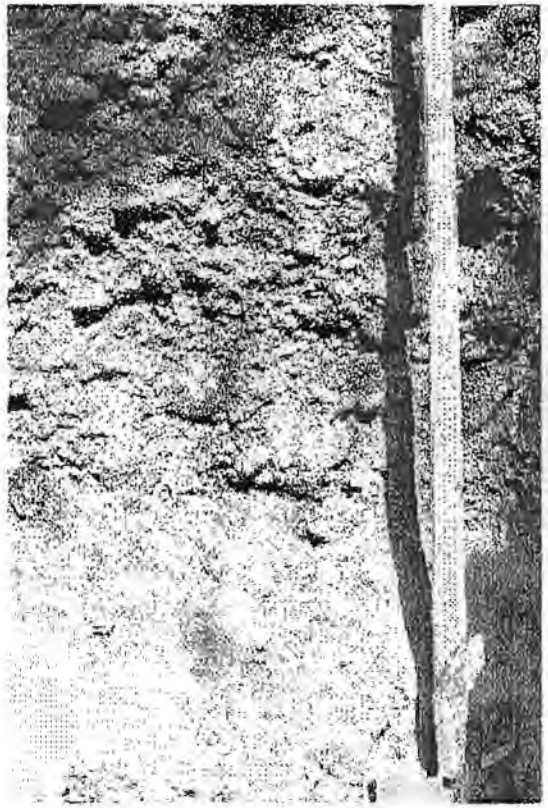
Log of Test Pit
TP-2

FIGURE: 2

Ground Water Science Services, LLC		TEST PIT LOG	TP-3
Project:	Hoodsport Thurman Property	Sheet:	3 of: 6
Client:	Gray & Osborne Engineering		
Logged By:	Nadine Romero	Equipment:	Backhoe
Operator:	Brian Sund; Libero Castro	Area:	North Above Finch Creek
LOG OF TEST PIT No:	3	Date:	5/3/00 Elevation: Aprox. 240'

PIT CONSTRUCTION	DEPTH FEET	SAMPLE No. DEPTH	CLASSIFICATION	COMMENTS
			Med stiff, red/brown clayey SAND w/GRVL (GC)	Moderate K-permeability
			Loose to Medium Dense, grey SAND (SP)	
				Moderate K-permeability
			Stiff, red sandy CLAY (CL)	Low K-permeability Unit
	5		Loose, red, coarse SAND and GRAVEL (SP-GW)	High K-permeability Unit
			w/ no clay matrix present	
			Med stiff, grey clayey fine SAND (SC-SP)	Low K-permeability Unit
	10		Loose, grey GRAVEL w/ SAND (GW-SW)	End of Test Pit @ 11.0 ft bgs
			(No Moisture Observed In Test Pit)	
	15			
	20			

PHOTOGRAPH OF TEST PIT



Thurman Property
Hoodsport, Washington

Log of Test Pit
TP-3

Ground Water Science Services

FIGURE: 3

Ground Water Science Services, LLC	TEST PIT LOG	TP-4

Project:	Hoodsport Thurman Property	Sheet:	4 of: 6
Client:	Gray & Osborne Engineering		
Logged By:	Nadine Romero	Equipment:	Backhoe
Operator:	Brian Sund; Libero Castro	Area:	North Above Finch Creek
LOG OF TEST PIT No:	4	Date:	5/3/00 Elevation: Aprox. 240'

PIT CONSTRUCTION	DEPTH FEET	SAMPLE No. DEPTH	CLASSIFICATION	COMMENTS
			Loose, GRAVEL, COBBLES w/ Sand (GW)	Very high K-permeability
				No Clay Observed
	5			
			(No Moisture Observed In Test Pit)	
	10			End of Test Pit @ 10.2 ft bgs
	15			
	20			

PHOTOGRAPH OF TEST PIT

Ground Water Science Services

Thurman Property
Hoodsport, Washington

Log of Test Pit
TP-4

FIGURE: 4

Ground Water Science Services, LLC		TEST PIT LOG	TP-5
Project:	Hoodsport Thurman Property	Sheet:	5 of: 6
Client:	Gray & Osborne Engineering		
Logged By:	Nadine Romero	Equipment:	Backhoe
Operator:	Brian Sund; Libero Castro	Area:	North Above Finch Creek
LOG OF TEST PIT No:	5	Date:	5/3/00 Elevation: Aprox. 240'

PIT CONSTRUCTION	DEPTH FEET	SAMPLE No. DEPTH	CLASSIFICATION	COMMENTS
			Med stiff,redbrwn clayey SAND w/GRVL (GC)	
				Med to low K
			Compact, red brwn, GRAVEL & SAND (GW)	Very Indurated, Med to low K
	5		Medium Dense, grey fine SAND w/ clay	"Lake Bed" type fine sand /clay
			seams (3" beds of alternating clay and fine sand)	End of Test Pit @8.0 ft bgs
	10		*(No Moisture Observed In Test Pit)*	
	15			
	20			

PHOTOGRAPH OF TEST PIT

Ground Water Science Services

Thurman Property
Hoodsport, Washington

Log of Test Pit
TP-5

FIGURE: 5

Ground Water Science Services, LLC	TEST PIT LOG	TP-6
Project: Hoodsport Thurman Property	Sheet: 6 of: 6	
Client: Gray & Osborne Engineering		
Logged By: Nadine Romero	Equipment: Backhoe	
Operator: Brian Sund; Libero Castro	Area: North Above Finch Creek	
LOG OF TEST PIT No: 6	Date: 5/3/00	Elevation: Aprox. 240'

PIT CONSTRUCTION	DEPTH FEET	SAMPLE No. DEPTH	CLASSIFICATION	COMMENTS
			Loose, red brown sandy GRAVEL w/ clay (GW)	Med K-permeability
			Loose, GRAVEL and SAND w/ no clay (GW)	High K-permeability
			w/ roots to 6 feet	
	5			
			Stiff, red stratfied CLAY (CL) mottling present	"Lake Bed" type fine sand /clay
			Hard Pan (Till Unit) (GC)	End of Test Pit @8.0 ft bgs
			(No Moisture Observed In Test Pit)	
	10			
	15			
	20			

PHOTOGRAPH OF TEST PIT

Thurman Property
Hoodsport, Washington

Log of Test Pit
TP-6

Ground Water Science Services

FIGURE: 6

Soil and Plant Laboratory, Inc.

P.O. Box 6566, Orange, California 92613-6566 • (714) 282-8777 • FAX (714) 282-8575
P.O. Box 153, Santa Clara, California 95052-0153 • (408) 727-0330 • FAX (408) 727-5125
P.O. Box 1648, Bellevue, Washington 98009-1648 • (425) 746-6665 • FAX (425) 562-9531
6555 Palomino Circle, West Linn, Oregon 97068 • (503) 557-4959 • FAX (503) 557-0713

GROUND WATER SCIENCE SERVICES
2401 Briston Ct., SW.
Olympia, WA 98502
Attention: Nadine Romero

SOIL APPRAISAL ANALYSIS
(A03)

Northwest Office
Lab Number 20412
Hoodsport, WA

Samples Taken: Samples Rec'd: 5/11/00

Sample #	Half Sat. %	pH	ECe	Org %	Gravel Coarse 5-12	Gravel Fine 2-5	Sand Very Coarse 1-2	Sand Coarse 0.5-1	Sand Med. to V. Fine 0.05-.5	Silt .002-.05	Clay 0-.002	USDA Soil Classification	Sample Description & Log Number
1	16	5.7	0.3	1.5	34.6	25.0	39.6	25.6	26.5	7.3	7.0	Loamy Sand Very Gravelly	TP-1 (0-1 ft) 00-A12687 32
													CEC = 14.1 meq/100g
													2-12Gravel = 59.6 %
													0.25-0.5 = 9.3 %
													0.1-0.25 = 8.4 %
													0.05-0.10 = 2.8 %
2	12	6.1	0.1	0.2	18.7	25.2	43.0	39.1	14.7	1.7	1.5	Sand Very Gravelly	TP-1 (3-4 ft) 00-A12688 32
													CEC = 20.9 meq/100g
													2-12Gravel = 43.9 %
													0.25-0.5 = 10.0 %
													0.1-0.25 = 4.5 %
													0.05-0.10 = 0.2 %
3	23	4.9	1.8	2.6	40.0	21.3	23.0	20.2	31.6	19.7	5.5	Sandy Loam Very Gravelly	TP-6 (0-1 ft) 00-A12689 32
													CEC = 27.0 meq/100g
													2-12Gravel = 61.3 %
													0.25-0.5 = 10.4 %
													0.1-0.25 = 13.4 %
													0.05-0.10 = 7.8

5/31/00

Half Saturation %=approximate field moisture capacity. Salinity = ECe(dS/m at 25 degree C). Gravel fraction expressed as percent by weight of oven-dried sample passing a 12mm (1/2 inch) sieve. Particle sizes in millimeters.

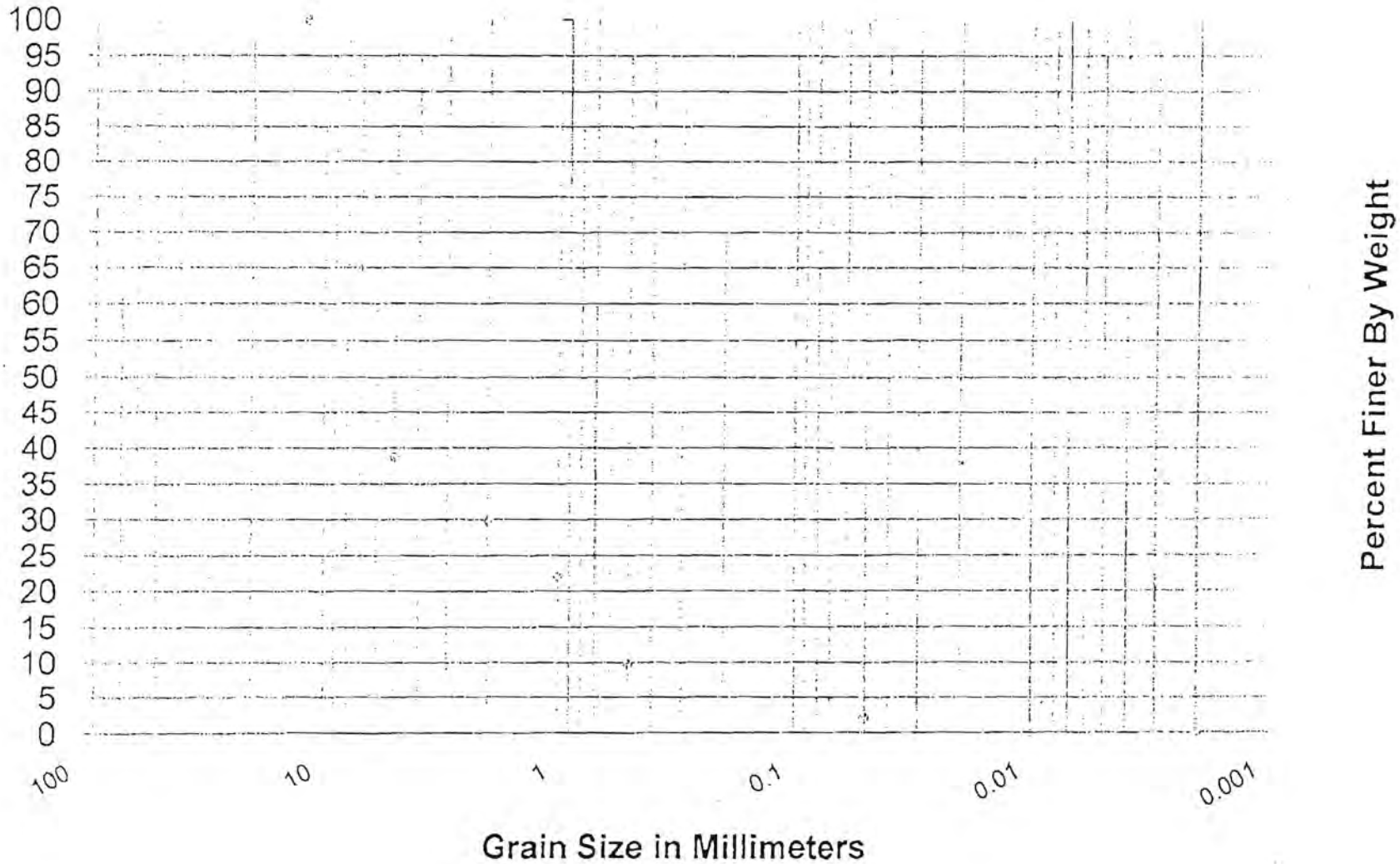
May 31, 2000

Above data are supplied without comments or recommendations.

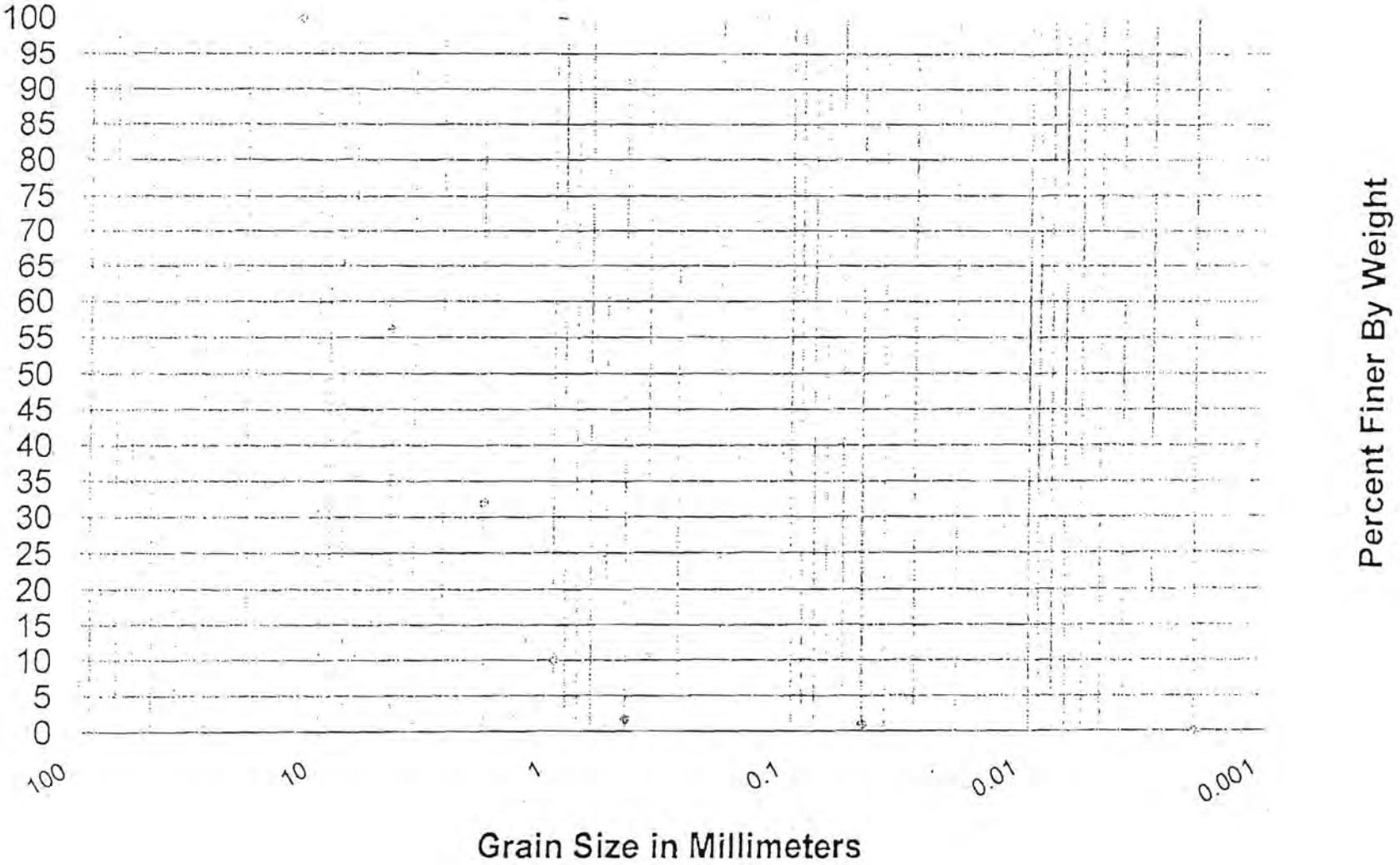
Dirk W. Munteau M.A. / dlb

Sincerely, Dirk W. Munteau M.A. DWM/dlb

Grain Size Distribution TP-6 (0-1 ft)



Grain Size Distribution TP-1 (3-4 ft)



Hazen Formula: K=C(D10)^2

C=40 to 80 for very fine sand, poorly sorted or w/ appreciable fines
D10 = .02 effective grain size

C	D10(cm)	D10^2	K cm/sec	ft/day =1 cm/sec	K ft/day
40	0.002	0.000004	0.00016	2835	0.4536
40	0.05	0.0025	0.1	2835	283.5
40	0.01	0.0001	0.004	2835	11.34
40	0.01	0.0001	0.004	2835	11.34
40	0.02	0.0004	0.016	2835	45.36

I = Volume of Waste Water Disposed On-site (in/yr)

gall/day	area (ft2)	ft3/gall	inches/ft	days/yr	I (in/yr)
10000	179200	0.13369	12	365	32.67642

Acreage (d)= 1.33
ft2= 57934.8

Nr = Nitrate Concentration in Wastewater
Nr = [I (Nw)(1-d) + (R)(Nb)] / (I + R)

Waste Water		Denitrification		Recharge				w/o dilution
I (in/yr)	Nw (mg/l)	d	1-d	R (in/hr)	Nb(mg/l)	R*Nb	I+R	Nr (mg/l)
32.676423	2	0.1	0.9	6	0.025	0.15	38.67642	1.524638443
32.676423	5	0.1	0.9	6	0.025	0.15	38.67642	3.805778611

N ave = ((Qgw * Nb) + (Qww * Nr))/(Qgw +Qww)

Transmissivity					Discharge in GW			Discharge in WW		
K (ft/day)	b (ft)	T	dh/dl	width	Qgw	Nb (mg/l)	Nr (mg/l)	Qww(gpd)	Qgw+Qww	N ave (mg/l)
50	8	400	0.033	160	2112	0.025	1.524638	100	2212	0.09279559
50	8	400	0.033	160	2112	0.025	3.805779	100	2212	0.19592128

Maximum Nitrate Loading on Ground Water

N max increase = (10- Nb)(10%)

Nb (mg/l)	10-Nb	(10- Nb)(10%) (mg/l)
0.025	9.975	0.9975

Max Nitrate Loading (mg/l)
Nb+ Nmax
1.0225

	Porosity	Depth		Area	Possible Storage (FT3) cu ft.	gall/ft3	gall
Calculations:	0.2		6	25600	30720	7.48	229785.6
	0.2		6	52600	63120	7.48	472137.6
2.34 Acres	0.2		6	102400	122880	7.48	919142.4
4.109 Acres	0.2		6	179200	215040	7.48	1608499

GPM	GPD 24 HRS
10	14400

HOMES	GPD	Total GPD	GPW	GPM	GPY
28	400	10000	70000	280000	3360000

APPLICATION

	inch/day	ft/day	area ft2	ft3	gpd
2.34 acres	1	0.083333	102400	8533.333	63829.33
	0.5	0.041667	102400	4266.667	31914.67
1.17 acres	1	0.083333	51200	4266.667	31914.67
	0.5	0.041667	51200	2133.333	15957.33
4.109 acres	0.5	0.041667	179200	7466.667	55850.67
	0.25	0.020833	179200	3733.333	27925.33
	0.1	0.008333	179200	1493.333	11170.13

	volume ft3	porosity	storage ft3	storage gall
each grid 40x40x6	9600	0.2	1920	14361.6

SOUND ANALYTICAL SERVICES, INC.

Client Name	Groundwater Science Services
Project Name	HOODSPORT STREAM
Date Received	05-10-00

General Chemistry Parameters

Client Sample ID			S-1		
Lab ID			89560-01		
Parameter	Method	Date Analyzed	Units	Result	PQL
Alkalinity (as CaCO3)	EPA 310.1	05-15-00	mg/L	16	5
Ammonia Nitrogen	EPA 350.1	05-15-00	mg/L	ND	0.04
Bicarbonate	SM 2320	05-15-00	mg/L	16	NC
BOD (5-day)	EPA 405.1	05-15-00	mg/L	ND	2
Carbonate	SM 2320	05-15-00	mg/L	0	NC
Color	EPA 110.2	05-10-00	color	ND	5
Conductivity	EPA 120.1	05-11-00	umhos/cm	41	10
Cyanide	EPA 335.3	05-18-00	mg/L	ND	0.05
Fecal Coliform, MF	SM 9222D	05-10-00	ECFU/100 ml	880	N/A
Hardness	EPA 130.2	05-15-00	mg/L	23	5
Total Coliform, MF	SM 9222B	05-10-00	CFU/100	150	N/A
Total Dissolved Solids	EPA 160.1	05-11-00	mg/L	26	10
Total Kjeldahl Nitrogen	EPA 351.2	05-19-00	mg/L	ND	0.5
Total Phosphorus	EPA 365.1	05-26-00	mg/L	0.02	0.01
Turbidity	EPA 180.1	05-10-00	NTU	0.5	0.2

SOUND ANALYTICAL SERVICES, INC.

Client Name	Groundwater Science Services
Project Name	HOODSPORT STREAM-2ND ANALYSIS
Date Received	05-30-00

General Chemistry Parameters

Client Sample ID	STREAM-2
Lab ID	90001-01

Parameter	Method	Date Analyzed	Units	Result	PQL
Fecal Coliform, MF	SM 9222D	05-30-00	CFU/100 mL	0	N/A
Total Coliform, MF	SM 92228	05-30-00	CFU/100 mL	17	N/A

SOUND ANALYTICAL SERVICES, INC.

Client Name	Groundwater Science Services
Client ID:	S-1
Lab ID:	89560-01
Date Received:	5/10/00
Date Prepared:	5/10/00
Date Analyzed:	5/10/00
% Solids	-
Dilution Factor	1

Anions by USEPA Method 300A

Analyte	Result (mg/L)	PQL	MDL	Flags
Fluoride	ND	0.06	0.02	
Chloride	0.91	0.3	0.1	
Nitrite	ND	0.03	0.01	
Nitrate	0.25	0.03	0.01	
Sulfate	0.91	0.3	0.1	

SOUND ANALYTICAL SERVICES, INC.

Client Name	Groundwater Science Services
Client ID:	S-1
Lab ID:	89560-01
Date Received:	5/10/00
Date Prepared:	5/15/00
Date Analyzed:	5/15/00
Dilution Factor	1

Metals by ICP - USEPA Method 6010

Analyte	Result (mg/L)	PQL	Flags
Barium	ND	0.005	
Beryllium	ND	0.002	
Chromium	ND	0.01	
Copper	ND	0.02	
Iron	ND	0.1	
Manganese	ND	0.015	
Nickel	ND	0.04	
Sodium	1.5	1	
Zinc	ND	0.02	

SOUND ANALYTICAL SERVICES, INC.

Client Name	Groundwater Science Services
Client ID:	S-1
Lab ID:	89560-01
Date Received:	5/10/00
Date Prepared:	5/15/00
Date Analyzed:	5/15/00
Dilution Factor	1

Metals by ICP-MS - USEPA Method 6020

Analyte	Result (mg/L)	PQL	Flags
Arsenic	ND	0.001	
Antimony	ND	0.003	
Cadmium	ND	0.0005	
Lead	ND	0.0005	
Selenium	ND	0.003	
Silver	ND	0.0005	
Thallium	ND	0.0005	

SOUND ANALYTICAL SERVICES, INC.

Client Name	Groundwater Science Services
Client ID:	S-1
Lab ID:	89560-01
Date Received:	5/10/00
Date Prepared:	5/11/00
Date Analyzed:	5/11/00
Dilution Factor	1

Mercury by CVAA - USEPA Method 7470

Analyte	Result (mg/L)	PQL	Flags
Mercury	ND	0.0002	

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: Batch QC
Lab ID: 89607-01
QC Batch Number: 826-45

Method Blank

Parameter	Result (mg/L)	PQL
Total Dissolved Solids	ND	10

Duplicate

Parameter	Sample Result (mg/L)	Duplicate Result (mg/L)	RPD (%)	Flag
Total Dissolved Solids	251	274	8.8	

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: S-1
Lab ID: 89560-01
QC Batch Number: 697-67

Method Blank

Parameter	Result (mg/L)	PQL
Alkalinity (as CaCO3)	ND	3

Duplicate

Parameter	Sample Result (mg/L)	Duplicate Result (mg/L)	RPD (%)	Flag
Alkalinity (as CaCO3)	16.4	16.4	0.0	

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: N/A
Lab ID: N/A
QC Batch Number: 534

Method Blank

Parameter	Result (mg/L)	PQL
BOD (5-day)	0.0	N/A

QC Check Standard

Parameter	Result (mg/L)	Mean Value (mg/L)	%D
Glucose	195	198	1.5

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: Batch QC
Lab ID: 89555-10
QC Batch Number: R1094

Method Blank

Parameter	Result (mg/L)	PQL
Total Kjeldahl Nitrogen	ND	0.5

Duplicate

Parameter	Sample Result (mg/L)	Duplicate Result (mg/L)	RPD (%)	Flag
Total Kjeldahl Nitrogen	5.7	5.4	5.4	

Matrix Spike

Parameter	Sample Result (mg/L)	Matrix Spike Result (mg/L)	Spike Amount (mg/L)	Recovery (%)	Flag
Total Kjeldahl Nitrogen	5.7	8.4	2.5	110	

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: Batch QC
Lab ID: 89555-10
QC Batch Number: R1095

Method Blank

Parameter	Result (mg/L)	PQL
Total Phosphorus	ND	0.01

Duplicate

Parameter	Sample Result (mg/L)	Duplicate Result (mg/L)	RPD (%)	Flag
Total Phosphorus	4.5	4.0	12	

Matrix Spike

Parameter	Sample Result (mg/L)	Matrix Spike Result (mg/L)	Spike Amount (mg/L)	Recovery (%)	Flag
Total Phosphorus	4.5	6.2	2.0	85	

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: Batch QC
Lab ID: 89548-05
QC Batch Number: 875

Method Blank

Parameter	Result (mg/L)	PQL
Cyanide	ND	0.05

Duplicate

Parameter	Sample Result (mg/L)	Duplicate Result (mg/L)	RPD (%)	Flag
Cyanide	ND	ND	NC	

Matrix Spike

Parameter	Sample Result (mg/L)	Matrix Spike Result (mg/L)	Spike Amount (mg/L)	Recovery (%)	Flag
Cyanide	ND	0.07	0.1	70	X7

Blank Spike

Parameter	Blank Spike Result (mg/L)	Spike Amount (mg/L)	Recovery (%)	Flag
Cyanide	ND	0.1	0.1	100

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: S-1
Lab ID: 89560-01
QC Batch Number: 709-133

Method Blank

Parameter	Result (NTU)	PQL
Turbidity	ND	0.2

Duplicate

Parameter	Sample Result (NTU)	Duplicate Result (NTU)	RPD (%)	Flag
Turbidity	0.45	0.46	2.2	

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: S-1
Lab ID: 89560-01
QC Batch Number: 705-50

Method Blank

Parameter	Result (color)	PQL
Color	ND	5

Duplicate

Parameter	Sample Result (color)	Duplicate Result (color)	RPD (%)	Flag
Color	ND	ND	NC	

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: S-1
Lab ID: 89560-01
QC Batch Number: 237

Method Blank

Parameter	Result (mg/L)	PQL
Hardness	ND	2

Duplicate

Parameter	Sample Result (mg/L)	Duplicate Result (mg/L)	RPD (%)	Flag
Hardness	22.5	22.5	0.0	

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: S-1
Lab ID: 89560-01
QC Batch Number: R1093

Method Blank

Parameter	Result (mg/L)	PQL
Ammonia Nitrogen	ND	0.04

Duplicate

Parameter	Sample Result (mg/L)	Duplicate Result (mg/L)	RPD (%)	Flag
Ammonia Nitrogen	ND	ND	NC	

Matrix Spike

Parameter	Sample Result (mg/L)	Matrix Spike Result (mg/L)	Spike Amount (mg/L)	Recovery (%)	Flag
Ammonia Nitrogen	ND	1.1	1.0	110	

SOUND ANALYTICAL SERVICES, INC.

QUALITY CONTROL REPORT

Client Sample ID: Batch QC
Lab ID: 89555-09
QC Batch Number: 775-45

Method Blank

Parameter	Result (umhos/cm)	PQL
Conductivity	ND	10

Duplicate

Parameter	Sample Result (umhos/cm)	Duplicate Result (umhos/cm)	RPD (%)	Flag
Conductivity	827	826	0.1	

SOUND ANALYTICAL SERVICES, INC.

Lab ID:	Method Blank - 1304
Date Received:	-
Date Prepared:	5/10/00
Date Analyzed:	5/10/00
% Solids	-
Dilution Factor	1

Anions by USEPA Method 300A

Analyte	Result (mg/L)	PQL	MDL	Flags
Fluoride	ND	0.06	0.02	
Chloride	ND	0.3	0.1	
Nitrite	ND	0.03	0.01	
Nitrate	ND	0.03	0.01	
Sulfate	ND	0.3	0.1	

SOUND ANALYTICAL SERVICES, INC.

Matrix Spike Report

Client Sample ID:	S-1
Lab ID:	89560-01
Date Prepared:	5/10/00
Date Analyzed:	5/10/00
QC Batch ID:	1304

Anions by USEPA Method 300A

Parameter Name	Sample Result (mg/L)	Spike Amount (mg/L)	MS Result (mg/L)	MS % Rec.	Flag
Fluoride	0	2	1.98	99	
Chloride	0.913	10	11.3	104	
Nitrite	0	1	1.01	101	
Nitrate	0.247	1	1.27	102	
Sulfate	0.911	10	11.4	105	

SOUND ANALYTICAL SERVICES, INC.

Duplicate Report

Client Sample ID:	S-1
Lab ID:	89560-01
Date Prepared:	5/10/00
Date Analyzed:	5/10/00
QC Batch ID:	1304

Anions by USEPA Method 300A

Parameter Name	Sample Result (mg/L)	Duplicate Result (mg/L)	RPD %	Flag
Fluoride	0	0	NC	
Chloride	0.913	0.928	-1.6	
Nitrite	0	0	NC	
Nitrate	0.247	0.247	0.0	
Sulfate	0.911	0.915	-0.4	

SOUND ANALYTICAL SERVICES, INC.

Lab ID:	Method Blank - T336
Date Received:	-
Date Prepared:	5/15/00
Date Analyzed:	5/15/00
Dilution Factor	1

Metals by ICP - USEPA Method 6010

Analyte	Result (mg/L)	PQL	Flags
Barium	ND	0.005	
Beryllium	ND	0.002	
Chromium	ND	0.01	
Copper	ND	0.02	
Iron	ND	0.1	
Manganese	ND	0.015	
Nickel	ND	0.04	
Sodium	ND	1	
Zinc	ND	0.02	

SOUND ANALYTICAL SERVICES, INC.

Blank Spike/Blank Spike Duplicate Report

Lab ID:	T336
Date Prepared:	5/15/00
Date Analyzed:	5/15/00
QC Batch ID:	T336

Metals by ICP - USEPA Method 6010

Compound Name	Blank Result (mg/L)	Spike Amount (mg/L)	BS Result (mg/L)	BS % Rec.	BSD Result (mg/L)	BSD % Rec.	RPD	Flag
Barium	0	4	3.53	88.2	3.49	87.3	-1	
Beryllium	0	0.1	0.0905	90.5	0.0894	89.4	-1.2	
Chromium	0	0.4	0.361	90.1	0.357	89.3	-0.89	
Copper	0	0.5	0.433	86.5	0.43	85.9	-0.7	
Iron	0	22	20	91	19.8	90.2	-0.88	
Manganese	0	1	0.891	89.1	0.88	88	-1.2	
Nickel	0	1	0.909	90.9	0.889	88.9	-2.2	
Zinc	0	1	0.909	90.9	0.9	90	-1	

SOUND ANALYTICAL SERVICES, INC.

Lab ID:	Method Blank - T336
Date Received:	-
Date Prepared:	5/15/00
Date Analyzed:	5/15/00
Dilution Factor	1

Metals by ICP-MS - USEPA Method 6020

Analyte	Result (mg/L)	PQL	Flags
Arsenic	ND	0.001	
Antimony	ND	0.003	
Cadmium	ND	0.0005	
Lead	ND	0.0005	
Selenium	ND	0.003	
Silver	ND	0.0005	
Thallium	ND	0.0005	

SOUND ANALYTICAL SERVICES, INC.

Blank Spike/Blank Spike Duplicate Report

Lab ID:	T336
Date Prepared:	5/15/00
Date Analyzed:	5/15/00
QC Batch ID:	T336

Metals by ICP-MS - USEPA Method 6020

Compound Name	Blank Result (mg/L)	Spike Amount (mg/L)	BS Result (mg/L)	BS % Rec.	BSD Result (mg/L)	BSD % Rec.	RPD	Flag
Arsenic	0	4	2.96	74.1	3.42	85.4	14	
Antimony	0	3	2.36	78.7	2.72	90.7	14	
Cadmium	0	0.1	0.0794	79.4	0.0921	92.1	15	
Lead	0	1	0.886	88.6	0.991	99.1	11	
Selenium	0	4	2.98	74.5	3.47	86.8	15	
Silver	0	0.6	0.025	90.2	40	104	14	
Thallium	0	4	3.32	82.9	3.8	94.9	13	

SOUND ANALYTICAL SERVICES, INC.

Lab ID:	Method Blank - T661
Date Received:	-
Date Prepared:	5/11/00
Date Analyzed:	5/11/00
Dilution Factor	1

Mercury by CVAA - USEPA Method 7470

Analyte	Result (mg/L)	PQL	Flags
Mercury	ND	0.0002	

SOUND ANALYTICAL SERVICES, INC.

Matrix Spike Report

Client Sample ID:	S-1
Lab ID:	89560-01
Date Prepared:	5/11/00
Date Analyzed:	5/11/00
QC Batch ID:	T661

Mercury by CVAA - USEPA Method 7470

Parameter Name	Sample Result (mg/L)	Spike Amount (mg/L)	MS Result (mg/L)	MS % Rec.	Flag
Mercury	0	0.002	0.00182	91	

SOUND ANALYTICAL SERVICES, INC.

Duplicate Report

Client Sample ID:	S-1
Lab ID:	89560-01
Date Prepared:	5/11/00
Date Analyzed:	5/11/00
QC Batch ID:	T661

Mercury by CVAA - USEPA Method 7470

Parameter Name	Sample Result (mg/L)	Duplicate Result (mg/L)	RPD %	Flag
Mercury	0	0	NC	

Sound Analytical Services, Inc.
ANALYTICAL & ENVIRONMENTAL CHEMISTS
4813 Pacific Hwy East • Tacoma, WA 98424
(253) 922-2310 • FAX (253) 922-5047
e-mail: info@saslab.com

DATA QUALIFIERS AND ABBREVIATIONS

B1: This analyte was detected in the associated method blank. The analyte concentration was determined not to be significantly higher than the associated method blank (less than ten times the concentration reported in the blank).

B2: This analyte was detected in the associated method blank. The analyte concentration in the sample was determined to be significantly higher than the method blank (greater than ten times the concentration reported in the blank).

C1: Second column confirmation was performed. The relative percent difference value (RPD) between the results on the two columns was evaluated and determined to be ≤ 40%.

C2: Second column confirmation was performed. The RPD between the results on the two columns was evaluated and determined to be > 40%. The higher result was reported unless anomalies were noted.

M: GC/MS confirmation was performed. The result derived from the original analysis was reported.

D: The reported result for this analyte was calculated based on a secondary dilution factor.

E: The concentration of this analyte exceeded the instrument calibration range and should be considered an estimated quantity.

J: The analyte was analyzed for and positively identified, but the associated numerical value is an estimated quantity.

MCL: Maximum Contaminant Level

MDL: Method Detection Limit

N: See analytical narrative.

ND: Not Detected

PQL: Practical Quantitation Limit

X1: Contaminant does not appear to be "typical" product. Elution pattern suggests it may be _____ .

X2: Contaminant does not appear to be "typical" product.

X3: Identification and quantitation of the analyte or surrogate was complicated by matrix interference.

X4: RPD for duplicates was outside advisory QC limits. The sample was re-analyzed with similar results. The sample matrix may be nonhomogeneous.

X4a: RPD for duplicates outside advisory QC limits due to analyte concentration near the method practical quantitation limit/detection limit.

X5: Matrix spike recovery was not determined due to the required dilution.

X6: Recovery and/or RPD values for matrix spike(/matrix spike duplicate) outside advisory QC limits. Sample was re-analyzed with similar results.

X7: Recovery and/or RPD values for matrix spike(/matrix spike duplicate) outside advisory QC limits. Matrix interference may be indicated based on acceptable blank spike recovery and/or RPD.

X7a: Recovery and/or RPD values for this spiked analyte outside advisory QC limits due to high concentration of the analyte in the original sample.

X8: Surrogate recovery was not determined due to the required dilution.

X9: Surrogate recovery outside advisory QC limits due to matrix interference.

Sound Analytical Services, Inc.
ANALYTICAL & ENVIRONMENTAL CHEMISTS
4813 Pacific Hwy East • Tacoma, WA 98424
(253) 922-2310 • FAX (253) 922-5047
e-mail: sainc1@uswest.net

12/2 9:12/15

SAS Lab No. 95560

TURNAROUND REQUEST (business days)
Standard (10 days) X
RUSH: 24 hrs ___ 48 hrs ___ 5 day ___

CHAIN OF CUSTODY/REQUEST FOR LABORATORY ANALYSIS

Client: Groundwater Science Svcs. (GWSS)

Project Name: Woodsport Stream

Contact: NADINE ROMERO

Phone No.: 360-754-3102

Fax No.: 360-705-6536

Email: nromero@nwnexus.com

Analyses Requested

Lab Use Only	Sample ID	Date	Time	Matrix	# of Containers	Chloride Interferences	BOD	Total Coliform Fecal Coliform	Alamona, TKN	Total Phosphate	Bicarb. Alkalinity Carb. Alk.
	S-1	5/9/06	10 a.m.	water	7	X	X	X	X	X	X

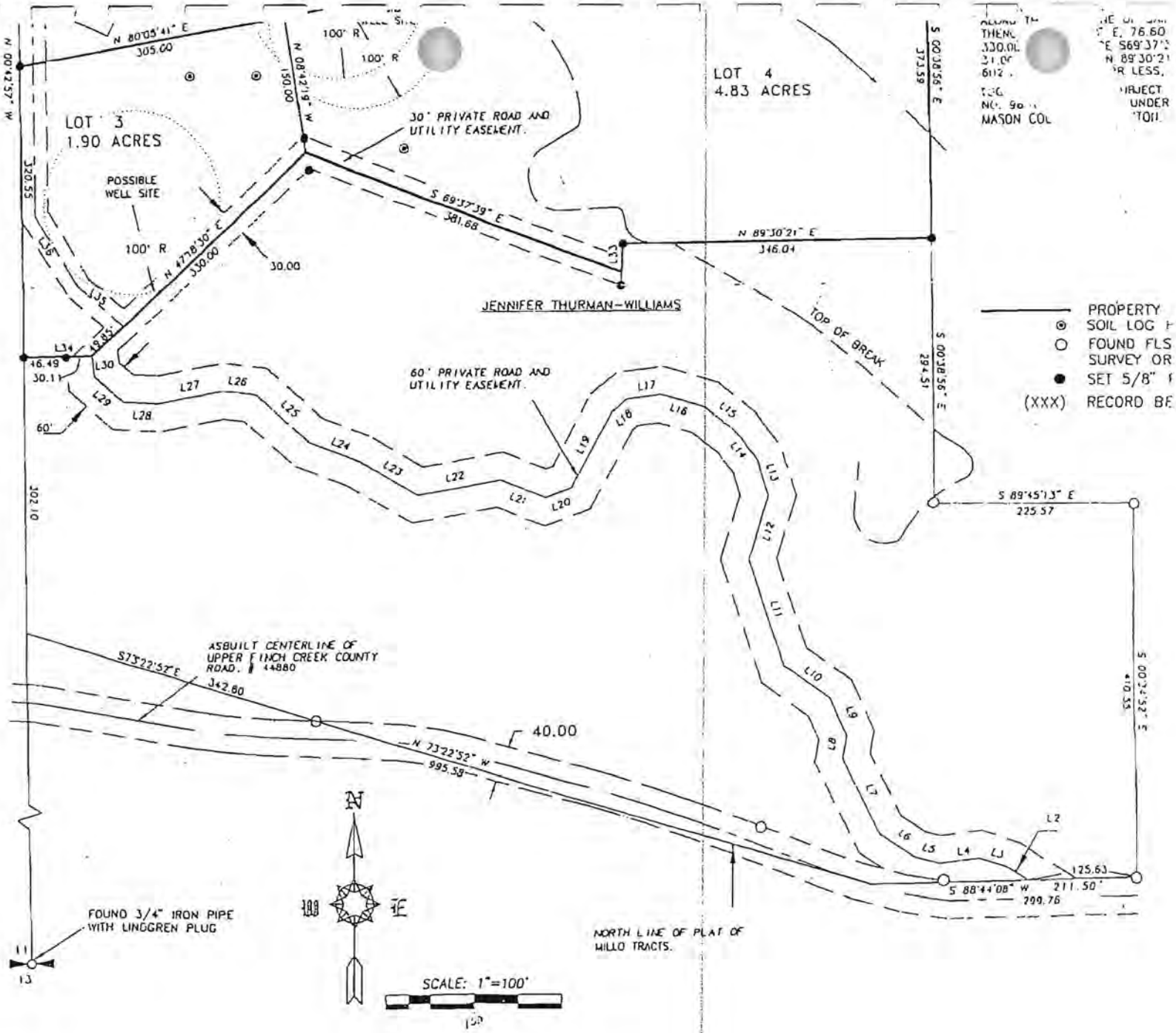
	Signature	Printed Name	Firm	Time/Date
Relinquished By:	[illegible]	Nadine Romero	GWSS	5/11/06
Received By	[illegible]	[illegible]	SAS	5/11/06
Relinquished By:				
Received By				
Relinquished By:				
Received By				

Special Instructions

1115 Am

[illegible] SW
Olympia, WA
98502

COC No


LOT 3
1.90 ACRES
POSSIBLE WELL SITE
100' R
LOT 4
4.83 ACRES
30' PRIVATE ROAD AND UTILITY EASEMENT.
60' PRIVATE ROAD AND UTILITY EASEMENT.
JENNIFER THURMAN-WILLIAMS
TOP OF BREAK
N 80°05'41" E
305.00
N 00°42'57" W
320.55
N 08°42'19" W
150.00
N 47°18'30" E
330.00
30.00
S 69°37'39" E
381.68
N 89°30'21" E
346.04
S 00°38'56" E
273.59
294.51
S 89°45'13" E
225.57
S 00°32'32" E
410.55
S 88°44'08" W
211.50
200.76
125.63
S 73°22'52" E
342.80
N 73°22'52" W
995.58
40.00
ASBUILT CENTERLINE OF UPPER FINCH CREEK COUNTY ROAD.
FOUND 3/4" IRON PIPE WITH LINDGREN PLUG
NORTH LINE OF PLAT OF HILLO TRACTS.
SCALE: 1"=100'
PROPERTY
SOIL LOG
FOUND FLS
SURVEY OR
SET 5/8"
(XXX) RECORD BE