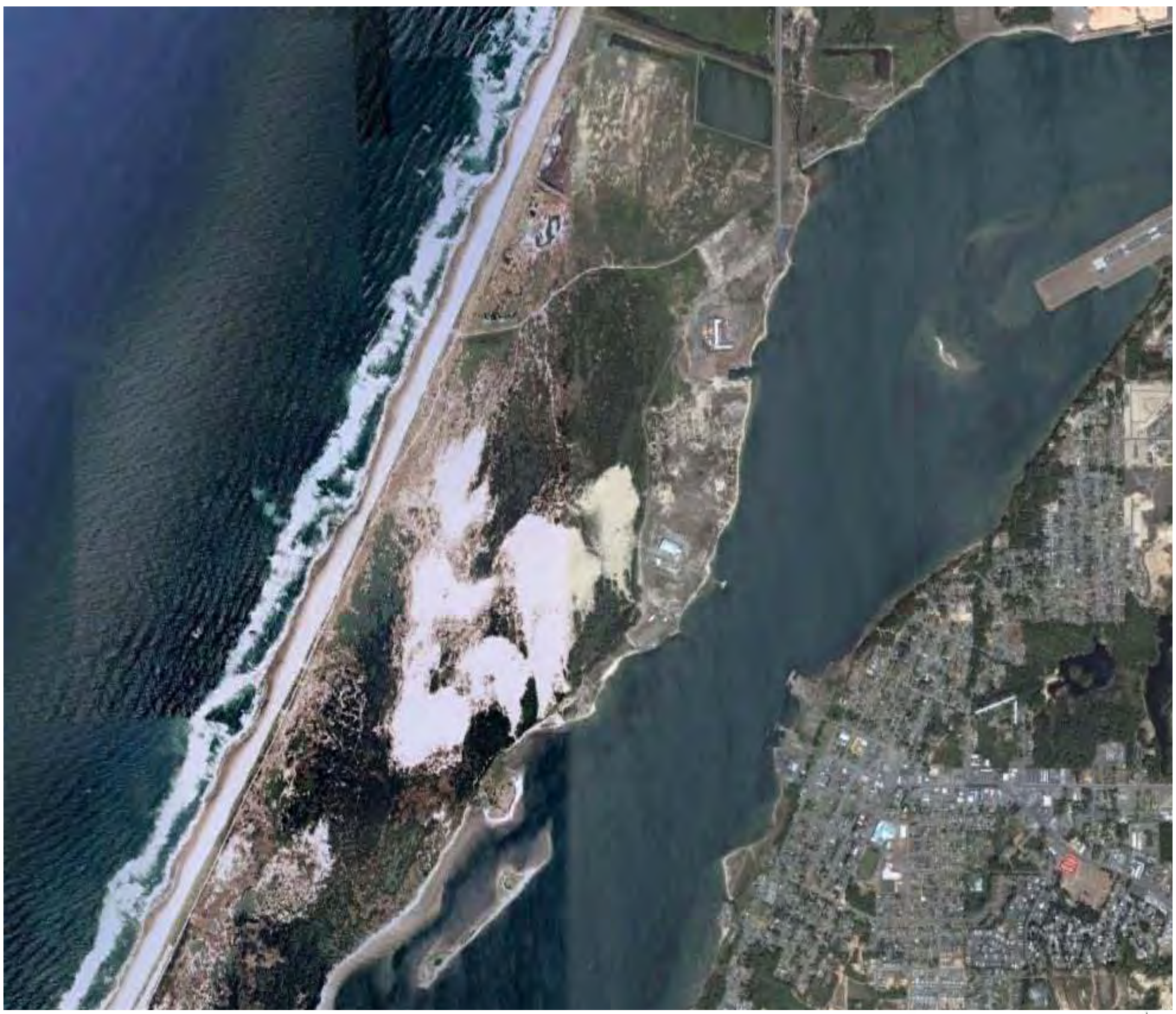
Feasibility Study:

North Spit Wastewater Treatment Plant

For

The City of Coos Bay



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Feasibility Study

North Spit Wastewater Treatment Plant

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1.0 Introduction

1.1 Project Description

1.1.1 Overview

The City of Coos Bay owns and operates two separate wastewater treatment facilities with each plant serving a portion of the community. Both treatment facilities are in need of upgrades and improvements to address age/condition-related deficiencies as well as water quality and capacity related deficiencies.

Presently, Wastewater Treatment Plant #2, on Cape Arago Highway, provides service to approximately 1/3 of the residents of Coos Bay plus service to the community of Charleston. The current peak flows are in the range of 5 million gallons per day (MGD) and it is expected that within the 20 year planning cycle, the peak flows will be in excess of 8.5 MGD.

The City has entered into a Mutual Agreement and Order (MAO) with the Oregon Department of Environmental Quality (DEQ) that includes milestones and tasks that the City must complete in order to remain in compliance.

The City is currently in the process of completing a pre-design report for improvements to the facilities located at Plant #2. Civil West Engineering Services, along with Century West Engineering and Esvelt Environmental Engineering, are currently progressing towards the completion of that report.

1.1.2 Purpose

During the pre-design process it was discovered that the treatment recommendations presented in the Facilities Plan were not adequate to comply with the ammonia limits of the discharge permit. As a result, additional real estate must be acquired or more expensive treatment methods must be used.

The City has expressed an interest in an evaluation the feasibility of treating the sewage on the north spit, where an existing lagoon and an existing ocean outfall are located. The lagoon was originally constructed in 1960 and served the Weyerhauser paper mill and was reconfigured in 1990. The site was recently reviewed as a possible dredge disposal location for the spoils for the marine terminal proposed Jordan Cove Energy (LNG) project.

The advantages of using the site include available property for current and future flows (including the possibility of accepting sewage which currently is treated by Plant #1), the potential to utilize the processes that have low operational requirements and the ocean outfall which permits a lower level of treatment prior to discharge as compared with discharge limitations in bay or estuary waters.

Challenges associated with the project include transmission of the sewage to the north spit site (2+ miles from WWTP #2 and across the bay), modification of the existing lagoon to meet current requirements, verification that the existing ocean outfall will meet the requirements and land use hurdles associated with the City of Coos Bay operating a treatment plant which is outside of their Urban Growth Boundary.

1.2 Project Study Area (North Spit Site)

The project study area includes the City of Coos Bay's wastewater treatment facility (Plant 2), a small section of the Empire Area of Coos Bay along Cape Arago Highway, the bay of Coos Bay from the Hollering Place to the North Spit anadromous facility, and on the North Spit, along the Trans Pacific Parkway to the Weyerhaeuser industrial waste treatment facility.

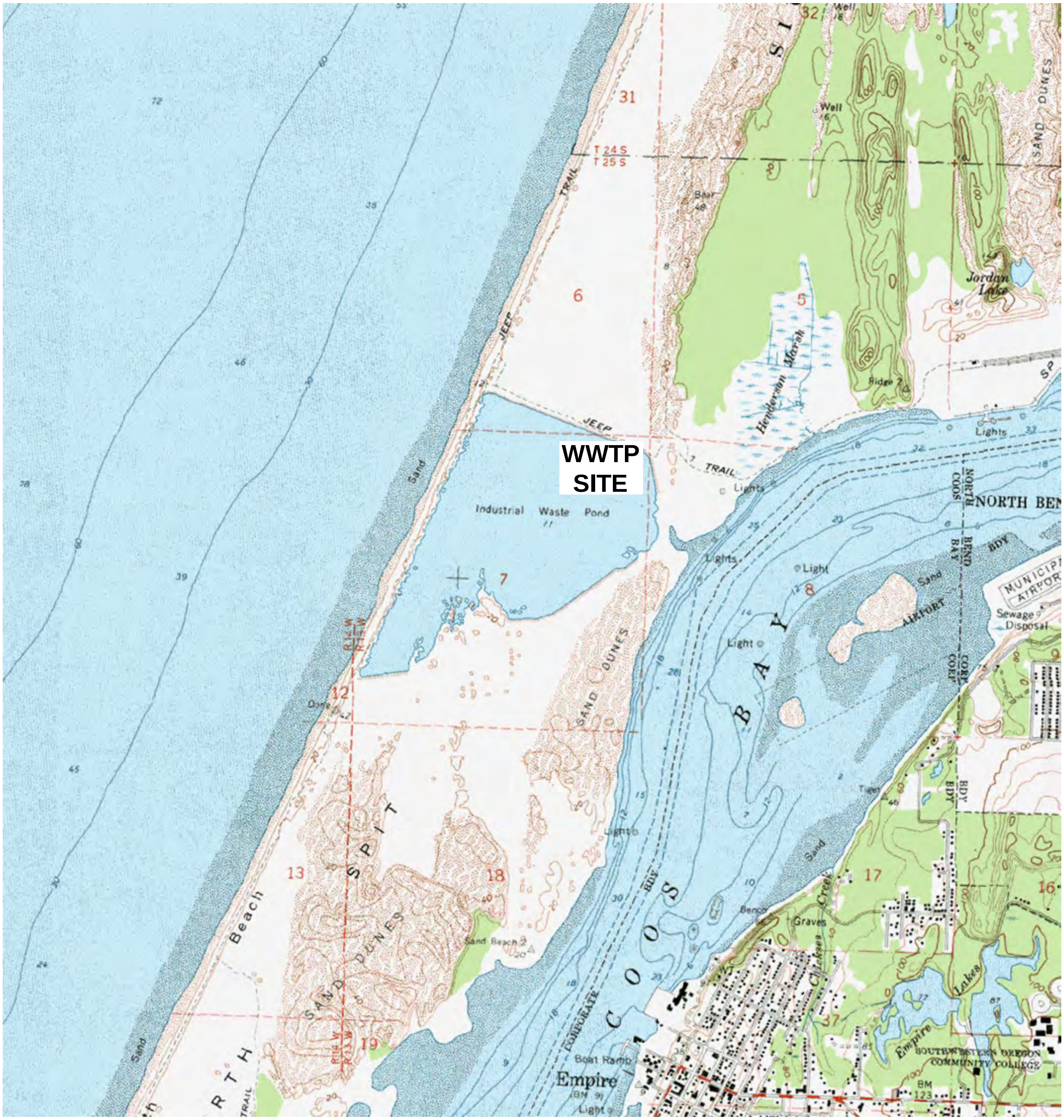
The location of the proposed wastewater treatment system is on Weyerhaeuser property on the North Spit of Coos Bay as shown in the Location Map, Figure 1.1. The area of interest is identified as "Industrial Waste Pond." Weyerhaeuser's use of the industrial waste pond shown in Figure 1.1 ceased in 1972 when the treatment system was reconfigured to include a 31 acre aerated stabilization basin (ASB), shown in Figure 1.2. The area outside the 31 acre lagoon (approximately 240 acres) is now a seasonally flooded wetland.

1.2.1 Site Topography

The topography of the study area is dominated by Coos Bay and the Pacific Ocean. The southern half of the project is located along the bay front in the Empire District. This area includes sand flats at the Plant 2 site and an upland terrace along Cape Arago Highway. On the north spit, the topography is more characteristic of an active dune with migrating and shifting sands forming temporary and semi-permanent sand dunes extending up as high as 100 feet. Along the Trans Pacific Parkway, the land is generally flat, where the roadway parallels the bay front. The area at the wastewater treatment plant site is more characteristic of a deflation plain subject to high ground water levels.

Flood Plain
The FEMA map indicating current 100 year flood elevations is presented in Figure 1.3. The proposed boundary of the V Zone: Coastal Flood Zone with Velocity Hazard (wave action), is also shown on the Figure. According to Jonathan Allen, Ph. D, Coastal Geomorphologist with the Oregon Department of Geology and Mineral Industries (DOGNAMI), the V-Zone shown in the figure is unofficial (it has not gone through a quality assurance process) but it generally shows the boundary of the new V-Zone designation.

An aerial map showing this boundary along with cross-section data provided by DOGNAMI is included in Appendix A "The data indicates that at the south end of the proposed treatment area, the beach has undergone significant erosion since 1998, with the shore having cut back ~25 m (80 ft) during the last decade. In contrast, the profiles to the north (mid-way along the pond area) have experienced much less erosion (~6 m (20 ft)), while the beach at the north end of the ponds have experienced no erosion.



SHN Consulting Engineers & Geologists, Inc.

City of Coos Bay
North Spit WWTP

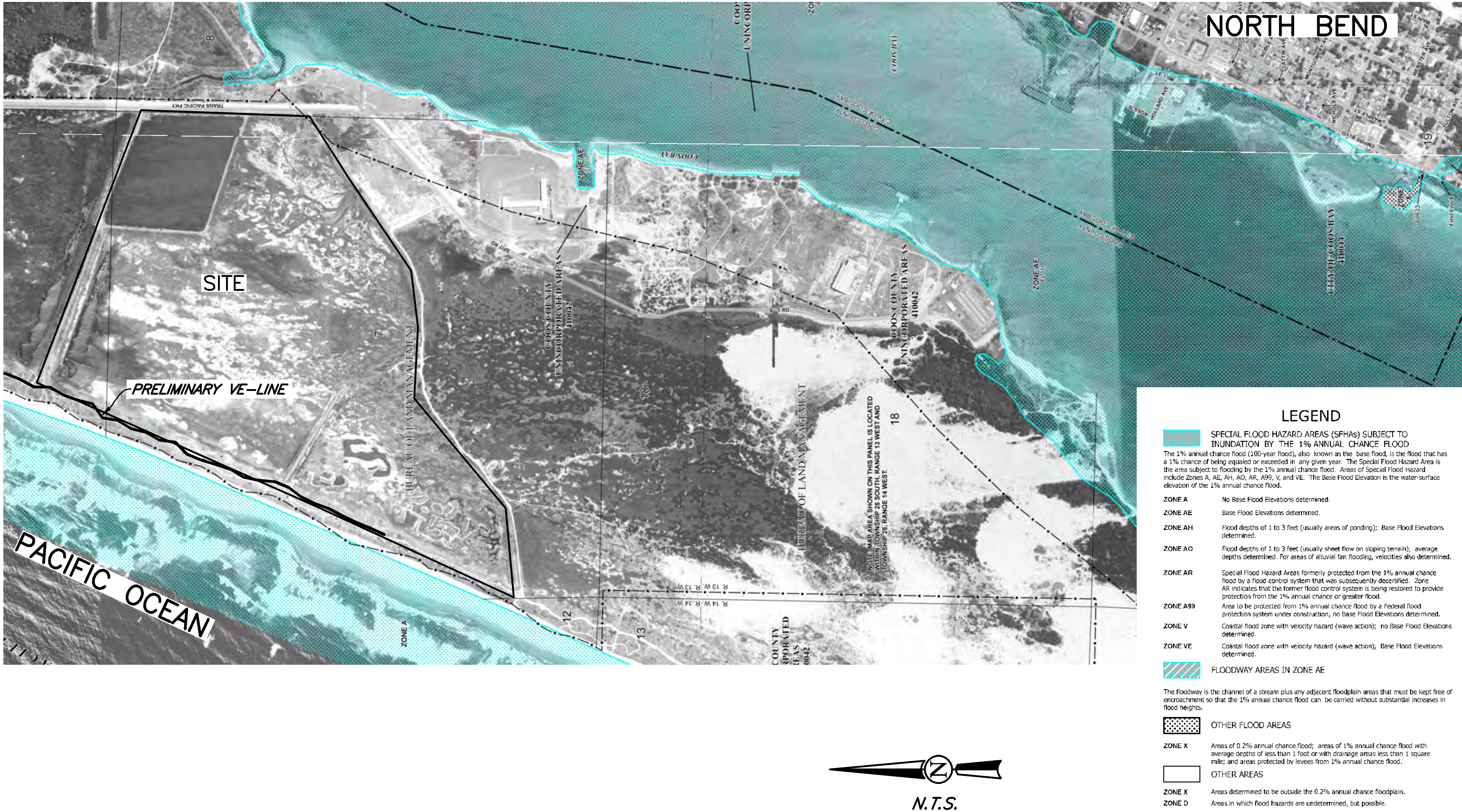
Location Map

SHN 609031.115

May 2010	609031-115-FIG-1-x-Location	Figure 1.1

N.T.S.

SHN Consulting Engineers & Geologists, Inc.	City of Coos Bay North Spit WWTP	Property of Interest SHN 609031.115
May 2010	609031-115-FIG-1-x-Location	Figure 1.2



LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAs) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Areas formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

SHN Consulting Engineers & Geologists, Inc.	City of Coos Bay Feasibility Study North Spit Coos Bay, Oregon	FEMA Map SHN 609031
	May 2010 / 609031-115-FIG-1-3-FEMA	Figure 1.3

\\Coosbay\projects\2009\609031-CBGeoGenSvc\115-N-Spit-WW-Trtmnt-Site\Figs , SAVED: 5/11/2010 11:48 AM SFRAZZINI, PLOTTED: 5/12/2010 5:06 PM , SHAWN FRAZZINI

1.2.2 Geologic Setting

The North Spit is located on the western edge of the Coos Bay Basin, a structurally downfolded and faulted basin with numerous minor anticlines (upfolded strata) and synclines (downfolded strata) (Beaulieu and Hughes, 1975). This basin was the site of repeated geologic deposits throughout the Tertiary period. The spit lies on the north projection of a north-south trending syncline that passes beneath South Slough. Bedrock below the sand dune deposits on the North Spit is Eocene age Coaledo formation. The Coaledo formation is a marine, sandstone unit with minor beds of siltstone (USACOE, 1981).

Geology within the project area is primarily Holocene –age fine to medium grained dune sands, averaging 100 feet or greater in depth. The chemical composition of the sands has been reported to have the presence of arsenic, chromium, iron, and manganese (HAR, 2000).

Hydrogeology

The eolian and upper marine sands are a prolific source of groundwater on the north spit. The area north of the existing basin have been designated a sensitive aquifer (Sweet-Edwards/EMCON, 1990). Precipitation, averaging over 65 inches per year, recharges the aquifer. Natural discharge from the aquifer is by underflow to Coos Bay and the Pacific Ocean and by evapotranspiration. Groundwater levels vary in direct response to incident precipitation variations and are locally influenced by tidal effects. During the winter months, groundwater levels can be at or above the ground surface.

The lagoon is underlain by 80 to 150 feet of Quaternary dune and marine sands that overlie older bedrock deposits (Robison, 1973). The bedrock (Tertiary marine siltstones, mudstones, and sandstones) has relatively low permeability and forms the base of the overlying freshwater aquifer. The bedrock crops out east of the North Spit and slopes westward toward the ocean. The base of the aquifer is about 160 feet below sea level at the project site (CH2MHill, 1984).

The Quaternary dune and marine sands forming the aquifer are relatively clean and uniform, except for minor layers of silt, clay, and organic matter. Horizontal permeability is generally greater than vertical permeability. Aquifer transmissivity ranges from less than 1×10^{-2} to 4×10^{-2} square feet per second (ft^2/sec) (Jones, 1992) and generally increase to the west as the aquifer thickens because of its sloping base. Near the lagoon, transmissivity is estimated at 4×10^{-2} ft/sec (Jones 1992). Aquifer recharge occurs by direct infiltration of precipitation.

The groundwater gradient has always been from the north wetland into the lagoon, even when the industrial discharge into the lagoons was active (CH2MHill, 1996b). Subsequently the intertidal area receives discharges from the lagoon, and to a much lesser extent (under 5%) the southern wetland receives discharges from the lagoon (Sweet-Edwards/Emcon, 1990). The aquifer is considered to be unconfined, and depth to water ranges from 5 to 50 feet MSL (HAR, 2000).

1.2.3 Historical Background

The industrial history of the site began in 1961 when the pulp mill was constructed by Menasha Wooden Ware Corporation (Menasha) as a neutral-sulfite, semi chemical process (NSSC) pulp mill (Tuppan 1995). As part of the pulp mill construction activities, a 270 acre wastewater oxidation lagoon, located 2 miles west of the pulp mill, was also constructed in 1960 by Menasha. Secondary treatment was accomplished by pumping from settling basins (located at the pulp mill) to the lagoon for aeration, evaporation, and infiltration (Tuppan 1995). Disposal of the treated effluent occurred via evaporation and infiltration until 1972, when a 4,800 foot long ocean outfall was constructed. The outfall was constructed to address a reduction in the infiltration capacity of the lagoon due to solids accumulation.

In 1990 the 30 acre aeration stabilization basin (ASB) was constructed in the northeast corner of the 270 acre lagoon to provide aeration prior to ocean discharge. In 1995 the pulp mill was shut down and converted to a recycling facility. This resulted in a reduction in effluent flows from 3.5 MGD to 2.5 MGD and a cessation of any contaminants of concern associated with the former pulp mill process.

In 2003 the company ceased operation of the container board facility and permanently closed the mill. Currently the only wastewater treated is landfill leachate which is pumped to settling basins and then to the stabilization basin before being discharged through the outfall. Some storm water enters the system, and Weyerhaeuser purchases water from nearby reservoirs at a rate of 300,000 to 500,000 gpd to maintain flow though the system to keep the diffusers from plugging and being inundated with sand (2004, NPDES Permit Evaluation Report). A summary of the history of the site is presented in Table 1.1.

Table 1.1
Site History

1960	270 acre wastewater treatment lagoon constructed
1961	Menasha operates NSSC pulp mill
1972	4,800 ft ocean outfall constructed, cease disposal via infiltration
1981	Weyerhaeuser purchases facility from Menasha
1990	Aerated Stabilization Basin (ASB) constructed in northeast corner of 270 acre lagoon
1995	Mill converted to 100 % recycle containerboard facility
1996	Discharge to lagoon ceased, wastewater effluent from ASB to ocean outfall
2003	Containerboard Facility shut down
Current	Only wastewater discharge, leachate from cell # 3 (construction debris landfill)

Contaminants of Concern

Contaminants of potential ecological concern (COPECs) were associated with the wastewater discharged to the lagoon from the former mill and containerboard facility. Based on available reports, and a scoping meeting CH2MHill held with Weyerhaeuser on February 13, 1995 (CH2MHill, 1996b), the following contaminants were identified as Contaminants of Potential Concern (COPECs):

- Arsenic
- Cadmium
- Chromium
- Copper
- Lead
- Iron

- Magnesium
- Cyanide
- Sulfides

The metals found in the lagoon were not part of the pulping process, but were associated with raw materials including recycled paperboard, virgin wood chips, and even industrial raw water supply, (Sweet-Edwards/Emcon, 1990)[1]. Cyanide and sulfides are associated with raw materials and the decomposition of sludge's in the lagoon (CH2MHill, 1996b), and were primarily associated with the former pulp mill process.

Weyerhaeuser has conducted sampling analysis for sludge from its mill discharges. These samples were analyzed for pesticides, herbicides, and semi-volatile compounds, all of which were below detection limits (CH2Mhill, 1996b). During file review, SHN was unable to confirm and or locate the sample results of the additional sludge sampling conducted by Weyerhaeuser.

1.3 Existing Characteristics

1.3.1 Sludge

SHN reviewed the following documents for information on sludge sampling of the 270 acre wastewater treatment lagoon and the stabilization basin (also called the ASB):

- "Wastewater Lagoon Ecological Risk Assessment" (1996, CH2M) An evaluation of the impact of leaving solids in place following cessation of discharge to the lagoon and
- "Weyerhaeuser Wastewater Treatment Systems Soils Sampling Plan", conducted in 2005 as part of the containerboard facility final closure.

1.3.2 Wastewater Lagoon

In 1996, disposal to the wastewater lagoons ceased, as effluent from the mill was pumped directly to the ASB, then to the ocean outfall. "The lagoons operated for a period of approximately 30 years, and biological solids produced during treatment, settled in the lagoons to a depth of less than a foot on average" (1996, CH2M). Direct estimates of toxicity to aquatic invertebrates conducted as part of the Risk Assessment showed no toxicity in the sediments of the wastewater lagoon and adjacent wetlands areas to the north and south. Sediment sampling determined that concentrations of the trace metals listed as contaminants of concern were below "conservative criteria protective of natural resources,[2] " and no unacceptable risks were identified. [3]

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[1] The 2000 HAR indicates that raw industrial supply water contained elevated levels of arsenic, chromium, iron and magnesium (2000 HAR).
[2] The conservative values were based on site-specific Critical Toxicity Values (CTVs) developed to be protective of plants, soil invertebrates and wildlife.
[3] Summary of Potential Risk Weyerhaeuser North Bend Mill Wastewater Lagoon" (1996, CH2M)

1.3.3 Stabilization Basin

The volume of the solids in the existing ASB pond is unknown. The stabilization basin was constructed in 1990 and received wastewater from the pulp mill until Weyerhaeuser completed a conversion of the facility to a containerboard facility with 100% recycle in 1995. The containerboard facility was closed in 2003 and discharge to the ponds was reduced to storm-water and leachate from a land-fill at the mill-site.

The stabilization basin, the ASB, was included in lagoon sludge sampling conducted by Weyerhaeuser in 2005. The results for the stabilization basin are summarized in Table 1.2. Concentrations were below federal industrial and residential standards for all metals listed as Contaminant of Potential Concern (COPEC). Concentrations of PCBs and volatile organic compounds were below induplotstrial standards and generally below residential standards, however levels of Dioxin, expressed as Toxic Equivalency Units (TEQ) were above EPA standards.

Table 1.2
Sludge Sampling of Stabilization Basin

	Weyerhaeuser 2005	CFR[2] Part 503	PRG[3] Standards	
	Stabilization Basin[1]	Pollutant Concentrations	Industrial	Residential
Constituent[4]	mg/kg	mg/kg	mg/kg	mg/kg
Arsenic	<20	41	2	0
Chromium	45	NA	450	210
Copper	216	1,500	41,000	3,100
Lead	130	300	800	400
Mercury	0.4	17	310	23
Nickel	33	420	20,000	1,600
Iron	16,500	NA	100,00	23,000
Selenium	<50	100	5,100	390
Zn	1420	2,800	100,000	23,000
Cyanide (free)	<2	NA	1.2×10^4	1.2×10^3
Dioxin[3]	0.309	NA	0.0384	0.004

1. Stabilization Basin referred to as ASB.
2. Includes Contaminant of Potential Ecological Concern (COPECs) included in Weyerhaeuser 2005 sampling.
3. Note: Cadmium (Cd) and Magnesium (Mg) pollutants in Table 3 of Part 503 were not analyzed
4. Toxic Equivalent Units (TEC) : Calculated value based on weighting of values for dioxin and dioxin like compounds

Contaminant concentrations from sludge sampling in the stabilization basin were also compared to maximum pollutant concentrations for the beneficial reuse of sewage sludge through land application as per 40 Code of Federal Registry (CFR) Part 503 Table 3 of Sec. 503.13.[4] Levels in the

[4] 40 CFR 503 Section 503.13 "If bulk sewage sludge is applied to agricultural land, forest, a public contact site, or a reclamation site The cumulative loading rate for each pollutant shall not exceed the cumulative pollutant loading rate for the pollutant in Table 2 of Sec. 503.13; or(ii) The concentration of each pollutant in the sewage sludge shall not exceed the concentration for the pollutant in Table 3."

sludge sampled were below the concentration limits in Part 503 Table 3 for land application for all constituents sampled.

1.3.4 Environmental Issues

The wetlands surrounding the existing stabilization basin were previously part of the 270-acre treatment lagoon. In 1993 Weyerhaeuser modified the treatment lagoon to minimize the environmental impact associated with the closure and enhance the beneficial use of the open water body for wildlife. The former lagoon area was allowed to cleanse itself through natural flushing processes and the accumulated sludge was retained in place after testing indicated that leachate quality were projected to fall within acceptable Oregon Groundwater Criteria (OAR 340.40).[5]

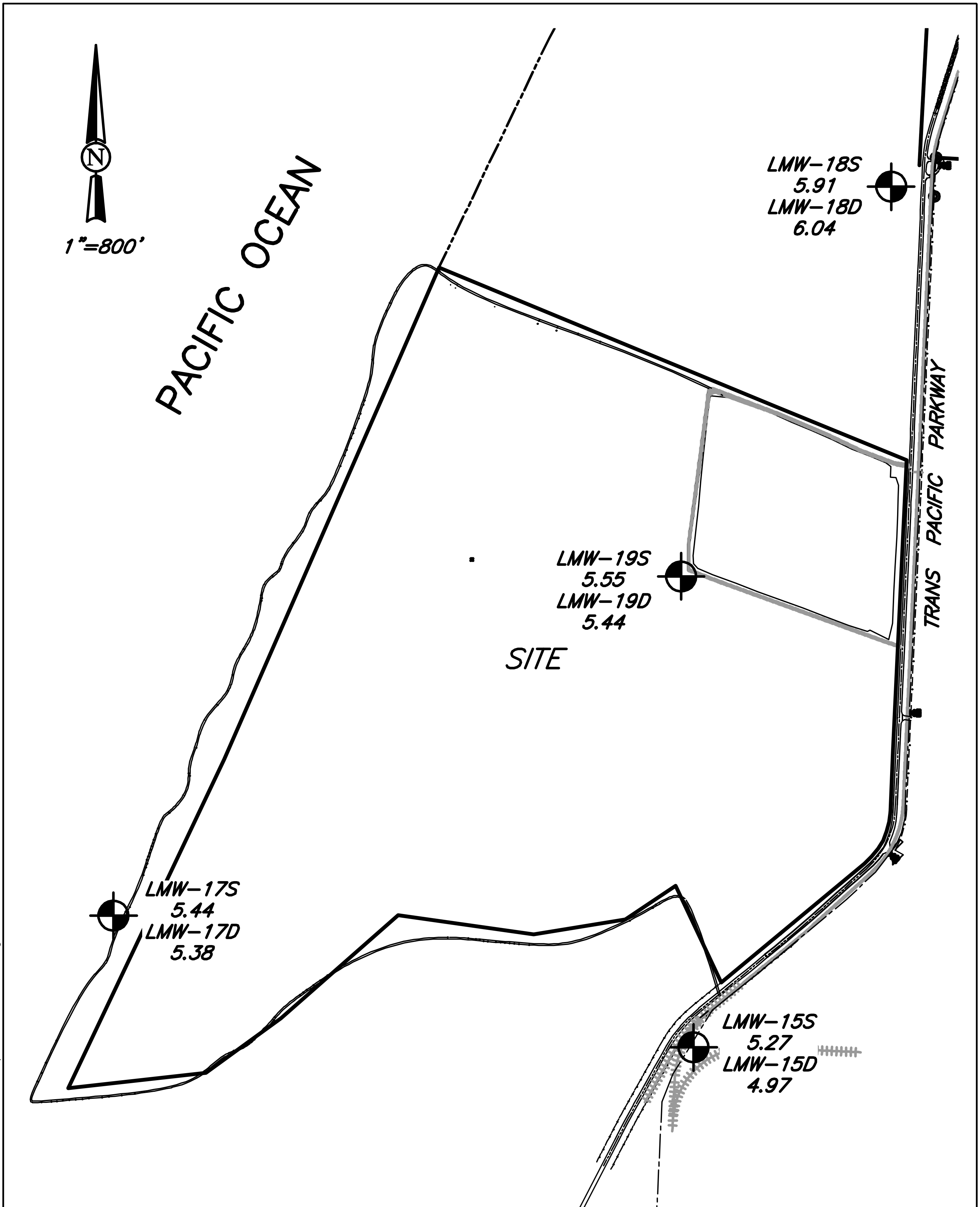
Groundwater Contamination Concerns

The former wastewater lagoon and stabilization basin is underlain by approximately 50 ft of dunal sand which in turn is underlain by marine derived sand. The total sand thickness ranges from 100-150 feet. Groundwater occurs in unconfined conditions in the sand with a water table ranging in elevation from 2-10 ft above MSL(5.5 -13.5 feet above MLLW).

Groundwater monitoring wells in place in the area of the former wastewater treatment lagoon and existing ASB are shown in Figure 1.4. Hydrographs from these monitoring wells from September 2000 through November 2005 included in Appendix B, indicate that the hydrological gradient is generally south and east towards the Bay.

Groundwater monitoring information from the "2005 Environmental Monitoring Report" (February 2006, Weyerhaeuser) is included in Table 1.3. Data from monitoring well MW 18 upgradient from the stabilization basin and former wastewater lagoon is used to define background levels. The background data shows high levels of iron, manganese, and color and seasonal fluctuations of monitored parameters. The data reported for 2005 indicates chloride, sulphate, and sodium levels in MW 17 and MW-19 above background levels. The report stated that monitoring has shown a steady improvement in levels of these constituents in MW-19S and 19D since the ASB ceased receiving wastewater in 2005.

[5] December 1993, CH2M, "Closure Plan for Wastewater Impoundments at Weyerhaeuser Paper Company North Bend, Oregon."



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SHN Consulting Engineers & Geologists, Inc.	City of Coos Bay Wastewater Treatment Plant Coos Bay, Oregon	Groundwater Wells
May 2010	609031-115-FIG-1-4-Groundwater	Figure 1.4

Table 1.3 Groundwater Quality Concentrations						
		Background		Additional Sampling		
		Shallow Wells	Deep Wells	Shallow Wells	Deep Wells	OAR Table
Constituent	Units	mg/L	mg/L	mg/L	mg/L	mg/L
pH		**6.2**	7.3	7.52	6.77	
Specific Conductance	uS/cm	136.0	307.0			
Temperature	°C	14.2	13.7	12.2	12.2	
Color	APHA Units	**27.0**	5.0			15
Chloride	mg/L	11.0	19.0	18.3	26.5	250
Sulfate	mg/L	<0.3	0.68	113.0		70
Sodium	mg/L	20.0	11.5	32.0	106	
Iron	mg/L	**16.8**	0.1	8.0	3.90	0.3
Manganese	mg/L	**0.08**	0.05	0.32	.01	0.05
Arsenic	mg/L	**0.05**	0.01	**0.02**	0.01	0.0539[3]
Chromium	mg/L	<0.01	<0.01	<0.01	0.02	0.05[3]

1. Weyerhaeuser North Bend Groundwater Quality Evaluation.
2. Concentration values shown in bold exceed OAR 340-40 Table 3 Guidance Criteria.
3. Interim standards OAR 340-40 Table 4A.
4. U.S. Primary MCL

Wetlands

The former wastewater treatment lagoon surrounding the ASB is no longer artificially flooded. Instead, a large portion of the area is seasonally flooded during wet-weather and the area has been allowed to revert back to wetland habitat. For all practical purposes, the majority of the 240 acre area could be considered jurisdictional wetlands, although migrating sands, over time, could change the ratio of upland and lowland. It is expected that the lagoon/wetlands area will continue to take on characteristics of a deflation plain with wetland areas similar to the areas further south and north. These areas, shown in Figure 1.5, are designated as emergent wetlands with areas of permanent and seasonal flooding.

The freshwater wetlands and ponds of the deflation plain support a diverse wildlife community and are some of the most productive habitats on the North Spit (Wilson-Jacobs 1983). Ranging from areas dominated by grasses and sedges to tall shrub thickets, the wetlands are used by many wildlife species to fulfill all or a portion of their habitat requirements.

The structurally diverse low shrub and thicket habitats contain the highest number of species in the wetland environment (USDA FS 1972). Muskrats, voles, rabbits, and other small mammals find food and shelter in the diverse vegetation and vertical structure of these areas. Predatory mammals (including shrews, mink, skunks, bobcats, foxes, and coyotes) forage on invertebrates, amphibians, birds, and small mammals. During the spring and summer, bats forage extensively on flying insects.

N
N.T.S.

P–PALUSTRINE

CLASS	RB–ROCK BOTTOM	UB–UNCONSOLIDATED BOTTOM	AB–AQUATIC BED	US–UNCONSOLIDATED SHORE	ML–MOSS–LICHEN	EM–EMERGENT	SS–SCRUB–SHRUB	FO–FORESTED
SUBCLASS	1 BEDROCK	1 COBBLE–GRAVEL	1 ALGAL	1 COBBLE–GRAVEL	1 MOSS	1 PERSISTENT	1 BROAD–LEAVED DECIDUOUS	1 BROAD–LEAVED DECIDUOUS
	2 RUBBLE	2 SAND	2 AQUATIC MOSS	2 SAND	2 LICHEN	2 NONPERSISTENT	2 NEEDLE–LEAVED DECIDUOUS	2 NEEDLE–LEAVED DECIDUOUS
		3 MUD	3 ROOTED VASCULAR	3 MUD		5 PHRAGMITES AUSTRALIS	3 BROAD–LEAVED EVERGREEN	3 BROAD–LEAVED EVERGREEN
		4 ORGANIC	4 FLOATING VASCULAR	4 ORGANIC			4 NEEDLE–LEAVED EVERGREEN	4 NEEDLE–LEAVED EVERGREEN
				5 VEGETATED			5 DEAD	5 DEAD
							6 DECIDUOUS	6 DECIDUOUS
							7 EVERGREEN	7 EVERGREEN

MODIFIERS

in order to more adequately descirbe the wetland and deepwater habitats, one or more of the water regime, water chemistry, soil, or special modifiers may be applied at the calss or lower leveil in thehierarchy. the farmed modifer may also be applied to the ecoogical system.

WATER REGIME			SPECIAL MODIFIERS	WATER CHEMISTRY			SOIL
NONTIDAL	**SALTWATER TIDAL**	**FRESHWATER TIDAL**		**COASTAL HALLNITY**	**INLAND SALINITY**	**PH MODIFIERS FOR ALL FRESH WATER**	
A TEMPORARILY FLOODED	L SUBTIDAL	S TEMPORAARILY FLOODED–TIDAL	B BEAVER	1 HYPERHALINE	7 HYPERSALINE	A ACID	G ORGAINC
B SATURATED	M IRREGULARLY EXPOSED	R SESONALLY FLOODED–TIDAL	D PARTLY DRAINED/DITCHED	2 EUHALINE	8 EUSALINE	T CIRCUMNEUTRAL	N MINERAL
C SEASONALLY FLOODED/ SATURATED	N REGULARLY FLOODED	T SEMIPERMANENTLY FLOODED–TIDAL	F FARMED	3 MIXOHALINE (BRACKISH)	9 MIXOLASINE	L ALKALINE	
F SEMIPERMANENTLY FLOODED	P IRREGULARY FLOODED	V PERMANENTLY FLOODED–TIDAL	H DIKED/IMPOUNDED	4 POLYHALINE	0 FRESH		
G INTERMITTENTLY EXPOSED			R ARTIFICIAL	5 MESOHALINE			
H PERMANENTLY FLOODED			S SPOIL	6 OLIGOHALINE			
J INTERMITTENTLY FLOODED			X EXCAVATED	7 FRESH			
K ARTIFICALLY FLOODED							

SHN Consulting Engineers & Geologists, Inc.

City of Coos Bay
Wastewater Treatment Plant
Coos Bay, Oregon

National Wetlands Inventory

May 2010 | 609031-115-FIG-1-5-Wetlands | Figure 1.5

Ponds provide areas of open water adjacent to the more heavily vegetated freshwater shrublands and thickets, and support a community of aquatic invertebrates, fish, and amphibians. Many of the species inhabiting the ponds are important food sources for other animals. Although the inland open water sites of the North Spit are not considered high quality nesting habitat for most species of waterfowl, they are used for foraging by a variety of migrating waterfowl during the spring, fall, and winter (Thornburgh 1991).

A combination of structurally complex habitat features and an abundant variety of available food sources support a variety of bird species. Waterfowl, shorebirds, passerines and raptors nest or forage in the freshwater wetlands, and migratory birds rest and feed there while traveling.

Following closure, the wastewater lagoon was the subject of an extensive monitoring and sampling plan. Field sampling of the lagoon /wetlands included birds, vegetation, sediments, surface water and pore water (June 1996, CH2M). A summary of field survey results for June 1996 is included as Table 1.4. A more recent survey and wetlands delineation was completed in December 1997 by Stuntzner Engineering and Forestry.

Table 1.4
Summary of Wetlands/Lagoon Field Surveys[1]

Common Name	Scientific Name	Habitat
Amphibians and Reptiles		
Pacific Treefrog	Hyla regilla	Emergent/scrub-shrub
Birds		
Red-winged Blackbird	Agelaius phoeniceus	Scrub-shrub
Mourning Dove	Zenaidura macroura	Scrub-shrub
Bonapartes Gull	Larus Philadelphia	Open Water
Great Blue Heron	Ardea Herodias	Mudflat/emergent
Swallow spp	Various	Open water (foraging)
Canada Goose eggs – predated	Branta Canadensis	Upland
Marsh Wren	Cistothorus palustris	Flyover
Turkey Vulture	Cathartes aura	Flyover
Cinnamon Teal	Anas cyanoptera	Open water
Killdeer	Charadrius vociferous	Scrub-shrub
American Goldfinch	Spinus tristis	Scrub-shrub
Mammals		
Deer spp	Various	Scrub-shrub

1. Conducted in June 1996 at Weyerhaeuser's Wastewater Lagoon.

Water chemical analysis for surface and pore waters in the lagoon/wetlands area indicate levels of chemicals of potential ecological concern above groundwater reference standards and CTVs for some metals.[6] However the Assessment (1996) concluded that "the surface waters do not produce chronic effects and any aquatic organisms inhabiting the lagoon were not affected."

[6] Conservative CTVs or Effect Range- Low concentrations (ERLs) developed in the Wastewater Lagoon Risk Assessment(1995, CH2M Hill).

Sediment samples collected in the intertidal and north intertidal (reference) sites confirmed that sediment concentrations of all COPECs discussed in the previous section were less than the Effect Range- Low concentrations (ERLs) or conservative CTVs referenced previously, and the chemistry on both areas was similar indicating the lack of influence on the intertidal area from the former lagoon (1997, CH2M).

1.3.5 Wetland Considerations

Based on conversations with Division of State Lands (DSL) and DEQ, use of the 31 acre ASB industrial treatment lagoon for municipal wastewater treatment should be considered acceptable. The ASB currently provides wastewater treatment under NPDES permit No. 96255. The facility is in active operation. Conversion to a regional treatment facility for the City of Coos Bay would be consistent with the current use. Low lying areas outside of the ASB, however, have been delineated as wetland habitat and any disturbances in these areas would require fill removal permits. Considering the value of coastal wetlands in the vicinity, fill or removal occurring in the wetland areas would likely require mitigation if avoidance and minimization cannot first be accomplished. Several options considered in this feasibility report have potential impacts to wetlands. In these alternatives, estimates of mitigation costs have been considered based on discussions with consultants who have worked on similar projects in the area.

The initial concept for a North Spit treatment facility for Coos Bay considered the use of constructed wetlands for enhanced treatment. The thought was that constructed wetlands could be located in open water areas, potentially providing additional or enchanced habitat. Considering the potential for DSC to take over jurisdiction current wetland areas and the potential to impact these areas using constructed wetland technologies, this concept may not be practical. The DSL may not consider the constructed wetland as equivalent habitat and instead, could consider it an impact to wetland habitats. Since enhanced treatment is not essential prior to discharge to the ocean, consideration of a constructed wetlands for treatment is not considered practical at this time. Futher discussions with DSL should explore this issue.

2.0 Design Criteria

2.1 Regulatory Constraints

2.1.1 NPDES Permit

The current limits for discharge into the bay are dictated by the City's current NPDES permit (#100771) and are shown below in Table 2.1.1a. The current permit has an expiration date of 12/31/2007 but is still in effect today. It is likely that if the decision is made to continue to discharge into the bay that the parameters listed below will be modified to become more stringent.

Table 2.1 (Current Design Discharge Parameters)

Parameter	Average Effluent Monthly, mg/L	Average Effluent Weekly mg/L	Montly average, ppd	Weekly average, ppd	Daily maximum, lbs
May 1 - October 31:					
BOD - 5	20	30	340	510	670
TSS	20	30	340	510	670
November 1 - April 30:					
BOD - 5	30	45	510	760	1000
TSS	30	45	510	760	1000
Other parameters:					
Fecal Coliform Bacteria	Shall not exceed a monthly mean of 14 organisms per 100 mL. Not more than 10 percent of the samples shall exceed 43 organisms per 100 mL.				
pH (year round)	6.0 - 9.0				
BOD and TSS Removal Efficiency	Shall not be less than 85%				
Total Residual Chlorine	0.02 mg/L monthly 0.05 mg/L daily				
Ammonia-N (May 1 - October 31)	20 mg/l monthly 30 mg/l daily				
Excess Thermal Load (May 1 - October 31)	37 Million kcals/day				

Based on recent discussions with DEQ for a similar ocean outfall, we anticipate approximate effluent limitations as shown in Table 2.2

Table 2.2 (Expected Design Discharge Parameters)

Parameter	Average Effluent		Montly average, ppd	Weekly average, ppd	Daily maximum, ppd
	Monthly, mg/L	Weekly mg/L			
May 1 - October 31:					
BOD - 5	30	45	510	765	1005
TSS	50	75	850	1275	1675
November 1 - April 30:					
BOD - 5	30	45	510	765	1005
TSS	50	75	850	1275	1675
Other parameters:					
Entercocci Bacteria	Shall not exceed a monthly geometric mean of 35 organisms per 100 mL. No single sample shall exceed 104 organisms per 100 mL.				
Fecal Coliform Bacteria	Shall not exceed a monthly mean of 14 organisms per 100 mL. Not more than 10 percent of the samples shall exceed 43 organisms per 100 mL.				
pH (year round)	6.0 - 9.0				
BOD and TSS Removal Efficiency	Shall not be less than 85% monthly average for BOD and 65% monthly average for TSS.				
Chlorine Produced Oxidants	Shall not exceed a montly average concentration of 0.21 mg/l and a daily maximum concentration of 0.31 mg/l.				

2.2 Flows and Loadings

The flow and load projections are based on current flows and loads and anticipated community growth. The WWTP No. 2 service area, comprised of a portion of Coos Bay and the community of Charleston, is projected to have a population of 12,440 by 2027. Based on the Facilities Plan completed in 2007 by The Dyer Partnership, future sanitary flows are projected by applying the anticipated population growth rate to the current sanitary flows. Anticipated flows and loadings are shown in Table 2.3 below:

Table 2.3

Wastewater Characteristics Factor	2003	2027
Flows, mgd:		
Average Dry Weather Flow (ADWF)	0.9	1.0
Average Wet Weather Flow (AWWF)	1.6	1.9
Average Annual Flow (AAF)	1.2	1.4
Maximum Month Dry Weather Flow (MMDWF)	1.2	1.4
Maximum Month Wet Weather Flow (MMWWF)	2.1	2.4
Maximum Week Wet Weather Flow (MWWWF)	2.3	2.7
Peak Day Flow (PDF)	4.5	5.5
Peak Wet Weather Flow (PWWF)	7.0	8.6
Loads:		
BOD, ppd		
Average	1,800	2,200
Max month	2,200	2,700
Peak Day	3,800	4,700
TSS, ppd		
Average	2,000	2,500
Max month	2,500	4,000
Peak Day	4,300	6,800

3.0 Preliminary Treatment Alternatives

3.1 Headworks

3.1.1 Description

The headworks assembly will be a concrete structure with two depressed flow channels. The primary channel will include a mechanically cleaned fine screen, and the bypass channel will be elevated and fitted with a manual bar screen. Downstream of the screen channels will be a grit chamber.

3.1.2 Site Evaluation

The headworks can be located either on the north spit, or can be before the pump station at the existing treatment site.

3.1.2.1 **North Spit Headworks Option**

Option 1, on the north spit, enables operation and maintenance of the treatment system to be in one location, saving time and energy. The facility would be elevated above the subsequent biological treatment stage to allow for gravity flow throughout the remainder of the treatment process.

3.1.2.2 **Existing Site Headworks Option**

Option 2, at the existing treatment plant site, would require an additional pump station or locating the headworks below grade. An additional pump station would be very expensive (~$2M) while locating the headworks below grade causes concerns with potential flooding and possible confined work spaces. The benefit to locating the pump station at the existing treatment site is the ability to use a slightly more efficient pump since the solids will have been screened out. There's also an option of providing some primary treatment on the existing site (see section 3.2) which would require the headworks be located on the existing site.

3.1.3 Preferred Location

Based on the additional costs & concerns associated with locating the headworks on or near the existing site, the north spit option is preferred.

3.1.4 Cost

The cost associated with the headworks is independent of its location and is summarized in Table 3.1.4 below:

Table 3.1.4, Headworks Cost Estimate

Headworks					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Mobilization & Permits (15%)	ls	1	$144,600.00	$144,600.00
2	Manual Bar Screen	ea	1	$10,000.00	$10,000.00
3	Mechanical Auger Screen	ea	2	$300,000.00	$600,000.00
4	Concrete Headworks Structure	cy	80	$800.00	$64,000.00
5	Steel Pile Foundations	ls	4	$35,000.00	$140,000.00
6	Screenings Handling & Disposal Equipment	ls	1	$15,000.00	$15,000.00
7	Force Main Pipe, Fittings, Supports, Etc.	ls	1	$15,000.00	$15,000.00
8	Metal Railings & Decking	ls	1	$20,000.00	$20,000.00
9	Electrical, Lighting & Controls	ls	1	$100,000.00	$100,000.00
		Construction Total			**$1,108,600.00**
		Contingency (15%)			$166,290.00
		Engineering (18%)			$199,548.00
		Administrative costs (3%)			$33,258.00
		Total Project Costs			**$1,507,696.00**

3.2 Primary Treatment

As precluded to above, there is an option to include a level of primary treatment on the existing treatment plant site prior to conveyance to the north spit. Because the existing plant would remain in service during construction of new facilities, the area south of Fulton Avenue would be used for any new treatment facilities. The area available is bounded by the Bay to the west, Fulton Avenue to the north and delineated wetlands to the south and east. The available property to locate any new facilities is approximately 100’ wide by 300’ long.

There is also potential for locating primary treatment on the north spit, outside of the wetland boundary. Since the potential primary treatment site on the north spit is larger than the land available at the existing site, there is no reason other than pump efficiency to include pretreatment or primary treatment at the existing site since any pump efficiencies will be negated by having to pump twice.

3.2.1 Performance

For the purposes of this study, it is assumed that each of the above referenced treatment alternatives is capable of providing a 25% removal of BOD and a 40% removal of TSS. These reductions would reduce the required footprint for secondary treatment on the north spit.

3.2.2 Evaluation

Although primary treatment reduces the loading in the treatment stream, the benefits of doing so on the existing site are minimal when compared with additional costs. The same treatment can be done on the north spit without the required additional pump station costs.

4.0 Pumping Requirements

4.1 Force Main

Two options are being considered to convey the wastewater to the north spit site and are discussed below. The first being a single force main drilled beneath the bay to convey all of the flows. The second option also would require a new force main drilled under the bay but includes the use of an existing 10″ HDPE pipe which was installed in 2005.

On both options, certain design considerations must be met. In accordance with the DEQ publication "Oregon Standards for Design and Construction of Wastewater Pump Stations", the pump and forcemain system shall have a maximum fluid velocity of not more than 8 feet per second. The pumping system shall also maintain a minimum velocity in the force main of 2.0 f.p.s. after an initial flushing at a minimum of 3.5 f.p.s. during each pumping cycle.

4.1.1 Single Force Main Option

This option entails pumping the entire volume through a single forcemain to the north spit. The pipe would go from the existing plant site, across the bay to an area just south of the Anadromous Facility and from there along the Trans Pacific Parkway to the treatment site. See Figure 4.1.1 below. The section from the existing treatment site to the other side of the bay (~5000 l.f.) would be installed using Horizontal Directional Drilling methodology, and the remainder (~6900 l.f.) would be laid using the traditional open trench method.

In order to transmit 8.6 MGD at a velocity of no more than 8 feet per second, a minimum of a 17.5″ inside diameter (I.D.) pipe must be used. For this study, an 18″ Fusible Sch. 80 PVC or a 22″ DR 11 HDPE (I.D. = 17.84″) are being considered. Using larger pipes exacerbates the issues noted below.

The PVC pipe has a pressure rating of 220 psi and the HDPE has a pressure rating of 160 psi. The elevation difference between the proposed wetwell low water elevation and the outlet of the pipe at the top of the proposed headworks is approximately 40′. The headloss associated with pumping 8.6 MGD at 8 feet per second through either of these pipes is approximately 131′. Including static head and dynamic losses, the total pumping head of 170′, which equates to 73 psi, is well below the pressure rating of both pipe types. Total head when pumping the required minimum 2.0 feet per second (2.3 MGD) drops to 51′ (22 psi). This large variation in pumping head will necessitate multiple pumps equipped with a detailed SCADA system to operate the pumps at the various flows.

While this option is the least expensive to construct, it has multiple disadvantages during the dry weather months. During dry weather, the average low flow is approximately 0.75 MGD. To pump that small of a flow in a relatively large forcemain with a minimum velocity of 2.0 feet per second, the pumps will only operate for a few minutes at a time. Furthermore, based on the average low flow, the wastewater will be in the forcemain for over 5 hours. The detention time in the forcemain will most likely go up to over 8 hours during daily periods of low flow, i.e. overnight. Based on the DEQ pump station design guidelines mentioned above, any forcemain with anaerobic detention times in excess of 35 minutes needs to have hydrogen sulfide (H_2S) controls. H_2S can be controlled with a chlorine drip into the wetwell, although this will require an on-site chlorine generator or chlorine delivery to the pump station.

Civil West Engineering Services, Inc.

DWG BY: MDW
DATE: MAY 2010

FORCE MAIN OPTIONS

COOS BAY WWTP #2

FIGURE 4.1.1

4.1.2 Double Force Main Option

Anticipating future need, in 2005 the City installed a 10″ DR11 HDPE (8.72″ I.D.) pipe under the bay from the end of Newmark Avenue to the same outlet described above near the fish hatchery. Using this pipe to pump dry weather flows (up to 1.3 MGD), in addition to an 18″ for high flows, allows for more efficient pumps to be used and reduces the detention time from over 4 hours to just over 2 hours.

Based on head losses through the smaller pipe, this option would be most efficient with new 12″ I.D. pipe on either side of the existing pipe. On the city side of the bay, traditional open trench pipe laying construction would be used to lay 3,700 l.f. of new forcemain along Cape Arago Highway north to Newmark Avenue. This section of pipe installation would be more expensive due to existing utilities than the section on the other side of the bay where the 12″ pipe could be installed in a joint trench with the proposed 18″ pipe.

Disadvantages to the double force main option include higher capital costs due to additional piping, and additional low flow pumps. As mentioned above, additional pumps will be more efficient, therefore a life cycle cost comparison must be done to determine the preferred method.

Tables 4.1.1 and 4.1.2 below summarize the estimated costs of these two conveyance options.

Table 4.1, Forcemain Cost Estimate (Single Force Main)

Forcemain (Single Force Main)					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Mobilization & Permits (15%)	ls	all	$463,500.00	$463,500.00
2	18-Inch HDPE Direct Bury Forcemain	lf	8000	$135.00	$1,080,000.00
3	18-Inch HDPE HDD Forcemain	lf	5000	$400.00	$2,000,000.00
4	AC Pavement Patch in North Spit Pkwy.	ls	all	$10,000.00	$10,000.00
		Construction Total			**$3,553,500.00**
		Contingency (15%)			$533,025.00
		Engineering (18%)			$639,630.00
		Administrative costs (3%)			$106,605.00
		Total Project Costs			**$4,832,760.00**

Table 4.2, Forcemain Cost Estimate (Double Force Main)

FORCEMAIN (DOUBLE FORCE MAIN OPTION)					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Mobilization & Permits (15%)	ls	all	$697,432.50	$697,432.50
2	18-Inch HDPE Direct Bury Forcemain	lf	8000	$180.00	$1,440,000.00
3	18-Inch HDPE HDD Forcemain	lf	5000	$400.00	$2,000,000.00
4	12-Inch HDPE Direct Bury Forcemain (Cape Arago Hwy)	lf	3695	$150.00	$554,250.00
5	12-Inch HDPE Direct Bury Forcemain (North Spit)	lf	7170	$90.00	$645,300.00
6	AC Pavement Patch in North Spit Pkwy.	ls	all	$10,000.00	$10,000.00
		Construction Total			**$5,346,982.50**
		Contingency (15%)			$802,047.38
		Engineering (18%)			$962,456.85
		Administrative costs (3%)			$160,409.48
		Total Project Costs			**$7,271,896.20**

4.2 Pump Station

Transfer of raw sewage from the current treatment site in Empire to the north spit requires a pump station. The pump station will be required to pump the Peak Wet Weather Flow (PWWF) with the largest pump out of service. Because different pumping arrangements will be required for the different force main configurations, there are two corresponding options for the pump station.

4.2.1 Single Force Main Pump Station

As discussed in Section 4.1.1, the pump station for a single 18″ force main will have to be able to pump between 8.6 MGD against 170′ of head and 2.3 MGD (2.0 fps) against 51′ of head. A cursory review of pump curves yields a system including three (2+1 redundancy) 250 horsepower pumps to pump between 4.3 MGD and 8.6 MGD and a single 85 horsepower pump to pump from 2.3 MGD to 4.3 MGD. All motors will be equipped with variable speed drives (VFDs) to maximize efficiency.

4.2.2 Double Force Main Pump Station

The pump station for the double force main option will have two different pump systems, one to pump the low summertime flows (<1.44 MGD) through the existing 10″ pipe and another to pump the high wintertime flows (1.44 MGD to 7.16 MGD) through the new 18″ pipe. During the PIF (>7.16 MGD) both sets of pumps will combine to pump the 8.6 MGD peak flow.

The first system, for low summertime flows, will consist of a single 20 horsepower pump to handle most dry weather flows (<1 MGD) and will include three (2+1 redundancy) 30 horsepower pumps to pump between 1 MGD and 1.44 MGD.

The second system will include four (3 +1 redundancy) 105 horsepower pumps to pump flows up to 7.16 MGD through the new 18″ pipe. All motors will be equipped with variable speed drives (VFDs) to maximize efficiency.

The maximum size of the wetwell is governed by the maximum detention time during dry weather flow, however, as noted in section 4.1.1, since the time in the pipe is well in excess of the 35 minute limit, the wastewater will need to be chlorinated. The 35 minute criterion is based on the potential generation of hydrogen sulfide and, with chlorination, detention time is not a factor. However, the formula for this determination is:

$V_{maximum} = (T_{minutes} \times Q_{min})$

$V_{maximum}$ = Maximum volume in gallons
$T_{minutes}$ = Maximum detention time in minutes (35 minutes)
Q_{min} = Dry Weather Flow (0.75 MGD = 520 gpm)

Therefore: $V_{maximum}$ = (35 minutes x 520 gpm) = 18200 gallons (2432 ft^3).

The minimum size of the wetwell is determined to keep the pumps from cycling more than 10 times per hour. The DEQ formula for this determination is:

$V_{minimum} = (T_{minutes} \times Q_{max}) / 4$

$V_{minimum}$ = Minimum volume in gallons
$T_{minutes}$ = Target time between pump starts in minutes (6 minutes)
Q_{max} = Pump design capacity (8.6 MGD = 5970 gpm)

Therefore: $V_{minimum}$ = (6 minutes x 5970 gpm)/4 = 8955 gallons (1197 ft^3).

Based on the pumps identified above, and according to the Flygt *Design Recommendation For Pump Stations with Midrange Centrifugal Wastewater Pumps*, the wetwell would be required to be at least 12' x 10' to fit the four pumps required in the single force main option and 12' x 15' to fit the eight pumps in the double force main option. This bottom area, combined with the maximum and minimum volumes calculated above, result in a storage depth of 10 to 20 feet. For the purpose of this evaluation, a storage depth of 15' will be used. The storage depth is the area between the pump start level (HWL) and the pump stop level (LWL). The minimum depth below the LWL for proper pump operation is 3'. The HWL must be a minimum of 1' below the invert of the inflow pipe. For this project, the invert of the sewer pipe is 6' below the ground surface. The combination of all these factors results in a total wetwell depth of 25'

Tables 4.2.1 and 4.2.2 below summarize the costs associated with the pump stations for the two different conveyance options:

Table 4.2.1, Pump Station Cost Estimate (Single Force Main)

PUMP STATION (SINGLE FORCE MAIN OPTION)					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Mobilization & Permits (15%)	ls	all	$189,798.00	$189,798.00
2	Construction Facilities and Temporary Systems	ls	all	$15,000.00	$15,000.00
3	Wetwell, Excavation and Installation	ls	all	$207,920.00	$207,920.00
4	Pumps, VFDs, Accessories, and Installation	ls	all	$405,600.00	$405,600.00
5	Electrical, Wiring, Panels, Level Controls, SCADA	ls	all	$90,100.00	$90,100.00
6	Site Piping, Valves, Fittings, Valve Vault, Installation	ls	all	$180,000.00	$180,000.00
7	Flowmeter and Vault	ls	all	$33,200.00	$33,200.00
8	Chlorine drip (Clorine generator)	ls	all	$70,000.00	$70,000.00
9	24-Inch Influent Pipe	lf	50	$240.00	$12,000.00
10	Influent Pipe Connection	ls	all	$25,000.00	$25,000.00
11	AC Pavement and Base	ls	all	$6,500.00	$6,500.00
12	Demolition and Abandonment of Existing Treatment Site	ls	all	$220,000.00	$220,000.00
		Construction Total			**$1,455,118.00**
		Contingency (15%)			$218,267.70
		Engineering (18%)			$261,921.24
		Administrative costs (3%)			$43,653.54
		Total Project Costs			**$1,978,960.48**

Table 4.2.2, Pump Station Cost Estimate (Double Force Main)

PUMP STATION (DOUBLE FORCE MAIN OPTION)					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Mobilization & Permits (15%)	ls	all	$202,401.00	$202,401.00
2	Construction Facilities and Temporary Systems	ls	all	$15,000.00	$15,000.00
3	Wetwell, Excavation and Installation	ls	all	$264,490.00	$264,490.00
4	Pumps, VFDs, Accessories, and Installation	ls	all	$416,650.00	$416,650.00
5	Electrical, Wiring, Panels, Level Controls, SCADA	ls	all	$106,500.00	$106,500.00
6	Site Piping, Valves, Fittings, Valve Vault, Installation	ls	all	$180,000.00	$180,000.00
7	Flowmeter and Vault	ls	all	$33,200.00	$33,200.00
8	Chlorine drip (Clorine generator)	ls	all	$70,000.00	$70,000.00
9	24-Inch Influent Pipe	lf	50	$240.00	$12,000.00
10	Influent Pipe Connection	ls	all	$25,000.00	$25,000.00
11	AC Pavement and Base	ls	all	$6,500.00	$6,500.00
12	Demolition and Abandonment of Existing Treatment Site	ls	all	$220,000.00	$220,000.00
		Construction Total			**$1,551,741.00**
		Contingency (15%)			$232,761.15
		Engineering (18%)			$279,313.38
		Administrative costs (3%)			$46,552.23
		Total Project Costs			**$2,110,367.76**

For the purposes of this study, the single force main option will be utilized. Should the north spit be a viable alternative for the City, additional investigation should be done to determine the life-cycle costs of each of these conveyance options.

5.0 Treatment Alternatives

Secondary treatment alternatives considered for a North Spit municipal wastewater treatment facility have been based on using stabilization lagoon technologies. Stabilization lagoon technologies will conform to DEQ's design guidelines for facultative lagoon systems. Facultative lagoons are the most common type of lagoon treatment system employed for municipal treatment facilities and can be provided with and without surface aeration.

The requirements for a lagoon system were evaluated based on the loading criteria presented in Section 2 both with and without primary treatment. The evaluation was based on the assumption that the lagoon system will provide secondary treatment to either raw municipal wastewater (screened) or effluent from primary treatment provided at the existing Plant 2 site; and that the secondary effluent will meet anticipated NPDES requirements for Total Suspended Solids (TSS) and Biological Oxygen Demand (BOD) for an ocean discharge.

Use of an enhancement marsh to provide tertiary treatment and nutrient removal is also evaluated in this section. The need for enhanced wastewater treatment using wetlands was not considered necessary based on an analysis of expected secondary effluent quality from the facultative lagoon system. Impacts to natural wetland systems currently existing on the site make this option less desirable.

5.1 Facultative Lagoon System

Facultative lagoons are typically 5-8 ft deep and employ a combination of aerobic, facultative, and anaerobic treatment. The upper layer of this type of lagoon is aerobic with photosynthesis, re-aeration (and in some cases surface aeration) providing oxygen for aerobic stabilization. Middle layers are facultative, employing a combination of aerobic and anaerobic biological activity to treat suspended organic material. The lower layer of a facultative pond includes an anaerobic zone where residual solids are digested and reduced to form a stabilized sludge.

Areal loading rates of 20-40 lbs/acre-day are used for facultative lagoon systems where average winter temperatures are 0-15 Degrees C (32-60 Degrees F).[7] For stabilization ponds without primary treatment it is also desirable to have the primary cells loaded at less than 90 lbs/acre-day.[8] When lagoon aeration is provided, areal loading rates are not considered limiting, instead, kinetics and detention time governs the design criteria.

5.1.1 Alternate 1 - Facultative System

Alternate 1 assumes all wastewater collected at the Plant 2 WWTP will be conveyed to the North Spit for treatment and disposal in a land based lagoon system. Treatment facilities to be provided on the North Spit include a new headworks, new parallel primary treatment lagoons, and secondary treatment in the existing ASB lagoon. Area requirements for a facultative lagoon system that has not received primary treatment were estimated based on projected maximum month biological oxygen demand (BOD) loading of 2,485 lbs/day and an overall design loading rate of 35

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[7] EPA Design Manual: Municipal Wastewater Stabilization Ponds

[8] WEF M0P 8

lbs/day-acre. The total surface area required for the lagoon system shown is 71 acres including 31 acres from the existing ASB.

A preliminary layout for alternative 1 is shown in Figure 5.1. The primary lagoon has been split into two parallel trains. This configuration minimizes overall loading on the primary cells while maintaining an optimum hydraulic flow pattern that promotes plug flow and minimizes short-circuiting. Due to the area required for treatment without the advent of aeration, the new primary lagoons are shown impacting wetland areas and mitigation costs will need to be considered in the cost of this alternative.

Expected Performance (BOD Removal)

Expected BOD removal in the lagoon system was modeled using two methodologies as described in the EPA Design Manual: the Wherner-Wilhelm Equation and the Plug Flow Model. The assumptions used in the analysis include:

- The detention time will be limited by the Maximum Month Flow (MMF) of 2.65 MGD,
- The depth of the Primary cells (with the active volume reduced to 5.5 ft to account for sludge accumulation) will be 7 feet deep
- The secondary cells will be 8 ft deep,
- The treatment system will provide 85% BOD removal,
- The lagoon is modeled as a Plug flow reactor with a dispersion constant equation of one,
- The lagoon is modeled using the Wherner–Wilhelm equation with a first order reaction rate for the facultative lagoon system (k20) at 0. 15 -day1,
- The plug flow reaction rate constant will vary with flow1,
- The minimum lagoon temperature will be 10.5 degrees C.

Table 5.1
Alternative 1

Primary Treatment at WWTP	NO				
Type of Treatment	Facultative Lagoon				
	Area	**Depth**	**Volume**	**Detention Time**	**Removal**
	Acres	**ft.**	**MG**	**days**	**Percent**
Primary Lagoons	40	7[1]	66.25	25	64%
Secondary Lagoons	31	8	79.5	30	76%
Overall Performance					92%
Potential Impacts to natural wetlands	30				

1. Actual process water depth is calculated at 5.5 feet, accounting for sludge accumulation.

The layout shown in Figure 5.1 is the preferred configuration for providing the required BOD removal. The two celled primary lagoon promotes plug flow and the area of 43 acre-ft includes

POND	AREA (ac)
1A	20.0
1B	20.0
2	15.5
3	15.5

N

1"=800'

SHN Consulting Engineers & Geologists, Inc.

City of Coos Bay
Wastewater Treatment Plant
Coos Bay, Oregon

Facultative Lagoon System
Alternate #1

May 2010 | 609031-115-FIG-5-x_Alternatives | Figure 5.1

\\Coosbay\projects\2009\609031-CBGeoGenSvc\115-N-Spit-WW-Trtmtnt-Site\Figs , SAVED: 5/18/2010 11:28 AM WWHITE, PLOTTED: 5/19/2010 11:52 AM, WALTER WHITE

additional capacity for future industrial discharges on the north spit which has not been taken into account in the loading projections for Plant 2. Alternative 1 requires no additional energy to accomplish the desired level of treatment.

Advantages

- Natural process
- Low complexity and operational cost
- No additional power cost
- Parallel primary cells provide reliability and operational flexibility

Disadvantages

- Large footprint
- Wetlands impact and potential mitigation requirements for approximately 30 acres

5.1.2 Alternative 2 - Facultative System with Primary Treatment

Alternative 2 assumes that preliminary and primary treatment will occur at the existing Plant 2 site with secondary treatment occurring at the North Spit site. The analysis of performance is based on the assumption that primary treatment at Plant 2 would provide 25 percent removal of BOD and 40 percent removal of total suspended solids (TSS).

Based on a BOD loading of 1,990 ppd (80% of the maximum month loading of 2,485 ppd) and an areal loading rate of 35 lbs/acre-day, the area required for the facultative lagoon systems would be reduced to 53 acres. The Wherner -Wilhelm equation predicts that an additional detention time of 45 days will be required in the secondary system to achieve an overall removal rate of 85% or better. Expected performance for the system proposed in Alternative 2 is summarized in Table 5.2.

Table 5.2
Alternative 2

Primary Treatment at WWTP	YES				
Type of Treatment	Facultative Lagoon				
	Area	**Depth**	**Volume**	**Detention Time**	**Removal**
	Acres	**ft.**	**MG**	**days**	**Percent**
Primary Lagoons	22	7[1]	37	14	51%
Secondary Lagoons	31	8	79.5	30	52%
Overall Performance					89%
Potential Impacts to natural wetlands	12				
1. Actual process water depth is calculated at 5.5 feet, accounting for sludge accumulation					

POND	AREA (ac)
1A	11.0
1B	11.0
2	15.5
3	15.5

N

1"=800'

\\Coosbay\projects\2009\609031-CBGeoGenSvc\115-N-Spit-WW-Trtmtnt-Site\Figs , SAVED: 5/18/2010 11:28 AM WWHITE, PLOTTED: 5/19/2010 11:53 AM, WALTER WHITE

SHN Consulting Engineers & Geologists, Inc.

City of Coos Bay
Wastewater Treatment Plant
Coos Bay, Oregon

Facultative Lagoon System
Primary Treatment Plant 2
Alternate #2

May 2010 | 609031-115-FIG-5-x_Alternatives | Figure 5.2

Advantages

- Natural process
- Low complexity and operational cost
- No additional power cost
- Smaller footprint than Alternative 1
- Parallel primary cells provide reliability and operational flexibility

Disadvantages

- Requires primary treatment and re-pumping at WWTP 2 at significant increased cost
- Wetlands impact and potential mitigation requirements for approximately 12 acres

5.1.3 Alternative 3 - Facultative System with Supplemental Aeration

In lagoon systems with supplemental aeration, the size of the facility is not based on areal loading rates, but rather, on the detention time. In addition, the supplemental aeration system needs be sized to provide sufficient air for oxidation of BOD and mixing.

Based on the reaction rate assumed for facultative pond systems, the Wherner-Wilhelm equation predicts that an 85% reduction in BOD will require a detention time of 55 days. With supplemental aeration this detention time can be provided in a smaller footprint by increasing the depth of the lagoon system. Ten feet is considered maximum for a facultative system[1] and was used to estimate the minimum area requirements for a facultative lagoon system with supplemental aeration.

If the depth of both primary and secondary ponds were increased to 10 ft., the required detention time could be provided with a primary lagoon having a water surface area of approximately 15 acres. If only the primary cells are provided with aeration and the average depth increased to 10 ft., the required area of the primary lagoons will be reduced to a total of 32 acres. That scenario is presented in Table 5.3.

Table 5.3 Alternative 3					
Primary Treatment at WWTP	NO				
Type of Treatment	Facultative Lagoon with Supplemental Aeration				
	Area	**Depth**	**Volume**	**Detention Time**	**Removal**
	Acres	**ft.**	**MG**	**days**	**Percent**
Primary Lagoons	32	10[1]	66.25	25	68%
Secondary Lagoons	31	8	79.5	30	52%
Overall Performance					85%
Potential Impacts to Wetlands	22				
1. Actual process water depth is calculated at 8.5 feet, accounting for sludge accumulation					

POND	AREA (ac)
1A	16.0
1B	16.0
2	15.5
3	15.5

N

1"=800'

SHN Consulting Engineers & Geologists, Inc.

City of Coos Bay
Wastewater Treatment Plant
Coos Bay, Oregon

Facultative Lagoon System
(with Supplemental Aeration)
Alternate #3

May 2010 | 609031-115-FIG-5-x_Alternatives | Figure 5.3

\\Coosbay\projects\2009\609031-CBGeoGenSvc\115-N-Spit-WW-Trtmtnt-Site\Figs , SAVED: 5/18/2010 11:28 AM WWHITE, PLOTTED: 5/19/2010 11:55 AM, WALTER WHITE

Aeration Requirements

Aeration requirements for both facultative systems with supplement aeration are generally based on oxidation of influent BOD. Based on a projected maximum month BOD load, aeration requirements for oxidation are estimated to be 105 to 130 HP depending on the efficiency of the aerator.

Advantages

- Smaller footprint than Alternative 1
- Parallel primary cells provide reliability and operational flexibility

Disadvantages

- Aerators add operational complexity and maintenance requirements
- Additional power costs (130 HP)

5.1.4 Alternative 4 - Aerated Partial Mix / Facultative System , Existing Basin

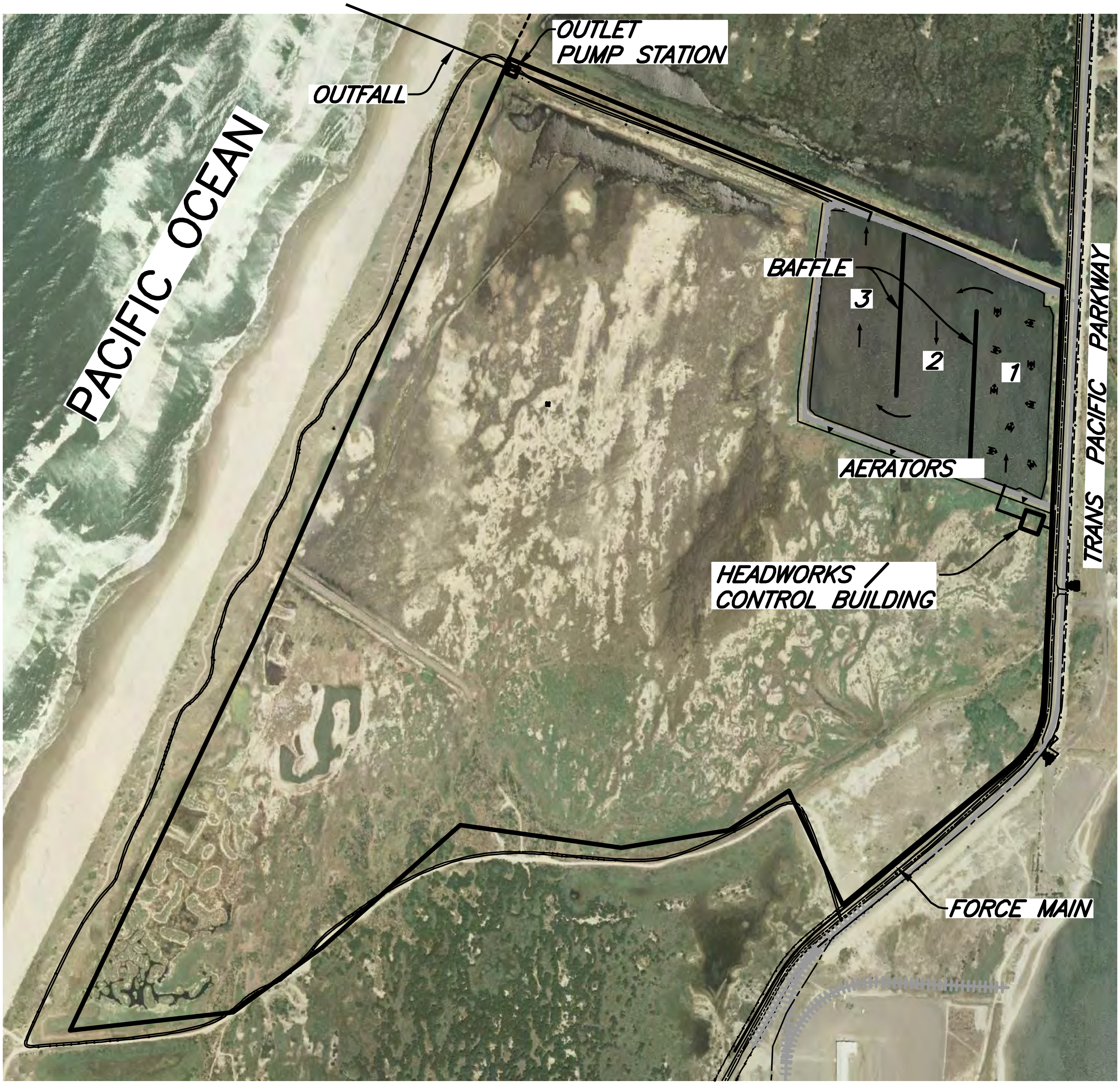
In a partially mixed aerated pond, aerators are used to transfer oxygen to the liquid to maintain aerobic conditions in the ponds. Oxygen requirements are based on the BOD to be oxidized and are reduced from one cell to the next as the wastewater flows through the system. Reaction rates for aerobic pond systems are greater than that for facultative ponds and therefore, the required detention times are greatly reduced.

The reaction rates assumed for aerated partially mixed systems assume complete oxygen dispersion. In practice, zones of complete oxygen dispersion vary depending on the type of aerator. It is generally assumed that, at a minimum 5 to 10 HP / MG is required for a partially mixed lagoon system.

Performance

It is estimated that the existing lagoon can provide approximately 30 days of detention time at the projected maximum month flows in 2030. Because it is not feasible to aerate the entire lagoon at a level that will maintain partial mixing, the lagoon will need to be divided into multiple cells to improve mixing characteristics.

One possible configuration, shown in Figure 5.4, provides three cells, each with 10 days of detention time. Aeration is provided in Cell 1 only (total 132 HP is assumed) which should provide partial mixing but still allow for settling of solids. Based on a more conservative reaction rate in cell 1 (considering the reduced rate of mixing) and reaction rates in Cell 2 and Cell 3 (based on facultative systems), the existing ASB configuration should provide the required 85% removal of BOD.



POND	AREA (ac)
1	10.3
2	10.3
3	10.3

N

1"=800'

SHN Consulting Engineers & Geologists, Inc.	City of Coos Bay Wastewater Treatment Plant Coos Bay, Oregon	Aerated Lagoon System (Partially Mixed) Alternate #4
	May 2010 / 609031-115-FIG-5-x_Alternatives	Figure 5.4

- Cell 1: kpm= 0.25-day kpm(w) = 0.14-day
- Cells 2, and 3, : kf = 0.066-day

Table 5.4 Alternative 4					
Primary Treatment at WWTP	NO				
Type of Treatment	Aerated Lagoon/Partial Mix (132 Hp)				
	Area	**Depth**	**Volume**	**Detention Time**	**Removal**
	Acres	**ft.**	**MG**	**days**	**Percent**
Secondary Lagoons	31	8	79.5	30	85%
Overall Performance					85%
Potential Impacts to Wetlands	0				

Advantages

- No wetlands impact
- Small footprint

Disadvantages

- First cell of ASB acts as primary,
- Lacks redundancy of parallel trains,
- Aerators add additional operational complexity and maintenance requirements,
- Annual power costs for 130 HP.

5.1.5 Alternative 5 - Aerated Partial Mix / Facultative System , Primary Cells

The design criteria for partially mixed lagoon systems, discussed in the previous section, were applied to a system with two parallel primary cells, (Figure 5.5). The addition of the primary cells allows for greater operational flexibility in terms of sludge collection and removal, and the increased detention time increases performance reliability as summarized in Table 5.5. Aeration requirements are estimated to be 90 HP reflecting lower mixing energy requirements when compared to Alternative 1.

POND	AREA (ac)
1A	4.6
1B	4.6
2	10.0
3	10.0
4	10.0

N

1"=800'

SHN Consulting Engineers & Geologists, Inc.

City of Coos Bay
Wastewater Treatment Plant
Coos Bay, Oregon

Aerated Lagoon System
Primary Treatment Plant 2
Alternate #5

May 2010 | 609031-115-FIG-Alt5-Fig5_5 | Figure 5.5

\\Coosbay\projects\2009\609031-CBGeoGenSvc\115-N-Spit-WW-Trtmtnt-Site\Figs , SAVED: 5/19/2010 11:45 AM WWHITE, PLOTTED: 5/19/2010 11:47 AM, WALTER WHITE

Table 5.5 Alternative 5					
Primary Treatment at WWTP	NO				
Type of Treatment	Facultative Lagoon With Supplemental Aeration				
	Area	**Depth**	**Volume**	**Detention Time**	**Removal**
	Acres	**ft.**	**MG**	**days**	**Percent**
Primary Lagoons	9.7	10[1]	19.9	7.5	65%
Secondary Lagoons	31	8	79.5	30	66%
Overall Performance					88%
Potential Impacts to wetlands	0				
1. Water depth 5.5 to account for sludge accumulation					

Advantages

- No wetlands impact
- Small footprint
- Primary cells provide reliability and operational flexibility

Disadvantages

- Aerators add additional operational complexity and maintenance requirements
- Power cost (90 HP)

5.1.6 Preferred Alternative

Selection of the preferred secondary treatment alternative was based on consideration of potential impacts to wetlands, constructability, capital cost, and annual operations and maintenance costs. The facultative lagoon system Alternative 1, and the partially mixed aerated lagoon systems, Alternatives 4 and 5 are feasible alternatives; however, Alternate 1 may have unacceptable costs due to potential impacts to wetlands.

Costs for the three alternatives are presented in Tables 5.6, 5.7, and 5.8. The facultative lagoon system covers a larger area and costs more to construct but if annual power cost is included in the cost comparison the facultative lagoon system, excluding the estimated cost of wetlands mitigation is comparable to the two aerated alternatives.

Table 5.6, Treatment Alternative 1 Cost Estimate

Treatment (Secondary Alternative #1)					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Mobilization & Permits (15%)	ls	all	$420,348.00	$420,348.00
2	Primary Berms	cy	107,230	$6.00	$643,380.00
3	Primary Lagoon Liner	sf	1,742,400	$0.60	$1,045,440.00
4	Secondary Lagoon Liner	sf	1,530,000	$0.60	$918,000.00
5	Yard Piping	ls	all	$50,000.00	$50,000.00
6	Fencing	lf	3,900	$20.00	$78,000.00
7	Secondary Baffle	ls	all	$67,500.00	$67,500.00
		Construction Total			**$3,222,668.00**
		Contingency (15%)			$483,400.20
		Engineering (18%)			$580,080.24
		Administrative costs (3%)			$96,680.04
		Wetlands Delineation			$60,000.00
		Secondary Sludge Survey			$60,000.00
		Remove Sludge			$3,000,000.00
		Total Project Costs			**$7,502,828.48**

Table 5.7, Treatment Alternative 4 Cost Estimate

Treatment (Secondary Alternative #1)					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Mobilization & Permits (15%)	ls	all	$420,348.00	$420,348.00
2	Primary Berms	cy	107,230	$6.00	$643,380.00
3	Primary Lagoon Liner	sf	1,742,400	$0.60	$1,045,440.00
4	Secondary Lagoon Liner	sf	1,530,000	$0.60	$918,000.00
5	Yard Piping	ls	all	$50,000.00	$50,000.00
6	Fencing	lf	3,900	$20.00	$78,000.00
7	Secondary Baffle	ls	all	$67,500.00	$67,500.00
		Construction Total			**$3,222,668.00**
		Contingency (15%)			$483,400.20
		Engineering (18%)			$580,080.24
		Administrative costs (3%)			$96,680.04
		Wetlands Delineation			$60,000.00
		Secondary Sludge Survey			$60,000.00
		Remove Sludge			$3,000,000.00
		Total Project Costs			**$7,502,828.48**

Table 5.8, Treatment Alternative 5 Cost Estimate

Treatment (Secondary Alternative #5)					
Item No.	**Description**	**Units**	**Quantity**	**Unit Cost**	**Total Cost**
1	Bonds, Insurance, Mobilization & Permits (15%)	ls	all	$375,423.30	$375,423.30
2	Primary Berms	cy	68,933	$6.00	$413,598.00
3	Primary Lagoon Liner	sf	392,040	$0.60	$235,224.00
4	Secondary Lagoon Liner	sf	1,530,000	$0.60	$918,000.00
5	Aerators	ls	all	$180,000.00	$180,000.00
6	Install	ls	all	$90,000.00	$90,000.00
7	Electrical	ls	all	$90,000.00	$90,000.00
8	Baffles	ls	all	$150,000.00	$150,000.00
9	Fencing	lf	6,000	$20.00	$120,000.00
10	Secondary Effluent	lf	1,700	$180.00	$306,000.00
		Construction Total			**$2,878,245.30**
		Contingency (15%)			$431,736.80
		Engineering (18%)			$518,084.15
		Administrative costs (3%)			$86,347.36
		Wetlands Delineation			$60,000.00
		Secondary Sludge Survey			$60,000.00
		Remove Sludge			$3,000,000.00
		Total Project Costs			**$7,034,413.61**

All alternatives include an estimate of cost for removal of sludge in the existing stabilization. In a sludge survey conducted by in 1993 as part of the "Closure Plan for the Wastewater Treatment Impoundments," the average sludge depth in the pond was 3.8 ft., which translates into an estimated volume of 40 million gallons (MG) or, assuming 6 percent concentration in the basin, approximately 10,000 dry tons of biosolids.

Since 1993 Weyerhaeuser's wastewater discharge to the stabilization basin decreased and in 2003 ceased all together. The volume of biosolids in the basin has probably been reduced since the last inventory because anaerobic digestion and solids wash-out would tend to reduce the volume of bio-solids. It is recommended that a solids survey be conducted to verify the actual biosolids inventory. Without this information, a $3 million placeholder has been included in the cost estimates to account for the potential of harvesting Biosolids. A sludge survey would allow a more accurate assessment of costs for this item.

Wetlands Impact

Alternative 1 has the potential to impact areas that may be considered jurisdictional wetlands by the Division of State Lands (DSL). It would be difficult to obtain the permits required for construction of the primary lagoons or enhancement wetlands without significant mitigation. Alternatives 4 and 5 both provide required secondary treatment without significantly impacting wetlands and are considered preferable to Alternative 1 for this reason. Discussions with DSL are needed to confirm the preferred alternative.

Selection of Preferred Option

Based on considerations of operational flexibility and reliability, Alternative 5 is considered the preferred alternative even though the project cost was estimated to be higher than Alternative 4. Alternative 5 has the following advantages:

- Parallel primary lagoons (one side can be taken off line for sludge removal and maintenance)
- Power requirements are less for Alternative 5
- Higher degree of reliability
- Excess capacity to handle additional loadings for future North Spit industries

The basis of selecting the preferred alternative is summarized in the evaluation matrix presented in Table 5.9. Rankings accorded on a scale of 1-5 with 5 being more positive.

Table 5.9
Evaluation Matrix for Selection of Preferred Alternative

Alts.		Cost	Power Usage	Impact to Wetlands[1]	Operation Complexity	Operation Flexibility[2]	Rank
1	Facultative System	2	5	1	5	1	14
2	Facultative System Primary WWTP #2	1	5	3	3	-	12
3	Facultative System Supplemental Air	3	1	2	4	1	11
4	Partial Mix /Facultative In Existing Basin	5	1	5	4	-	15
5	Partial Mix /Facultative With Primary Cells	4	2	5	4	1	16

1. Verify with DSL.
2. Operational flexibility provided by parallel primary trains

5.1.7 Biosolids Management

The primary cells in the combined aerated / facultative process selected as the preferred alternative will be partially mixed. Air is provided at a low intensity that allows for settling of solids throughout the basin. It is recommended that an area of increased depth be provided in the influent portion of the ponds to allow solids to accumulate. A few acre portion of each pond would be increased in depth to 12 ft.

Biosolids Volume

Solids accumulation over time was estimated based on yield factor for sludge yield per pound of BOD applied. Due to endogenous respiration the solids generated per pound of BOD are significantly less than typical activated sludge process. Yield estimates of 0.5 pound of biosolids produced per pound of BOD applied are considered appropriate.[9] Projected BOD loading at Plant #2 in 2030 is 2,245 l/day[10]. Based on a yield factor of 0.5 solids will accumulate at a rate of 204 tons per year.

Based on BOD removal rates, a percent of the solids or 132 tons will be removedin the primary cells. If the settled solids are assumed to have a concentraton of 6% there will be an accumulation of 1.5 acre ft in the primary cells per year or based on the 9 acre area approximately 0.18 ft per year.

The volume of biosolids generated facultative processes in the secondary lagoon and the suspended solids that are carried over from the primary accumulate will accumulate in the secondary ponds at a rate of 71.4 tons per year. If the settled solids are assumed to have a concentraton of 6% there will be an accumulation of 0.90 acre ft in the secondary cells per year or, based on 30 acres, 0.03 ft per year.

Management Plan

Biosolids removal from the primary cells should be taken into account in evaluating the combined Aerated /Facultative Lagoon System. Biosolids should be removed every 10-15 years. Cost estimates for dredging and hauling the biosolids were based on an estimated 300.00 per dry ton for dredging and hauling, an annual cost of approximately 40,000 per year. The annual costs are based on using the City's permitted land application sites.

5.1.8 Expected Performance (Nitrogen Removal)

Nitrogen removal in facultative lagoon systems is positively correlated with temperature, pH, and detention time. Nitrogen removal is believed to occur according to the following processes:

- Gaseous ammonia stripping to atmosphere
- Ammonia assimilation in algal biomass
- Nitrate assimilation in algae
- Biological nitrification-denitrification

There are several empirically derived models that predict total nitrogen removal in lagoon systems Volatilization of ammonia is considered the major pathway for nitrogen removal and is the basis for development of these models. Volatilization is highly dependent upon pH and alkalinity. In general, higher pH and alkalinity levels promote more volatilization of ammonia.

Based on a detention time of 37 days at the maximum month wet weather flow (MMWWF) of 2.65 MGD and an assumed alkalinity of 100 mg/l (pH 7.67), ammonia removal in the aerated lagoon system was calculated to be 36 percent. Based on the projected (2030) maximum month ammonia nitrogen (NH3-N) loading of 408 ppd (2009, EEE), the NH-3 influent concentration during MMWWF would be approximately 19 mg/l and with an anticipated effluent concentration of 12 mg/l.

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[9] 2005, "National Manual of Good Practice for Biosolids", The National Biosolids Partnership
[10] "Technical Memo EE2"

Nitrogen removal is affected by detention time. At the projected Average Dry Weather Flow (ADWF) of 1 MGD and a detention time of 91 days; predicted nitrogen removal increases to 50 percent. At ADWF the maximum month NH3-N loading would result in influent NH-3 concentrations of 49 mg/l and effluent concentration of 25 mg/l.

5.2 Enhanced Treatment Wetlands

Preliminary assessment of treatment options included enhanced treatment wetlands. Wetlands are effective in treating biological oxygen demand, suspended solids nitrogen and phosphorous, as well as removing metals, organics, and pathogens. In addition wetlands provide filtering and adsorption mechanism for the removal of constituents such as pharmaceuticals that are present in wastewater in low concentrations making removal by other means difficult.

Constructed free-water surface wetlands are planted with alternating cells of emergent herbaceous wetlands plants such as bulrush and free-water surface zones with submerged aquatic vegetation rooted to the bottom, such as Potamogeton or pondweed. The open water surface areas are designed to promote nitrification of ammonia to nitrate which is denitrified in the more densely emergent vegetated cells.

In discussions with the Division of State Lands (DSL) it was stated that constructed wetlands for wastewater treatment would not be considered equivalent to natural wetlands. Therefore the enhanced treatment wetlands originally considered would be difficult to construct and are not recommended as part of the facility.

6.0 Disinfection Alternatives

DEQ requires that effluent be disinfected, after treatment, equivalent to thorough mixing with sufficient chlorine to provide a residual of at least 1 part per million after 60 minutes of contact time. The existing stabilization basin and outfall does not include a disinfection system.

6.1 Method Alternatives

Two primary alternatives have been considered to meet the DEQ requirements.

6.1.1 Ultraviolet

In a typical lagoon treatment system, the ultraviolet transmissivity of the effluent is poor enough to render ultraviolet disinfection impractical. DEQ has recently reviewed a system which is capable of treating low transmissivity effluent, however at this time the system does not have the capacity to treat the magnitude of flows expected at WWTP #2. However, should the north spit be a viable option of treatment, further investigation should be done to review this methodology and determine if the technology will allow for a larger scale use.

6.1.2 Sodium Hypochlorite

In order to avoid the hazards associated with use and storage of gaseous chlorine, a sodium hypochlorite injection system is recommended between the northwest corner of the secondary lagoon and the outfall pump station. Based on the Washington State Department of Ecology Criteria for Sewage Works Design, the dosage range of primary effluent is from 5 to 10 mg/L of chlorine for appropriate disinfection. A chlorine contact chamber will be sized to allow a contact time sufficient to meet the effluent requirements discussed in Table 2.2.

To meet these requirements a chlorine contact chamber 10' wide by 10' tall by 500' long must be constructed. A serpentine configuration results in a chamber 10' high by 50' wide by 100' long. The estimated costs associated with disinfection are summarized below in table 6.1.2.

Table 6.1.2, Disinfection Cost Estimate

DISINFECTION					
1	Potable water to site (2" HDPE)	lf	1500	$15.00	$22,500.00
2	Chlorine Monitor	ls	1	$4,000.00	$4,000.00
3	Chlorine contact chamber	cy	450	$800.00	$360,000.00
4	Injector pump & mixer	ls	1	$100,000.00	$100,000.00
5	Housing	sf	425	$250.00	$106,250.00
6	Disinfection pump	ls	1	$50,000.00	$50,000.00
7	Mag meter	ls	1	$35,000.00	$35,000.00
8	Electrical	ls	1	$5,000.00	$5,000.00
		Construction Total			**$682,750.00**
		Contingency (15%)			$102,412.50
		Engineering (18%)			$122,895.00
		Administrative costs (3%)			$20,482.50
		Total Project Costs			**$928,540.00**

7.0 Evaluation of Existing Outfall

One of the advantages of the proposed North Spit treatment facility is the potential to utilize the existing ocean outfall for effluent disposal. The existing outfall system consists of an effluent pump station and a 30 inch diameter concrete coated, epoxy lined, steel pipe extending approximately 4,500 ft off-shore to a depth of approximately 60 ft MLLW. At the terminus of the outfall pipe, a bifurcated diffuser was installed creating a “Y” configuration. Each branch of the diffuser contains 16 three inch discharge ports which are equipped with a flapper style check valve. It is proposed that this outfall system be used for the discharge of treated wastewater from the North Spit treatment system with only minor modification. Based on a preliminary review of the existing outfall capacity, a new effluent pump station will be required.

7.1.1 Description

Constructed in 1972, the existing outfall and effluent pump station were originally designed to allow discharge of treated wastewater by gravity or pumping. Under gravity flow, wastewater is allowed to flow through the effluent pump station wetwell and overflow into the 30-inch outfall pipe. With this mode of operation, the outfall has an estimated capacity of 3 MGD. For flow greater than 3 MGD it is necessary to pump effluent through the outfall pipe and an effluent pump station was installed for this purpose. This pump station was originally equipped with a large vertical turbine pump that discharges through a 12-inch header pipe back into the 30-inch outfall line downstream of the pump station. During pumping, the gravity line is isolated from the pump discharge by closing an overflow gate. Pumping equipment has since been removed from the existing facility, although replacement equipment should be easily retrofit into the existing wetwell structure with any future upgrade.

When Weyerhaeuser constructed the ASB, an 18-inch discharge line was installed from the northwest corner of the ASB basin to a manhole upstream of the effluent pump station wet-well. The manhole was equipped with a parshal flume for flow monitoring prior to discharge. The existing flow metering equipment appears to be functioning, although new level measuring equipment should be provided.

Currently the outfall remains operational and flows delivered to the ASB from Weyerhaeuser’s leachate collection system are discharged by gravity to the Pacific Ocean. Existing flows discharged from the stabilization basin average 0.5 MGD. During the dry season the discharge is supplemented by pumping ground water (purchased from the Coos Bay North Bend Water Board) through the system. Ground water is discharged through the system to maintain minimum flow through the diffusers in an effort to keep the ports operational and sand from inundating the diffuser ports.

7.1.1 Outfall Capacity Analysis

Preliminary head-loss calculations indicate that the capacity of the 18-inch line from the ASB to the effluent pump station is limiting. If the pipeline from the ASB were upsized to 24 –inches, the capacity of the inlet pipe from the ASB to the effluent pump station would be increased to greater than 7 MGD. The available head at the effluent pump station would not be sufficient to promote gravity flow beyond an estimated 3 MGD. Further upsizing of the outfall line does not appear practical; therefore, effluent pumping would need to be provided for peak events.

The addition of effluent pumping equipment allows discharge from the outfall under all operating conditions including during ocean storm surge events. Preliminary sizing of the required pumping equipment indicates 10 HP pumps should provide the necessary capacity. Two pumps should be provided for redundancy. Pumps should operate with VFDs to expand the systems operating range and to help realize power savings at lower flow conditions (flows above 3 MGD).

7.1.2 Diffusers

At a depth of 61.4 feet below mean lower low water (MLLW), the outfall pipe is connected to a bifurcated "Y" shaped diffuser. From the junction of the outfall "Y", each diffuser arm consists of a 252 foot long, 20 –inch diameter pipe equipped with 16 diffuser nozzles, each with 3-inch diameter ports. Each port is fitted with a flapper style check valve.

The condition of the diffusers is periodically assessed by divers. SHN obtained videos from Weyerhaeuser for review of a survey completed in 2005. This survey showed diffusers heavily encrusted with barnacles and starfish. The ports on the exposed diffuser nozzles were open and were passing flow. However there was a large section of pipe and diffusers, approximately 1/3 of the total diffuser length that was covered with sand. Flow or areas of bottom disturbance were not visible from the ports buried in sand.

Replacement of the flapper style check valves with nozzle style diffusers (RedValve™) is recommended. Nozzles will improve the discharge characteristics, both from an ecological and a maintenance perspective. Properly sized, nozzles tend to increase the discharge velocity, creating a jet which improves the initial mixing of the discharge with the surrounding water. Within this immediate zone of contact, (referred to as the zone of immediate dilution or ZID) dilution and rapid mixing occurs before the discharge fans out and forms a plume. Constituents causing acute toxicity concerns (such as ammonia) are permitted within the ZID, as long as the threshold criteria are not exceeded at the fringe of the ZID. Using nozzles to create a discharge jet increases the acceptability of the ZID and the initial dilution of toxins. In addition to reducing toxicity concerns, nozzles help to keep sand away from the diffuser port since the high discharge velocity scours and carries sand away from the port opening. A model of the diffuser should be conducted to verify the benefits of a nozzle upgrade and to confirm that toxicity at the fringe of the ZID is not a concern.

7.1.3 Effluent Pump Station Upgrade

For the basis of this feasibility analysis, it was assumed that the effluent pump station would be constructed using the existing outfall, wet-well, and flow metering equipment. Two vertical turbine pumps would be installed in the wetwell similar to the original configuration. Pumping equipment would be supplied 480 Volt power using the existing power feed supply. New pump controls, VFDs, and level metering equipment would be installed in the existing control building. As previously discussed, the section of 18-inch line from the ASB to the effluent pump station will need to be replaced with a 24-inch diameter or larger pipe. Outfall port check valves would also be replaced with the diffuser nozzles described above.

Table 7.1, Outfall Improvements Cost Estimate

Outfall					
Item No.	Description	Units	Quantity	Unit Cost	Total Cost
1	Bonds, Insurance, Mobilization & Permits (15%)	ls	all	$74,850.00	$74,850.00
2	Effluent Pump Station	ls	all	$90,000.00	$90,000.00
3	Replace Diffusers	ls	all	$75,000.00	$75,000.00
4	AC Pavement Patch in North Spit Pkwy.	ls	all	$10,000.00	$10,000.00
5	Secondary Effluent Pipe	lf	1800	$180.00	$324,000.00
		Construction Total			**$573,850.00**
		Contingency (15%)			$86,077.50
		Engineering (18%)			$103,293.00
		Administrative costs (3%)			$17,215.50
		Total Project Costs			**$780,436.00**

8.0 Project Summary

Based on the data gathered and presented in this report, the following is a summary of what, in our opinion, is the most feasible system should the City decide to further pursue the north spit as a treatment option in lieu of upgrading Wastewater Treatment Plant #2.

As calculated below, in Table 8.2.1, the approximate costs for transmission and treatment on the north spit are $18.7 million, and do not include any potential acquisition costs that may or may not be necessary.

Preliminary estimates for reconstruction of the existing treatment plant are approximately $16.2 million, including land acquisition costs. Potential political hurdles with construction on the available land include neighborhood concerns regarding odor, noise, and visual nuisances.

Although the calculations were not finalized for this report, it is reasonable to assume that there would be some O&M cost savings with the north spit option. More information is needed to determine the exact value and the present worth of those savings.

The North Spit option includes costs for several uncertain issues. The most notable cost item addresses the responsibility for sludge removal in the ASB. The cost estimate includes a $3 million placeholder to cover dredge and disposal costs. Additional items need to be resolved including a determination of wetland impacts and whether DEQ will require installation of an impermeable liner for the ASB.

8.1 Layout Summary

The preferred layout consists of a new pump station on or near the existing treatment plant site which would then pump the sewage through an 18" pipe under Coos Bay and to the north spit treatment site. Treatment on the north spit site would consist of an elevated headworks and a partially mixed lagoon system with two parallel primary treatment cells. Effluent would flow via gravity into a new chlorine contact chamber and then, with the help of a small pump station, be mixed with the Pacific Ocean at the end of the existing outfall. This treatment system has the flowing advantages when compared to a mechanical plant employing an activated sludge process and a discharge to Coos Bay.

- Natural "green system",
- Low operational complexity, and reduced operation costs,
- Long detention times allowing ammonia and total nitrogen removal,
- Significant reduction of biosolids produced (approximately 50 percent of what is currently produced),
- Potential for future expansion through additional aeration,
- Ocean disposal permit conditions, and

8.2 Costs

The estimated costs associated with the treatment system described in this report are summarized in Table 8.2 below:

Table 8.2, Cost Estimate Summary & Total

NORTH SPIT TREATMENT OPTION			
1	Pump Station (Single Force Main)	Section 4.2.1	$1,455,000.00
2	Forcemain (Single Force Main)	Section 4.1.1	$3,554,000.00
3	Headworks	Section 3.1	$1,109,000.00
4	Treatment (Preferred Secondary Alternative #5)	Section 5.1.5	$2,878,000.00
5	Disinfection	Section 6.1.2	$683,000.00
6	Backup Power (Pump Station)		$129,000.00
7	Backup Power (Treatment)		$42,000.00
8	Office and Laboratory		$440,000.00
9	Outfall	Section 7.1.3	$574,000.00
		Total Estimated Construction Costs	**$10,863,000.00**
		Recommended Contingency (20%)	$1,629,000.00
		Engineering Budget (15%)	$1,955,000.00
		Administrative Costs (3%)	$326,000.00
		Wetlands Deliniation	$60,000.00
		Secondary Lagoon Sludge Survey	$60,000.00
		Remove Sludge	$3,000,000.00
		Specialty Engineering , i.e. Geotech & Environmental (2%)	$217,000.00
		Total Recommended Project Budget	**$18,111,000.00**

9.0 References

Bureau of Land Management (2006). "Final North Spit Plan – An Update to the Coos Bay Shorelands Plan of 1995." NR: Bureau of Land Management.

CH2M Hill (1997). "Weyerhaeuser Demonstration Wetland Monitoring – June 1996 Sampling." NR: CH2M Hill.

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Sweet-Edwards/EMCON (1990). "Phase III Wastewater Lagoon Ground Water Investigation and Concentration Limit Variance Request." NR: Sweet-Edwards/EMCON.

U.S. Army Corps of Engineers (1981). "North Bay Marine Industrial Park, North Spit, Coos County, Oregon." NR: U.S. Army Corps of Engineers

Weyerhaeuser (1993). "Closure Plan for Wastewater Treatment impoundments at Weyerhaeuser Paper Company." NR: Weyerhaeuser.

Weyerhaeuser (2004). *National Pollutant Discharge Elimination System Permit Evaluation and Fact Sheet*. NR: Weyerhaeuser.

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Appendix A

Additional Mapping

COASTAL FLOOD ZONE


N
1411
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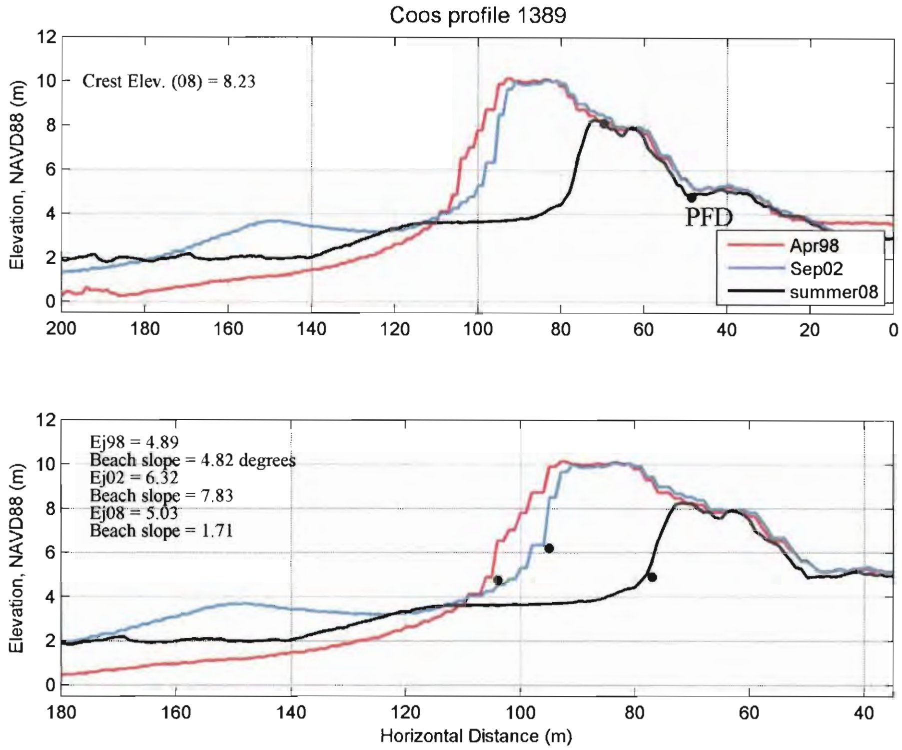
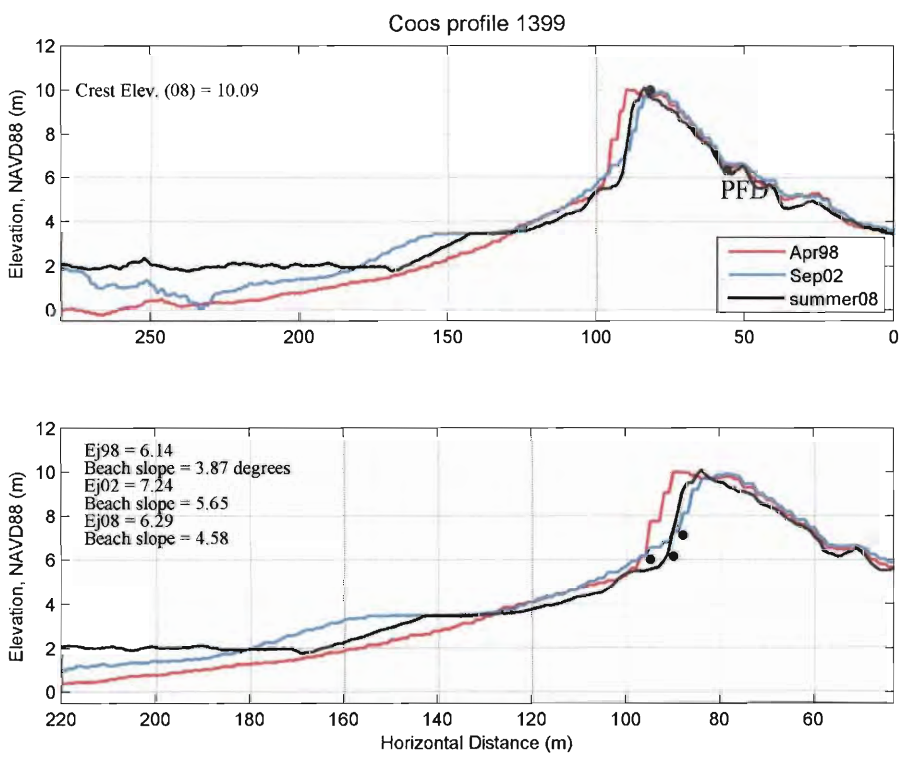
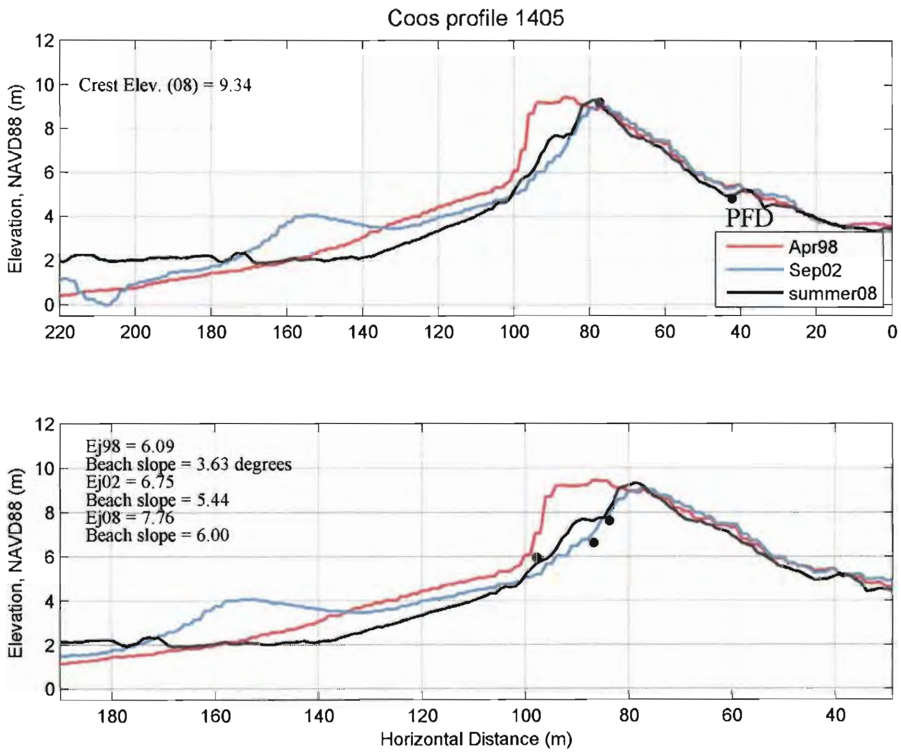
1387
1386
1385
1384
0 500 1,000 Feet
0 150 300 Meters



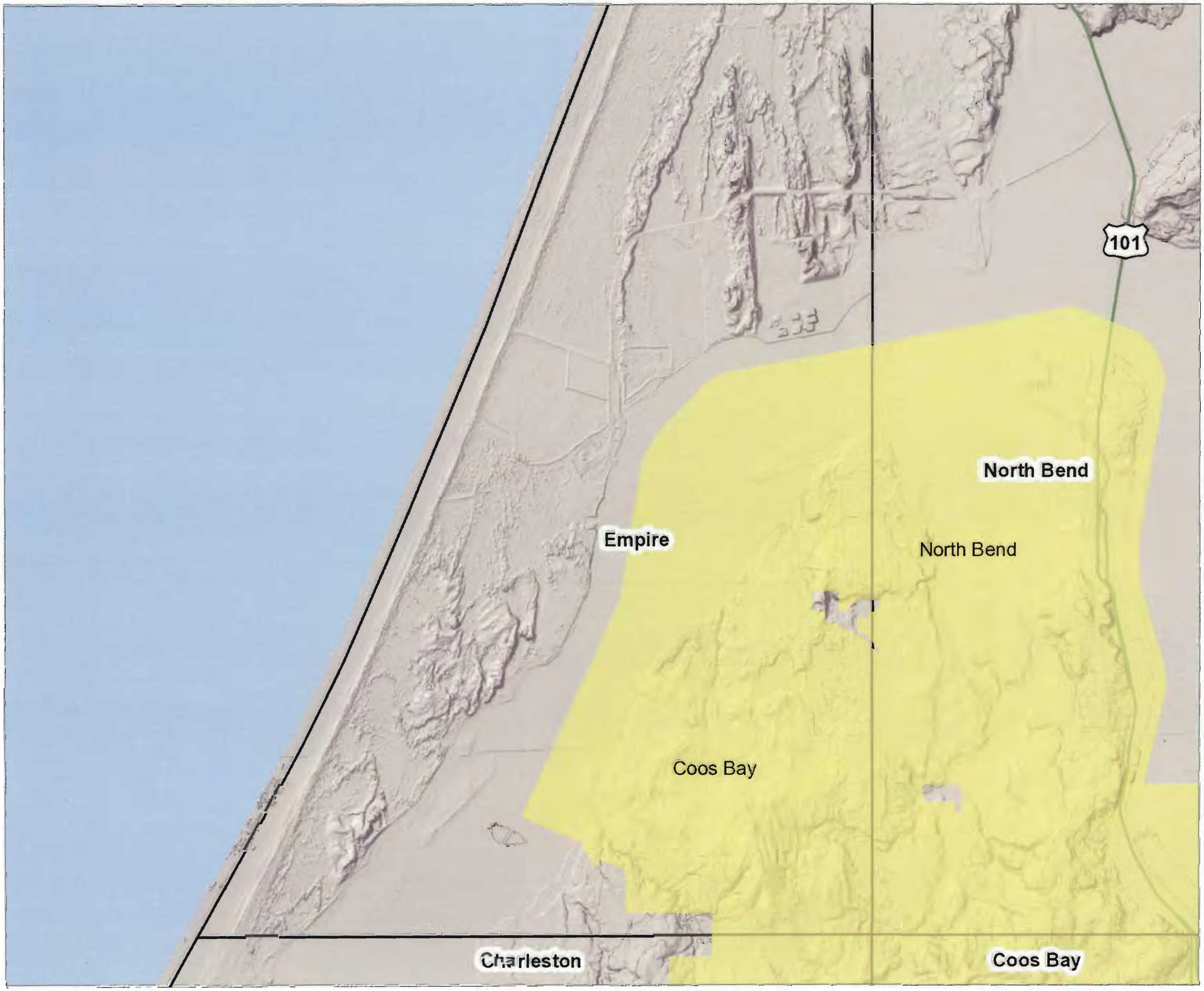
Coos profile 1389
Crest Elev. (08) = 8.23
PFD
Apr98
Sep02
summer08
Elevation, NAVD88 (m)
Ej98 = 4.89
Beach slope = 4.82 degrees
Ej02 = 6.32
Beach slope = 7.83
Ej08 = 5.03
Beach slope = 1.71
Horizontal Distance (m)



Coos profile 1399
Crest Elev. (08) = 10.09
PFD
Apr98
Sep02
summer08
Elevation, NAVD88 (m)
Ej98 = 6.14
Beach slope = 3.87 degrees
Ej02 = 7.24
Beach slope = 5.65
Ej08 = 6.29
Beach slope = 4.58
Horizontal Distance (m)


Coos profile 1405
Crest Elev. (08) = 9.34
PFD
Apr98
Sep02
summer08
Elevation, NAVD88 (m)
Ej98 = 6.09
Beach slope = 3.63 degrees
Ej02 = 6.75
Beach slope = 5.44
Ej08 = 7.76
Beach slope = 6.00
Horizontal Distance (m)

LIDAR DATA



DOGAMI Lidar Data Viewer
3/24/2010

Oregon Department of
Geology and Mineral Industries

ASSESSORS MAP

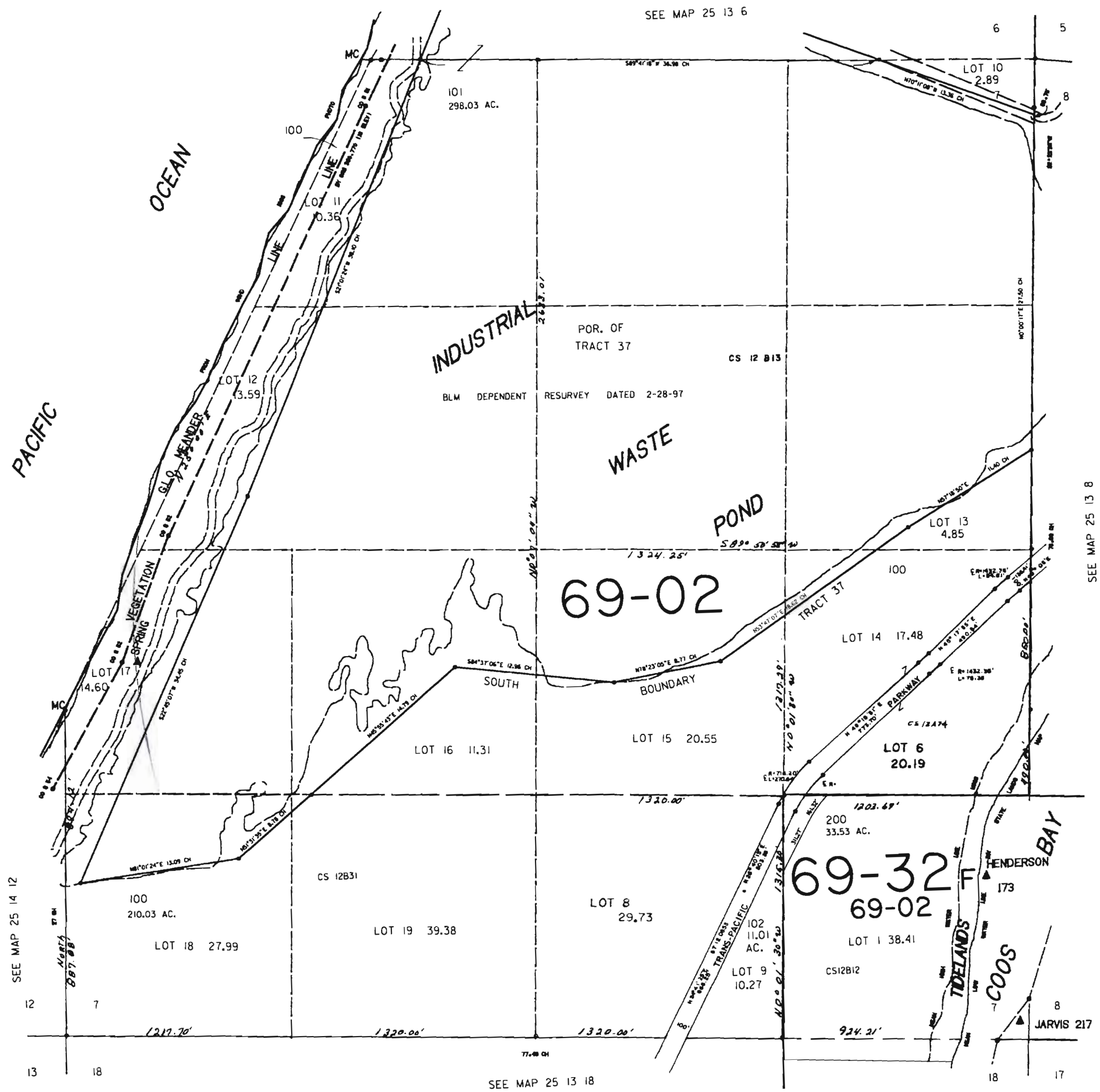
THIS MAP WAS PREPARED FOR ASSESSMENT PURPOSE ONLY

SECTION 7 T.25S. R.13W. W.M.
COOS COUNTY

1" = 400'

25 13 7

CANCELLED
100A1
105
106
103
102
104
201
202



4-26-2005

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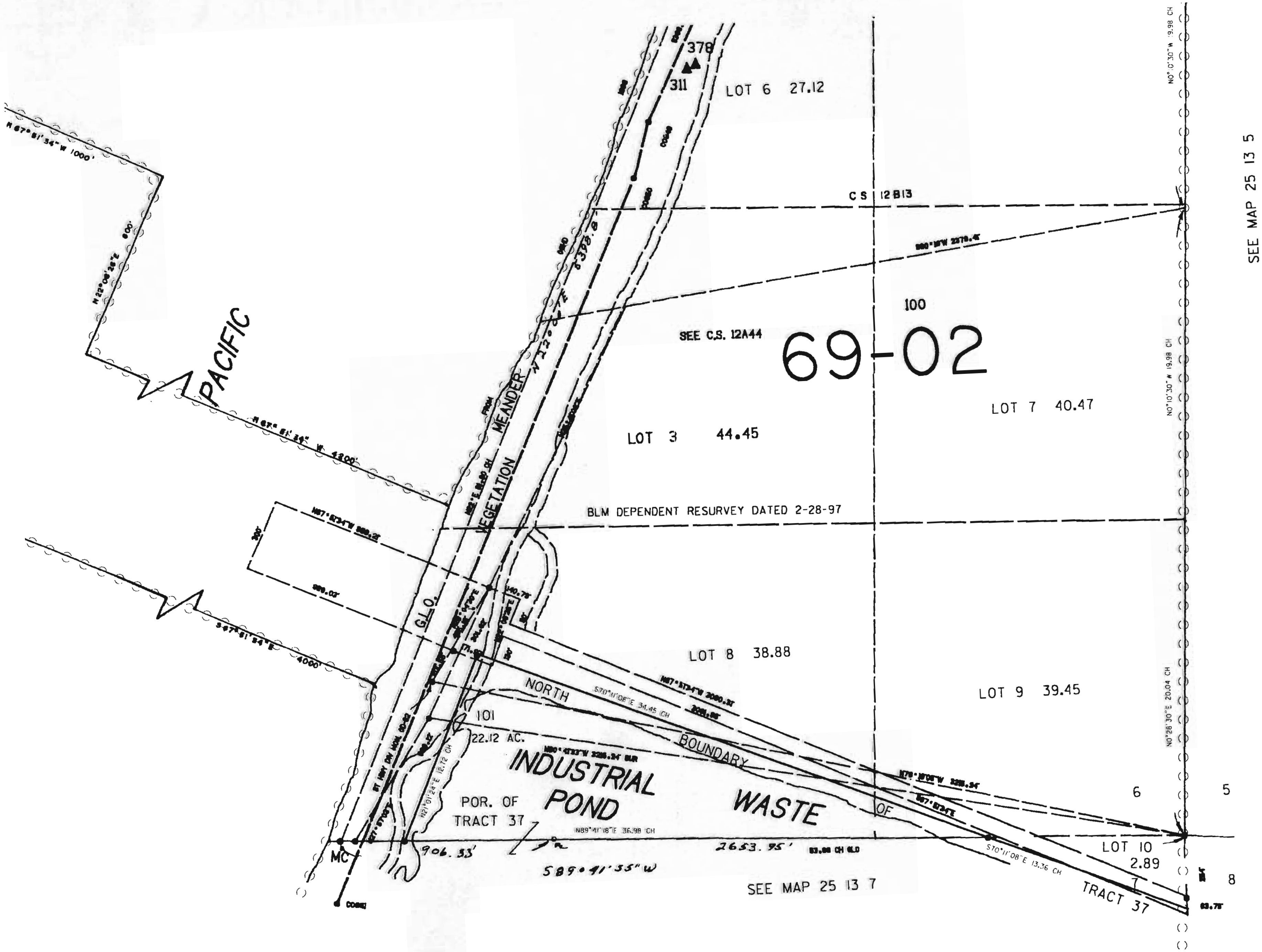


69-02
100
SEE MAP 25 13 5
C S 12B13
SEE C.S. 12A44
LOT 6 27.12
LOT 7 40.47
LOT 3 44.45
BLM DEPENDENT RESURVEY DATED 2-28-97
LOT 8 38.88
LOT 9 39.45
LOT 10
2.89
101
22.12 AC.
NORTH
BOUNDARY
OF
INDUSTRIAL
WASTE
POND
POR. OF
TRACT 37
TRACT 37
PACIFIC
MEANDER
VEGETATION
G.L.O.
MC
906.33'
2653.95'
S89°41'55"W
SEE MAP 25 13 7
6
5
8
N67°51'34"W 1000'
N22°08'26"E 900'
N67°51'34"W 4300'
S67°51'34"E 4000'
S70°11'08"E 34.45 CH
S70°11'08"E 13.36 CH
N21°01'24"E 12.72 CH
N89°41'18"E 36.98 CH
N0°28'30"E 20.04 CH
N0°10'30"W 19.98 CH
N0°0'30"W 19.98 CH

$DATE$

25 13 6

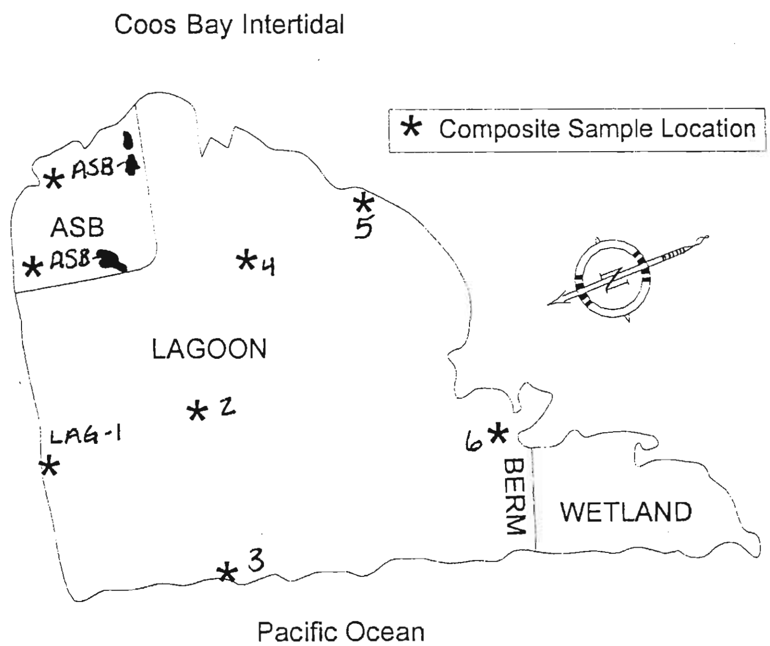
Appendix B

Solids Data

SLUDGE SAMPLING

ASB and Lagoon - Two composite samples will be collected from the Aerated Stabilization Basin, one each from the influent and effluent regions. Each composite sample will be obtained from four grab samples of sludges at different ASB depths in influent and effluent regions. The two sampling regions are designed to define sediment constituents of potentially varied settling rate materials. Six composite samples will be collected from the lagoon area historically used as a final settling pond; one from the proximity of each of five lagoon locations previously sampled (depths and shore line of central, eastern, and western lagoon) for ecological assessment and one adjacent to the berm defining the created wetland on the southern end of the lagoon. The six locations allow for definition of constituents at different depths and regions of the lagoon. (Figure 2) Placement of sampling locations in regions previously sampled for ecological assessment may allow for potential future analysis of rates of change in constituent levels.

Figure 2



3 Sampling Methods

Samples obtained from beneath water of a depth of 3 feet or greater will be collected using an Ekman dredge lowered from the side of a boat. In shallow locations, along shorelines, sludges will be sampled using a hand trowel. All samples will be obtained from the surface to depths of 3 to 6 inches. Multiple, individual samples will be homogenized in a stainless steel bowl prior to placement into the appropriate composite sample bottles for the analyses listed in Attachment 1.

Weyerhaeuser 2005 North Bend Lagoon Sludge

CN		NSB-1	NSB-2	SSB-1	SSB-2	ASB -2	LAG-1	LAG-3	LAG-5	LAG-6	Industrial PRG Standards	Residential PRG Standards	Oregon Department of Environmental Quality Level II Screening Level Values: Soils (mg/kg), Terrestrial Receptors: Plants	Inverts	Birds	Mammals	Sediment mg/kg: Freshwater	Marine	Bioaccumulation
mg/kg method AM 1-335.2	CN	<2	<2	<2	<2	3	<0.5	<0.5	<0.5	<0.5									

Metals		NSB-1	NSB-2	SSB-1	SSB-2	ASB -2	LAG-1	LAG-3	LAG-5	LAG-6	Industrial PRG Standards	Residential PRG Standards	Soils (mg/kg), Terrestrial Receptors: Plants	Inverts	Birds	Mammals	Sediment mg/kg: Freshwater	Marine	Bioaccumulation
mg/kg											mg/kg	mg/kg							
OD Basis	Ag	<1	2	1	2	3	<1	<1	<1	<1	5,100	390					4.5	0.7	
	Al	14600	31800	18400	31700	24700	2520	2640	5890	2460	100,000	76,000	50	600	450	107			
	As	<20	<20	<20	<20	<20	<20	<20	<20	<20	2	0	10	60	10	29	6	7	4
method	Ba	329	663	365	585	696	18.4	28.6	77.6	20.3	67,000	5,400	500	3000	85	638		48	
E-3050/	Be	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	1,900	150	10			83			122
E-6010	Co	7	12	5	11	8	1	2	5	1	1,900	900	20	1000		150			
	Cr	25	50	33	55	45	5	5	20	5	450	210	1	0.4	4	340000	37	52	42000
	Cu	58	185	119	218	216	<2	<2	32	2	41,000	3,100	100	50	190	390	36	19	10
	Fe	32000	31900	15500	23200	16500	3720	3020	10500	3190	100,000	23,000	10	200					
	Hg	<0.1	0.3	0.2	0.3	0.4	<0.1	<0.1	<0.1	<0.1	310	23	0.3	0.1	1.5	73	0.2	0.1	
	Mn	493	919	465	822	839	26.1	27.7	286	24.3	19,000	1,800	500	100	4125	11000	1100		
	Ni	20	39	24	42	33	5	<5	12	<5	20,000	1,600	30	200	320	625	18	16	316
	Pb	30	90	80	130	130	<10	<10	53	<10	800	400	50	500	16	4000	35	30	128
	Sb	<10	<10	<10	<10	<10	<10	<10	<10	<10	410	31	5			15	3	9	10
	Se	<50	<50	<50	<50	<50	<50	<50	<50	<50	5,100	390	1	70	2	25		1	0.1
	Sn	<5	11	6	11	13	<5	<5	<5	<5	100,000	47,000	50	2000					
	Sr	103	211	116	216	180	4.3	5.7	25.5	6.1	100,000	47,000				32875			
	Tl	<50	<50	<50	<50	<50	<50	<50	<50	<50	67	5	1			1			0.7
	V	35	74	43	75	57	6	5	13	5	1,000	78	2		47	25		57	
	Zn	389	1390	871	1490	1420	11	16	64	13	100,000	23,000	50	200	60	20000	123	124	3

? okay

RP Fluid		NSB-1	NSB-2	SSB-1	SSB-2	ASB -2	LAG-1	LAG-3	LAG-5	LAG-6	Industrial PRG Standards	Residential PRG Standards	Soils (mg/g), Terrestrial Receptors: Plants	Inverts	Birds	Mammals	Sediment mg/g: Freshwater	Marine	Bioaccumulation
mg/g											mg/g	mg/g							
	Propylene glycol	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	100	30							
	PPG	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001									

PCBs		NSB-1	NSB-2	SSB-1	SSB-2	ASB -2	LAG-1	LAG-3	LAG-5	LAG-6	Industrial PRG Standards	Residential PRG Standards	Soils (ug/kg), Terrestrial Receptors: Plants	Inverts	Birds	Mammals	Sediment ug/kg: Freshwater	Marine	Bioaccumulation
ug/kg											ug/kg	ug/kg							
	Aroclor-1016	<80	<160	<200	<250	<380	<33	<33	<48	<36	21,000	3,900				100000			0.42
	Aroclor-1221	<160	<330	<390	<500	<760	<66	<66	<96	<72	740	220							
method	Aroclor-1232	<80	<160	<200	<250	<380	<33	<33	<48	<36	740	220							
EPA 8082	Aroclor-1242	<80	<160	<200	<250	<380	<33	<33	<48	<36	740	220			1500	5000			0.002
	Aroclor-1248	<80	<160	<200	<250	<380	<33	<33	<48	<36	740	220							0.004
	Aroclor-1254	**59 J**	**120 JP**	**30 JP**	**78 JP**	**150 J**	<33	<33	**52**	**12 J**	740	220			700	4000			0.01
	Aroclor-1260	**32 JP**	**83 JP**	<200	**37 JP**	<380	<33	<33	**29 J**	<36	740	220							

NOTE: J indicates the concentration is below the quantitation limit and therefore is an estimate.
NOTE: P indicates more that 25% difference between the dual column quantitation.

	1,2,4-Trimethylbenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			170	52							
	sec-Butylbenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			220	220							
	1,3-Dichlorobenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			600	530						0.17	
	p-Isopropyltoluene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15											
	1,4-Dichlorobenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			7.9	3.4		20				0.11	
	1,2-Dichlorobenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			600	600						0.013	
	n-Butylbenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			240	240							
	1,2-Dibromo-3-chloropropane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			2	0.46							
	1,2,4-Trichlorobenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			220	62		20				0.005	

											Industrial	Residential	Soils (mg/kg)				Sediment		
											PRG	PRG	Terrestrial Receptors				mg/kg		
PAHS		NSB-1	NSB-2	SSB-1	SSB-2	ASB -2	LAG-1	LAG-3	LAG-5	LAG-6	**Standards**	**Standards**	**Plants**	**Inverts**	**Birds**	**Mammals**	**Freshwater**	**Marine**	**Bioaccumulation**
mg/kg	Hexachlorobutadiene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			220	6.2						0.001	
	Naphthalene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			190	56	10			3900	0.176	0.035	
	1,2,3-Trichlorobenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15						20					

											Industrial	Residential	Soils (mg/kg)				Sediment		
											PRG	PRG	Terrestrial Receptors				mg/kg		
DioxinFuran		NSB-1	NSB-2	SSB-1	SSB-2	ASB -2	LAG-1	LAG-3	LAG-5	LAG-6	**Standards**	**Standards**	**Plants**	**Inverts**	**Birds**	**Mammals**	**Freshwater**	**Marine**	**Bioaccumulation**
mg/kg (ppm)											mg/kg	mg/kg							
method	2,3,7,8-TCDD	4.40E-06	3.33E-05	1.27E-05	2.41E-06	3.93E-06	5.70E-07	2.80E-07	3.57E-06	ND(5.3E-07)	1.60E-05	3.9E-06					9.00E-06	4.00E-06	8.50E-07
1613A	2,3,7,8-TCDF	5.06E-05	3.12E-04	1.47E-04	3.26E-04	5.97E-04	NE(9.0E-07)	1.20E-06	3.28E-05	ND(6.3E-07)			6.00E+02						
EPA	TEQ	32.7	199	126	691	309	0.43	1.84	33.5	0.44	*38.4 ppt*	*4 ppt*							

NOTE: *the results for cis-1,3-Dichloropropene and trans-Dichloropropene were combined into 1,3-Dichloropropene
NOTE: ** indicates that the PRG value for "Xylenes" as a collective unit was used as the standard.
M,p-xylene and o-xylene were not summed because of differences in the Oregon DEQ Screening Level Values
NOTE: J indicates the concentration is below the quantitation limit and therefore is an estimate.
NOTE: P indicates more that 25% difference between the dual column quantitation.
NOTE: DioxinFuran - A detection limit was calculated from 2.5 times the signal in the area of the elution of 13C12-2,3,7,8-TCDD (TCDF) whenever a sample contained no detectable TCDD (TCDF). This value is reported in parenthesis.
NOTE: TEQ standard calculated based on EPA guidance regarding dioxin in soils at CERCLA and RCRA sites

value or detection limit exceeds Industrial and Residential PRG standard
value or detection limit exceeds Residential PRG standard
value or detection limit exceeds Residential PRG standard

Value changed 4/20/05 and still above PRG
Value changed 4/20/05
Value changed 4/20/05 and below PRG because of data review.

PAHS		NSB-1	NSB-2	SSB-1	SSB-2	ASB -2	LAG-1	LAG-3	LAG-5	LAG-6	Industrial PRG Standards	Residential PRG Standards	Soils (mg/kg) Terrestrial Receptors Plants	Inverts	Birds	Mammals	Sediment mg/kg Freshwater	Marine	Bioaccumulation
mg/kg											mg/kg	mg/kg							
	Dichlorodifluoromethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			310	94							
method	Chloromethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			160	47							
EPA 8260	Vinyl chloride	< 0.73	0.36	< 0.71	0.19	< 1.2	< 0.15	< 0.15			1	0				20			0.03
	Bromomethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			13	4							
	Chloroethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			7	3							
	Trichlorofluoromethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			2,000	390							
	1,1-Dichloroethene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			410	120							
	Acetone	< 7.3	< 18	< 7.1	< 9.7	< 12	< 1.5	< 1.5			54,000	14,000				1250			0.29
	Acrolein	< 7.3	< 18	< 7.1	< 9.7	< 12	< 1.5	< 1.5			0	0							
	Carbon disulfide	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			720	360							
	Methylene chloride	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			21	9				730			0.93
	trans-1,2-Dichloroethene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			230	69				2500			
	Methyl-t-butyl ether	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			70	32							
	1,1-Dichloroethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			1,700	510							
	2,2-Dichloropropane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15											
	cis-1,2-Dichloroethene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			150	43				2500			
	2-Butanone	< 3.6	< 9.0	< 3.6	< 4.8	< 6.0	< 0.80	< 0.80			110,000	20,000							
	Bromochloromethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15											
	Chloroform	< 0.22	< 0.64	< 0.21	< 0.29	< 0.36	< 0.15	< 0.15			0	0				1875			3.66
	1,1,1-Trichloroethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			1,200	1,200				55550			1800
	Carbon tetrachloride	< 0.25	<0.61	< 0.24	< 0.33	< 0.41	< 0.15	< 0.15			1	0		1000		2000			6.08
	1,1-Dichloropropene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15											
	Benzene	< 0.73	[illegible]	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			1	1				3300			3.92
	1,2-Dichloroethane	< 0.20	< 0.50	< 0.20	< 0.27	< 0.34	< 0.15	< 0.15			1	0			70	2780			3.43
	Trichloroethene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15											
	1,2-Dichloropropane	< 0.73	< 0.56	< 0.71	< 0.31	< 0.38	< 0.15	< 0.15			1	0		700					
	Dibromomethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			230	67							
	Bromodichloromethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			2	1							
	1,3-Dichloropropene*	<1.5	<1.2	<1.4	<0.62	<0.77	<0.30	<0.30			2	1							
	4-Methyl-2-pentanone	< 3.6	< 9.0	< 3.6	< 4.8	< 6.0	< 0.80	< 0.80			47,000								3.81
	Toluene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			520	520	200			1440			5.3

PAHS		NSB-1	NSB-2	SSB-1	SSB-2	ASB -2	LAG-1	LAG-3	LAG-5	LAG-6	Industrial PRG Standards	Residential PRG Standards	Soils (mg/kg) Terrestrial Receptors Plants	Inverts	Birds	Mammals	Sediment mg/kg Freshwater	Marine	Bioaccumulation
mg/kg	1,1,2-Trichloroethane	< 0.73	< 0.79	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			1.6	0.73				55550			
	Tetrachloroethene	< 0.73	< 0.55	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			1.3	0.48							
	1,3-Dichloropropane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			360	100							
	2-Hexanone	< 3.6	< 9.0	< 3.6	< 4.8	< 6.0	< 0.80	< 0.80											
	Dibromochloromethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			2.6	1.1							
	1,2-Dibromoethane	< 0.23	<0.58	< 0.23	< 0.31	<0.38	< 0.05	< 0.05			0.073	0.032							
	Chlorobenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			530	150		40					
	1,1,1,2-Tetrachloroethane	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			7.3	3.2							
	Ethylbenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			400	400						0.004	
	m,p-Xylene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			420**	270							
	o-Xylene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			420**	270	1						
	Styrene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			1700	1700	300						
	Bromoform	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			220	62							
	Isopropylbenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			2000	570							
	Bromobenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			92	28							
	1,2,3-Trichloropropane	< 0.23	<0.58	< 0.23	< 0.31	<0.38	< 0.05	< 0.05			0.076	0.034							
	1,1,2,2-Tetrachloroethane	< 0.73	< 0.56	< 0.71	< 0.45	< 0.48	< 0.15	< 0.15			0.93	0.41							
	n-Propylbenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			240	240							
	2-Chlorotoluene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			560	160							
	4-Chlorotoluene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15											
	1,3,5-Trimethylbenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			70	21							
	tert-Butylbenzene	< 0.73	< 1.8	< 0.71	< 0.97	< 1.2	< 0.15	< 0.15			390	390							

PAHS		NSB-1	NSB-2	SSB-1	SSB-2	ASB -2	LAG-1	LAG-3	LAG-5	LAG-6	Industrial PRG Standards	Residential PRG Standards	Soils (ug/kg) Terrestrial Receptors Plants	Inverts	Birds	Mammals	Sediment ug/kg Freshwater	Marine	Bioaccumulation
ug/kg											ug/kg	ug/kg							
	Naphthalene	730	3600	1300	5800	5000	<1.6	5.5	120	1.8	190,000	56,000					0.176	0.035	
	2-Methylnaphthalene	210	1000	200	460	620	<1.6	3.6	80	<1.8								0.02	
method	Acenaphthylene	210	1200	370	960	1700	<1.6	1.7	34	<1.8							0.16	0.006	
8270-PAHS	Acenaphthene	68	340	54	1700	260	<1.6	<1.7	13	<1.8	29,000,000	3,700,000	20000				0.29	0.007	
	Dibenzofuran	81	340	110	680	590	<1.6	1.6 J	36	<1.8	1,600,000	150,000				2	5.1	0.11	
	Fluorene	84	400	100	1200	510	<1.6	<1.7	16	<1.8	26,000,000	3,700,000		30000			0.077	0.021	
	Phenanthrene	250	1100	480	2400	2000	<1.6	8.6	180	2.8							0.042	0.086	
	Anthracene	49	180	47	380	200	<1.6	<1.7	16	<1.8	100,000,000	22,000,000					0.057	0.047	
	Fluoranthene	330	950	380	1400	1600	1.6	11	220	3.4	22,000,000	2,300,000					0.111	0.113	
	Pyrene	190	860	250	900	1100	1.5 J	10	200	3.2	29,000,000	2,300,000					0.053	0.152	
	Benzo(a)anthracene	<18	<110	<33	<110	65	<1.6	1.5 J	17	<1.8	2,100	620					0.032	0.075	
	Chrysene	<18	<110	<33	170	92	<1.6	1.7	30	<1.8	210,000	62,000					0.057	0.107	
	Benzo(b)fluoranthene	38	<110	<33	290	130	<1.6	3.4	37	<1.8	2,100	620						1.8	
	Benzo(k)fluoranthene	19	<110	<33	120	82	<1.6	1.7 J	21	<1.8	21,000	6,200					0.027	1.8	
	Benzo(a)pyrene	36	140	68	340	120	<1.6	2.6	14	<1.8	210	62				125000	0.032	0.089	0.1
	Indeno(1,2,3-cd)pyrene	25	140	41	230	93	<1.6	3.0	14	<1.8	2,100	620					0.017	0.6	
	Dibenzo(a,h)anthracene	<18	<110	<33	<110	<56	<1.6	<1.7	<2.5	<1.8	210	62					0.033	0.006	
	Benzo(g,h,i)perylene	42	170	61	230	160	<1.6	2.6	19	<1.8							0.3	0.67	

Weyerhaeuser Analytical & Testing Services
32901 Weyerhaeuser Way South
Federal Way, WA 98003

Service Request 05-1056

Plant

Report

North Bend Lagoon Sludge - Dioxin

Compound Name	Blank 5/17/2005 (ppt)	NSB-1 1/17/05 0949 05-1056-001 (ppt)	NSB-2 1/17/05 1015 05-1056-002 (ppt)	SSB-1 1/17/05 1050 05-1056-003 (ppt)	SSB-2 1/17/05 1034 05-1056-004 (ppt)	ASB-2 1/17/05 1127 05-1056-005 (ppt)	LAG-1 1/17/05 1140 05-1056-006 (ppt)
TCDD	#######	98.0	1080	519	604	1330	ND(0.63)
2,3,7,8-TCDD	#######	5.43	48.1	23.9	32.4	54.9	ND(0.63)
PCDD	#######	68.5	761	367	537	936	ND(0.65)
1,2,3,7,8-PCDD	#######	6.82	56.8	30.0	55.4	70.3	ND(0.65)
HxCDD	#######	111	894	381	3360	1080	11.9
1,2,3,4,7,8-HxCDI	#######	6.43	52.9	19.6	79.7	48.0	ND(0.80)
1,2,3,6,7,8-HxCDI	#######	11.2	85.1	53.6	932	92.9	1.43
1,2,3,7,8,9-HxCDI	#######	12.6	106	48.2	189	96.5	1.14
HpCDD	#######	260	1630	790	32400	1320	20.4
1,2,3,4,6,7,8-HpCI	#######	152	936	440	18000	722	9.80
OCDD	#######	1210	6690	4050	152000	6140	67.0
TCDF	#######	359	3290	1500	2490	4350	ND(0.56)
2,3,7,8-TCDF	#######	61.2	560	234	421	693	ND(0.56)
PCDF	#######	152	1150	552	1780	1650	ND(0.41)
1,2,3,7,8-PCDF	#######	10.7	75.5	27.4	125	108	ND(0.41)
2,3,4,7,8-PCDF	#######	15.7	ND(8.6)	62.4	153	162	ND(0.41)
HxCDF	#######	87.1	699	242	3300	686	ND(0.51)
1,2,3,4,7,8-HxCDF	#######	12.5	111	41.1	209	106	ND(0.51)
1,2,3,6,7,8-HxCDF	#######	8.83	ND(11.0)	28.4	107	71.5	ND(0.51)
2,3,4,6,7,8-HxCDF	#######	7.31	65.5	23.3	86.3	52.2	ND(0.51)
1,2,3,7,8,9-HxCDF	#######	ND(1.7)	ND(11.0)	ND(8.6)	20.8	12.4	ND(0.51)
HpCDF	#######	123	827	172	2950	344	1.99
1,2,3,4,6,7,8-HpCI	#######	52.0	359	76.9	1010	140	0.94
1,2,3,4,7,8,9-HpCI	#######	4.39	28.5	7.24	53.1	24.8	ND(0.53)
OCDF	#######	136	973	139	644	324	1.93
EPA TEQ	0.00	32.7	199	126	691	309	0.43

Approved: Randy Eatherton
Telephone: (253) 924-6431

Date: 6/23/2005

EPA: 1613B

Weyerhaeuser Analytical & Testing Services
32901 Weyerhaeuser Way South
Federal Way, WA 98003

Service Request 05-1056

Report

North Bend Lagoon Sludge - Dioxin

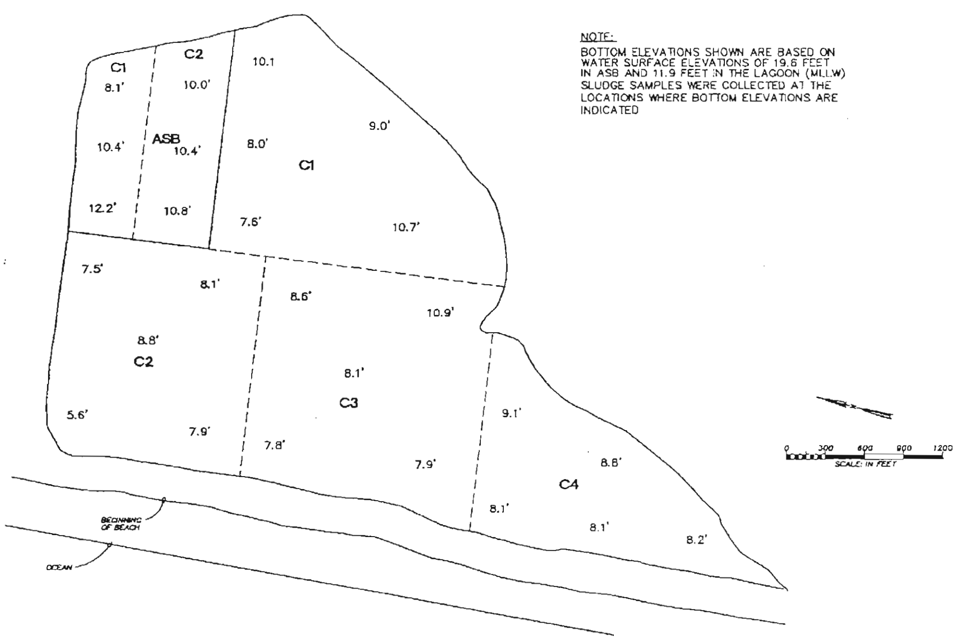
Compound Name	Blank 3/16/2005 (ppt)	LAG-3 1/17/05 1200 05-1056-007 (ppt)	LAG-5 1/17/05 1245 05-1056-008 (ppt)	ASB-D 2/14/05 1325 05-1056-009 (ppt)	LAG-6 1/17/2005 05-1056-010 (ppt)	WL-1 2/14/2005 05-1056-011 (ppt)
TCDD	#######	**3.42**	**184**	**850**	**0.52**	ND(0.38)
2,3,7,8-TCDD	#######	**0.35**	**3.19**	**30.4**	ND(0.46)	ND(0.38)
PCDD	#######	**5.51**	**197**	**676**	ND(0.69)	ND(0.28)
1,2,3,7,8-PCDD	#######	ND(0.43)	**10.4**	**44.1**	ND(0.69)	ND(0.28)
HxCDD	#######	**31.6**	**660**	**1340**	**11.6**	**1.54**
1,2,3,4,7,8-HxCDI	#######	ND(0.97)	**8.52**	**31.3**	**0.13**	ND(0.23)
1,2,3,6,7,8-HxCDI	#######	ND(0.97)	**74.7**	**176**	**1.55**	**0.32**
1,2,3,7,8,9-HxCDI	#######	**3.67**	**40.3**	**97.8**	**1.12**	**0.42**
HpCDD	#######	**49.7**	**866**	**2600**	**21.0**	**14.9**
1,2,3,4,6,7,8-HpCI	#######	**23.6**	**417**	**1620**	**8.96**	**8.45**
OCDD	**1.19**	**173**	**2670**	**16300**	**57.6**	**60.3**
TCDF	#######	**8.05**	**211**	**2610**	ND(0.59)	ND(0.26)
2,3,7,8-TCDF	#######	**1.21**	**31.1**	**408**	ND(0.59)	ND(0.26)
PCDF	#######	**2.99**	**101**	**908**	ND(0.36)	**0.49**
1,2,3,7,8-PCDF	#######	ND(0.28)	**5.37**	**57.5**	ND(0.36)	ND(0.18)
2,3,4,7,8-PCDF	#######	**0.43**	ND(0.6)	**92.5**	ND(0.36)	ND(0.18)
HxCDF	#######	**3.74**	**96.8**	**444**	ND(0.49)	**1.44**
1,2,3,4,7,8-HxCDF	#######	**0.47**	**8.65**	**66.5**	ND(0.49)	ND(0.49)
1,2,3,6,7,8-HxCDF	#######	**0.34**	**6.26**	**37.9**	ND(0.49)	ND(0.49)
2,3,4,6,7,8-HxCDF	#######	**0.29**	**4.84**	**34.2**	ND(0.49)	ND(0.49)
1,2,3,7,8,9-HxCDF	#######	ND(0.25)	**0.67**	**6.07**	ND(0.49)	ND(0.49)
HpCDF	#######	**4.79**	**92.4**	**700**	**1.73**	**5.76**
1,2,3,4,6,7,8-HpCI	#######	**1.93**	**36.1**	**196**	**0.85**	**2.64**
1,2,3,4,7,8,9-HpCI	#######	ND(0.78)	**4.01**	**25.8**	ND(0.47)	ND(0.36)
OCDF	#######	**4.63**	**64.5**	**930**	**1.25**	**6.81**
EPA TEQ	**0.00**	**1.84**	**33.5**	**223**	**0.44**	**0.25**

Approved: Randy Eatherton
Telephone: (253) 924-6431

Date: 6/23/2005

EPA: 1613B

SLUDGE INVENTORY



NOTE:

BOTTOM ELEVATIONS SHOWN ARE BASED ON WATER SURFACE ELEVATIONS OF 19.6 FEET IN ASB AND 11.9 FEET IN THE LAGOON (MLLW) SLUDGE SAMPLES WERE COLLECTED AT THE LOCATIONS WHERE BOTTOM ELEVATIONS ARE INDICATED

FIGURE 2
LAGOON BOTTOM ELEVATIONS

CH2M HILL

REUSE OF DOCUMENTS

NORTH BEND, OREGON

Weyerhaeuser

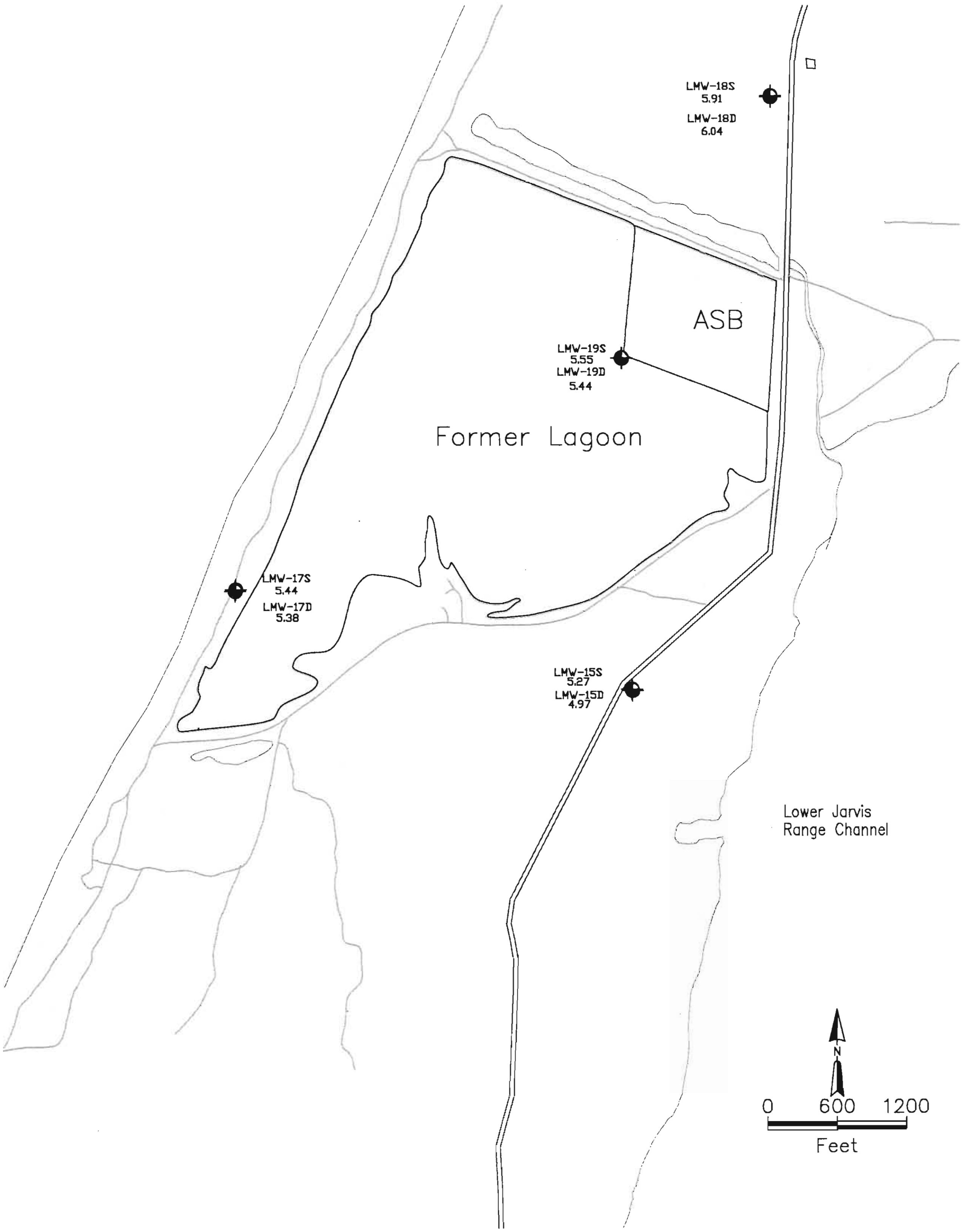
ASB AND LAGOON
SLUDGE SAMPLING LOCATIONS

PRELIMINARY

Table 1 Weyerhaeuser Lagoon Closure Sludge Volume and Weight Data Summary											
Section	Avg. Pond Depth (ft.)	Avg. Sludge Depth (ft.)	Section Area (sf)	In-Place Wet. Vol. (cf)	H_20 (%) wt.	Solids (%) wt.	Wet Dens. (lb/cf)	Sludge Dry Wt. (tons)	Approx. Dewater % Solids (%)	Approx. Dewater Wt. (tons)	Approx. Dewater Volume (cy)
ASB	9.29	3.28	1,646,000	5,399,000	88.8	11.2	72.7	22,000	30	73,300	74,700
C1	2.85	0.80	3,172,000	2,538,000	72.2	27.8	70	24,700	30	82,300	87,100
C2	4.33	0.87	2,681,000	2,332,000	89.7	10.3	70	8,400	30	28,000	29,600
C3	3.23	0.23	2,980,000	685,000	88.8	11.2	70	2,700	30	9,000	9,500
C4	3.43	0.83	2,118,000	1,758,000	90.8	9.24	70	5,700	30	19,000	20,100
Totals								63,500		211,600	221,000

Appendix C

Groundwater Data



Weyerhaeuser	Title: ASB Area Wells Groundwater Elevation March 2005	Drawn:	Designed:	Project No.:
		Checked:	Approved:	
		Date:	Revised:	Figure No. 6

Figure 7.
ASB-Area Groundwater Elevation Hydrograph

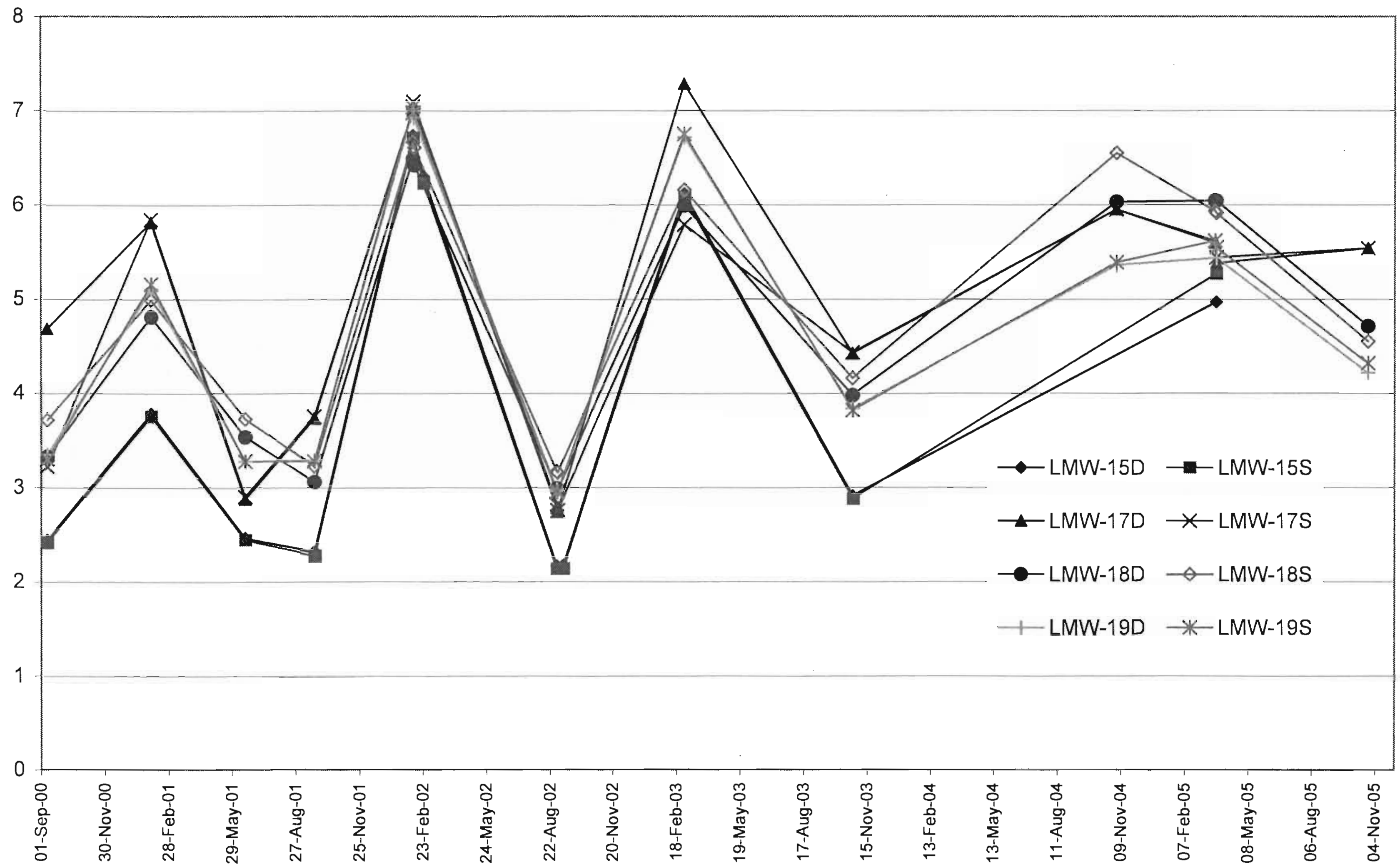


Table A-1
North Bend Well Completion

Site ID	Completion Date	Easting (X) NAD83 (ft)	Northing (Y) NAD83 (ft)	Borehole Depth (ft)	Well Depth (ft)	Surface Elevation (ft)	TOC Elevation (ft)	Screen Interval (ft)	Notes
LF-02	12-Jan-82	3927152.3	667187.8	30.0	17.5	15.1	17.80	7.5 - 17.5	Decommissioned 10/31/05
LF-02R	31-Oct-05	3927164.8	667190.3	18.0	17.0	14.6	17.07	7.0 - 17.0	Drilled to replace LF-02
LF-04A	13-Jan-82	3927558.9	667164.5	28.5	9.0	11.3	13.81	4.0 - 9.0	
LF-04B	13-Jan-82	3927559.9	667164.1	28.5	19.0	11.3	13.71	14.0 - 19.0	
LF-05R	05-Mar-91	3926885.4	666417.5	30.0	27.0	11.1	12.83	17.0 - 27.0	
MW-13	10-Nov-93	3926508.0	666385.2	28.0	22.5	9.6	11.66	11.6 - 21.6	
MW-16A	23-Oct-96	3927275.7	666561.1	13.0	8.0	11.5	13.62	3.2 - 8.0	
MW-16B	24-Oct-96	3927279.5	666565.3	25.0	25.0	11.6	13.51	15.0 - 25.0	
MW-17	24-Oct-96	3927941.0	667602.0	20.0	20.0	14.2	16.31	10.0 - 20.0	Decommissioned 2/1/06
LMW-15D	01-Sep-90	3916054.3	659180.9	60.0	51.0	10.6	12.43	41.0 - 51.0	Repaired 10/31/05
LMW-15S	01-Sep-90	3916048.3	659193.2	35.0	33.0	10.5	13.02	23.0 - 33.0	Repaired 10/31/05
LMW-17D	01-Sep-90	3912631.1	660054.2	60.0	52.0	15.8	17.28	42.0 - 52.0	
LMW-17S	01-Sep-90	3912625.8	660038.6	40.0	33.0	14.7	16.54	23.0 - 33.0	
LMW-18D	26-May-94	3917242.1	664283.2	51.0	50.7	8.4	10.74	40.5 - 50.1	
LMW-18S	25-May-94	3917242.4	664291.9	24.0	21.3	8.9	11.37	11.1 - 20.7	
LMW-19D	25-Sep-97	3915961.3	662031.6	60.0	60.0	13.0	14.65	50.0 - 60.0	
LMW-19S	23-Sep-97	3915957.9	662036.9	37.0	37.0	13.0	15.74	27.0 - 37.0	

All wells constructed of 2" diameter Schedule 40 PVC pipe and screen with 0.010" slots (0.020" slots MW-16A, MW-16B, MW-17)

LF-02R, LMW-15, LMW-18, LMW-19 surveyed by Stuntzner Engineering 2/22/2006

Table A-2
North Bend Containerboard Mill
Groundwater Elevation Data, Mill Area
2004-2005

PERIOD: From 03/17/2004 thru 02/01/2006 - Inclusive

SITE	DATE	MP ELEVATION (feet)	TIME	DEPTH TO WATER (feet)	DELTA WATER ELEV (feet)	WATER ELEV. (feet)
LF-02	3/17/2004	17.80	11:54	8.44	NA	9.36
LF-04A	3/17/2004	13.81	12:33	3.67	NA	10.14
LF-05R	3/17/2004	12.83	11:12	7.52	NA	5.31
MW-13	3/17/2004	11.66	09:25	5.98	NA	5.68
MW-16B	3/17/2004	13.51	10:31	7.67	NA	5.84
MW-17	3/17/2004	16.31	13:08	3.45	NA	12.86
LF-02	11/3/2004	17.80	09:48	9.70	-1.26	8.10
LF-04A	11/3/2004	13.81	09:28	3.62	0.05	10.19
LF-05R	11/3/2004	12.83	10:41	8.69	-1.17	4.14
MW-13	11/3/2004	11.66	11:10	7.34	-1.36	4.32
MW-16B	11/3/2004	13.51	10:10	8.54	-0.87	4.97
MW-17	11/3/2004	16.31	11:41	5.56	-2.11	10.75
LF-02	3/22/2005	17.80	14:00	9.45	0.25	8.35
LF-04A	3/22/2005	13.81	14:05	3.48	0.14	10.33
LF-04B	3/22/2005	13.71	14:08	6.77	NA	6.94
LF-05R	3/22/2005	12.83	13:50	8.33	0.36	4.50
MW-13	3/22/2005	11.66	13:54	6.79	0.55	4.87
MW-16A	3/22/2005	13.62	13:43	8.05	NA	5.57
MW-16B	3/22/2005	13.51	13:41	8.10	0.44	5.41
MW-17	3/22/2005	16.31	14:14	4.75	0.81	11.56
LF-02	3/24/2005	17.80	12:05	9.50	-0.05	8.30
LF-04A	3/24/2005	13.81	12:00	3.65	-0.17	10.16
LF-04B	3/24/2005	13.71	12:03	6.71	0.06	7.00
LF-05R	3/24/2005	12.83	12:14	7.63	0.70	5.20
MW-13	3/24/2005	11.66	12:17	6.46	0.33	5.20
MW-16A	3/24/2005	13.62	12:07	7.79	0.26	5.83
MW-16B	3/24/2005	13.51	12:10	8.00	0.10	5.51
MW-17	3/24/2005	16.31	10:00	5.00	-0.25	11.31
LF-04A	10/25/2005	13.81	13:46	4.90	-1.25	8.91
LF-05R	10/25/2005	12.83	11:03	8.78	-1.15	4.05
MW-13	10/25/2005	11.66	11:50	7.63	-1.17	4.03
MW-16B	10/25/2005	13.51	12:31	8.80	-0.80	4.71
MW-17	10/25/2005	16.31	14:30	7.18	-2.18	9.13
LF-02	10/31/2005	0.00	12:00	NM	NA	NA
MW-17	2/1/2006	0.00	12:00	NM	NA	NA

Table A-3
North Bend Containerboard Mill
Groundwater Elevation Data, ASB Area
2004-2005

PERIOD: From 11/03/2004 thru 10/25/2005 - Inclusive

SITE	DATE	MP ELEVATION (feet)	TIME	DEPTH TO WATER (feet)	DELTA WATER ELEV (feet)	WATER ELEV. (feet)
LMW-17D	11/3/2004	17.28	13:55	11.33	NA	5.95
LMW-17S	11/3/2004	16.54	13:33	10.59	NA	5.95
LMW-18D	11/3/2004	10.42	12:28	4.39	NA	6.03
LMW-18S	11/3/2004	11.42	13:37	4.87	NA	6.55
LMW-19D	11/3/2004	14.84	09:48	9.48	NA	5.36
LMW-19S	11/3/2004	14.78	15:02	9.39	NA	5.39
LMW-18D	11/24/2004	10.42	00:00	6.21	-1.82	4.21
LMW-18S	11/24/2004	11.42	00:00	6.04	-1.17	5.38
LMW-19D	12/11/2004	14.84	00:00	8.91	0.57	5.93
LMW-19S	12/11/2004	14.78	00:00	8.87	0.52	5.91
LMW-17D	3/22/2005	17.28	14:53	11.68	-0.35	5.60
LMW-17S	3/22/2005	16.54	14:50	10.92	-0.33	5.62
LMW-18D	3/22/2005	10.74	14:30	4.70	1.83	6.04
LMW-18S	3/22/2005	11.37	14:26	5.44	0.55	5.93
LMW-19S	3/22/2005	15.74	15:20	10.12	-0.29	5.62
LMW-15D	3/24/2005	12.43	11:52	7.46	NA	4.97
LMW-15S	3/24/2005	13.02	11:48	7.75	NA	5.27
LMW-17D	3/24/2005	17.28	11:30	11.90	-0.22	5.38
LMW-17S	3/24/2005	16.54	10:42	11.10	-0.18	5.44
LMW-18D	3/24/2005	10.74	10:10	4.70	0.00	6.04
LMW-18S	3/24/2005	11.37	10:12	5.46	-0.02	5.91
LMW-19D	3/24/2005	14.65	10:20	9.21	-0.49	5.44
LMW-19S	3/24/2005	15.74	10:25	10.19	-0.07	5.55
LMW-17D	10/25/2005	17.28	15:40	11.74	0.16	5.54
LMW-17S	10/25/2005	16.54	15:32	11.00	0.10	5.54
LMW-18D	10/25/2005	10.74	18:12	6.03	-1.33	4.71
LMW-18S	10/25/2005	11.37	18:40	6.82	-1.36	4.55
LMW-19D	10/25/2005	14.65	19:51	10.43	-1.22	4.22
LMW-19S	10/25/2005	15.74	20:07	11.42	-1.23	4.32

Table B-1
North Bend Mill
Groundwater Results

PERIOD: From 03/23/2005 thru 10/25/2005 - Inclusive
SAMPLE TYPE: Water

SITE	DATE	RESULT TYPE	Barium (mg/l)	Chromium (mg/l)	Iron (mg/l)	Manganese (mg/l)	Strontium (mg/l)
Oregon GroundWater Ref LvlOAR 340-40-020			1.0	0.05	0.3	0.05	
US Primary MCL			2	0.1			
US Secondary MCL					0.3	0.05	
LF-02	03/23/2005	Prim	0.012	---	[15.8]	[0.899]	0.42
LF-02	03/23/2005	Dup 1	0.012	---	[15.8]	[0.895]	0.417
LF-04A	03/23/2005	Prim	0.019	---	[17.9]	[1.63]	0.337
LF-04A	03/23/2005	Dup 1	---	---	---	---	---
LF-04A	10/25/2005	Prim	0.013	---	[11.7]	[0.814]	0.203
LF-04A	10/25/2005	Dup 1	0.013	---	[11.7]	[0.813]	0.203
LF-05R	03/23/2005	Prim	<0.005	---	0.22	[0.265]	0.154
LF-05R	10/25/2005	Prim	<0.005	---	0.1	[0.16]	0.115
LF-05R	10/25/2005	Dup 1	---	---	---	---	---
LMW-17D	10/25/2005	Prim	---	<0.01	[6.83]	[0.206]	---
LMW-17S	10/25/2005	Prim	---	<0.01	[6.58]	[0.109]	---
LMW-18D	10/25/2005	Prim	---	<0.01	0.07	[0.052]	---
LMW-18S	10/25/2005	Prim	---	<0.01	[16.8]	[0.082]	---
LMW-18S	10/25/2005	Dup 1	---	<0.01	[16.8]	[0.083]	---
LMW-19D	10/25/2005	Prim	---	0.03	[0.93]	0.011	---
LMW-19S	10/25/2005	Prim	---	<0.01	[9.43]	[0.52]	---
MW-13	03/23/2005	Prim	0.011	---	[5.89]	[0.393]	0.284

MW-16B duplicate results 10/25/05 are rejected

[x]=Greater than Action Level ---=Not analyzed

Table B-1
North Bend Mill
Groundwater Results

PERIOD: From 03/23/2005 thru 10/25/2005 - Inclusive
SAMPLE TYPE: Water

SITE	DATE	RESULT TYPE	Barium (mg/l)	Chromium (mg/l)	Iron (mg/l)	Manganese (mg/l)	Strontium (mg/l)
Oregon GroundWater Ref LvlOAR 340-40-020			1.0	0.05	0.3	0.05	
US Primary MCL			2	0.1			
US Secondary MCL					0.3	0.05	
MW-13	03/23/2005	Dup 1	0.011	---	[6.29]	[0.39]	0.282
MW-13	10/25/2005	Prim	0.01	---	[6.52]	[0.35]	0.23
MW-16B	03/23/2005	Prim	<0.005	---	[1.74]	0.028	0.139
MW-16B	10/25/2005	Prim	<0.005	---	0.12	0.048	0.089
MW-16B	10/25/2005	Dup 1	<0.005	---	[13.6]	[0.164]	0.071
MW-17	03/23/2005	Prim	<0.005	---	[8.21]	[0.153]	0.054
MW-17	10/25/2005	Prim	<0.005	---	[13.2]	[0.158]	0.069

MW-16B duplicate results 10/25/05 are rejected

[x]=Greater than Action Level ---=Not analyzed

SLUDGE INVENTORY

Appendix C

Groundwater Data