RE: NEXT First Open Record Submittal (App DR 21-03; V 21-05 and CU 21-04) Email 2.D

Stephenson, Garrett H. <GStephenson@SCHWABE.com>

Wed 1/26/2022 5:44 PM

To: **ePermits - Planning <planning@columbiacountyor.gov>**; Jacyn Normine <Jacyn.Normine@columbiacountyor.gov>

Cc: 'Jesse Winterowd' <jesse@winterbrookplanning.com>; Robin McIntyre <Robin.McIntyre@columbiacountyor.gov>; Robert Wheeldon <Robert.Wheeldon@columbiacountyor.gov>; 'Brian Varricchione (BVarricchione@mcknze.com)' <BVarricchione@mcknze.com>

Here is the fourth of four sections of Mackenzie Exhibit B, noted below.

Garrett H. Stephenson
Shareholder
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gstephenson@schwabe.com

Schwabe Williamson & Wyatt
Please visit our COVID-19 Resource page

From: Stephenson, Garrett H.
Sent: Wednesday, January 26, 2022 5:41 PM
To: 'planning@columbiacountyor.gov' <planning@columbiacountyor.gov>; 'Jacyn.Normine@columbiacountyor.gov' <Jacyn.Normine@columbiacountyor.gov>
Cc: 'Jesse Winterowd' <jesse@winterbrookplanning.com>; 'Robin McIntyre' <Robin.McIntyre@columbiacountyor.gov>; 'Robert Wheeldon' <Robert.Wheeldon@columbiacountyor.gov>; 'Brian Varricchione (BVarricchione@mcknze.com)' <BVarricchione@mcknze.com>
Subject: RE: NEXT First Open Record Submittal (App DR 21-03; V 21-05 and CU 21-04) Email 2.C

Here is the third of four sections of Mackenzie Exhibit B, noted below.

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Subject: RE: NEXT First Open Record Submittal (App DR 21-03; V 21-05 and CU 21-04) Email 2.B

Here is the second of four sections of Mackenzie Exhibit B, noted below.

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Subject: RE: NEXT First Open Record Submittal (App DR 21-03; V 21-05 and CU 21-04) Email 2.A

To Whom it May Concern:

As you can see below, I attempted to send a large PDF file that enclosed NEXT's updated Stormwater Management Plan, which was Exhibit B to Mackenzie's letter submitted as part of our first open record submittal. In our third email, sent at 4:58 PM, we included a link to this document in case the file was too large. Indeed it was, and I have now received bounce back emails from the County (see attached). The County can nonetheless find that the document link is sufficient to submit the document prior to 5:00 PM.

Nonetheless, I understand that the County will accept documents until midnight because it did not indicate a time cutoff at the hearing. Therefore, we have reformatted the document and provide it in sections which are hopefully small enough to be accepted by the County's email server.

Please confirm that you have received this document and that it is part of the record under one or both methods of submittal discussed above.

Thanks!

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From: Stephenson, Garrett H.
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Cc: 'Jesse Winterowd' <jesse@winterbrookplanning.com>; 'Robin McIntyre' <Robin.McIntyre@columbiacountyor.gov>; 'Robert Wheeldon' <Robert.Wheeldon@columbiacountyor.gov>; 'Christopher Efird' <chris@nextrenewables.com>; 'Brian Varricchione (BVarricchione@mcknze.com)' <BVarricchione@mcknze.com>; 'Gene Cotten' <gene@nextrenewables.com>; 'Laurie Parry' <Laurie@stewardshipsolutionsinc.com>
Subject: RE: NEXT First Open Record Submittal (App DR 21-03; V 21-05 and CU 21-04) Email 2

To Whom it may Concern:

Please find attached Exhibit B to the Mackenzie exhibit referenced in email one.

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From: Stephenson, Garrett H.
Sent: Wednesday, January 26, 2022 4:41 PM
To: 'planning@columbiacountyor.gov' <planning@columbiacountyor.gov>
Cc: Jesse Winterowd <jesse@winterbrookplanning.com>; 'Robin McIntyre' <Robin.McIntyre@columbiacountyor.gov>; Robert Wheeldon <Robert.Wheeldon@columbiacountyor.gov>; 'Christopher Efird' <chris@nextrenewables.com>; Brian Varricchione (BVarricchione@mcknze.com) <BVarricchione@mcknze.com>; Gene Cotten <gene@nextrenewables.com>; Laurie Parry <Laurie@stewardshipsolutionsinc.com>
Subject: NEXT First Open Record Submittal (App DR 21-03; V 21-05 and CU 21-04) Email 1

To Whom it may Concern:

Please find attached NEXT's first open record submittal, which includes additional factual testimony. This is the first of a few emails, given the size of some of the files. Please confirm that you have received this, include this in the official record, and place it before the Board.

Thank you,

Garrett H. Stephenson
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Schwabe Williamson & Wyatt
Please visit our COVID-19 Resource page

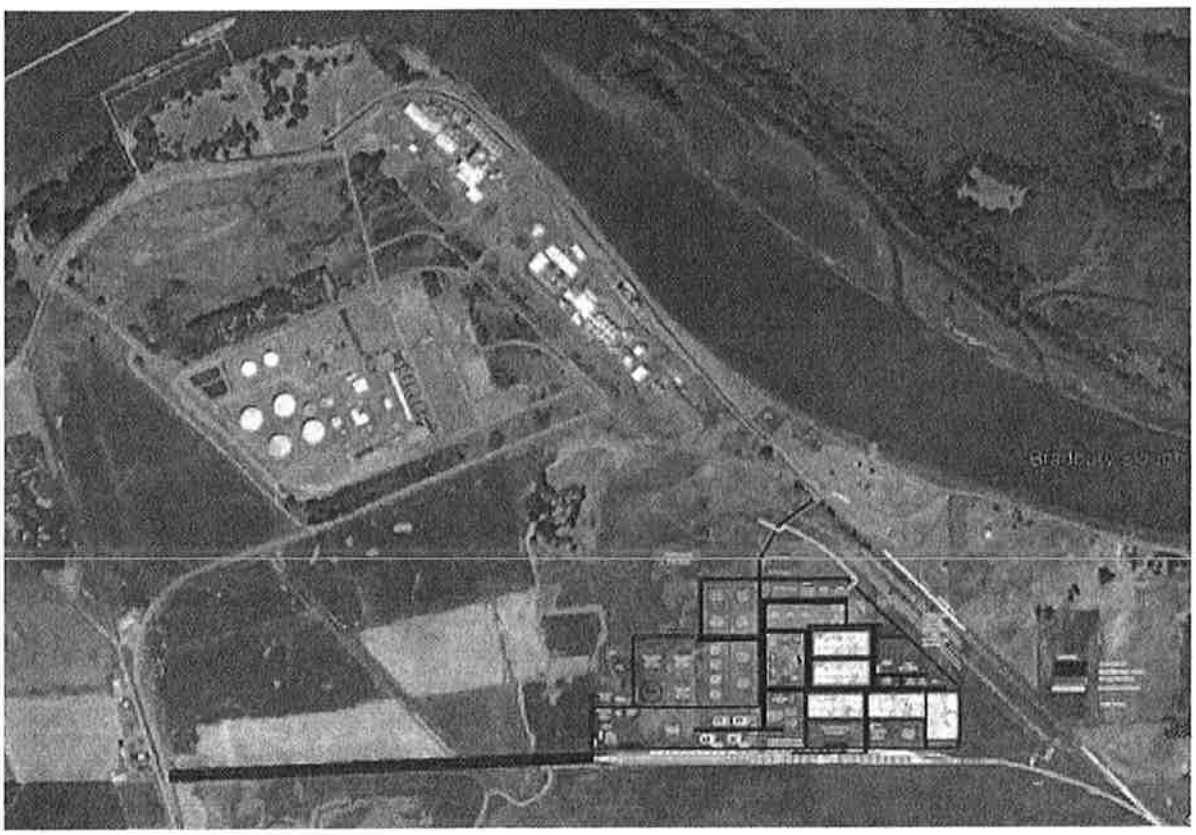
  - Instrument Air/Plant Air Shelter - TBD

- All buildings sizes and contents to be confirmed.
- All building overpressure design to be confirmed with blast study
- Substation buildings will be combination of MCC shelters and RIE rooms. The MCC buildings will be integral to the local operator shelters.

NEXT Renewable Fuels, Inc.

NEXT Renewable Fuels, Oregon LLC
Renewable Diesel Project

WASTEWATER-STORM WATER
DESIGN BASIS

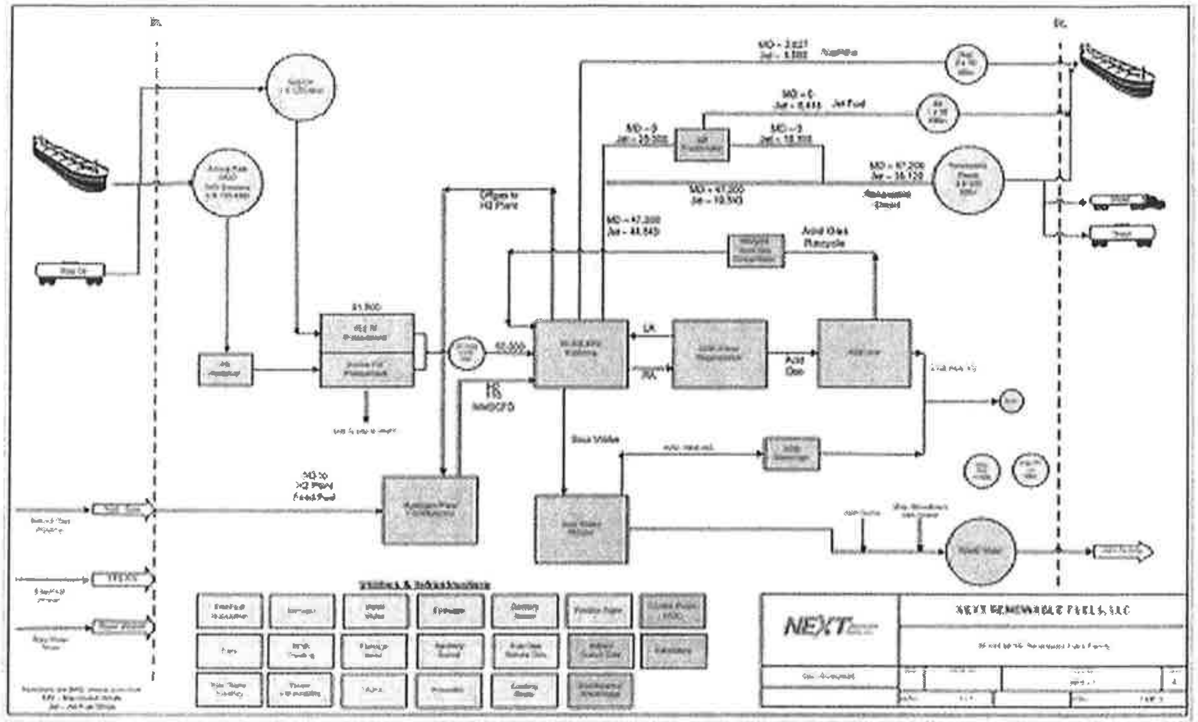


April 15, 2021
Rev 0

I. INTRODUCTION

NEXT Renewable Fuels, Oregon LLC is a private company focused on producing and delivering clean transportation fuels. NEXT plans to build a Green Diesel facility located at Port Westward, Oregon. The general block flow diagram for the Green Diesel facility is shown in Figure 1.

Figure 1 – Green Diesel Flow Diagram



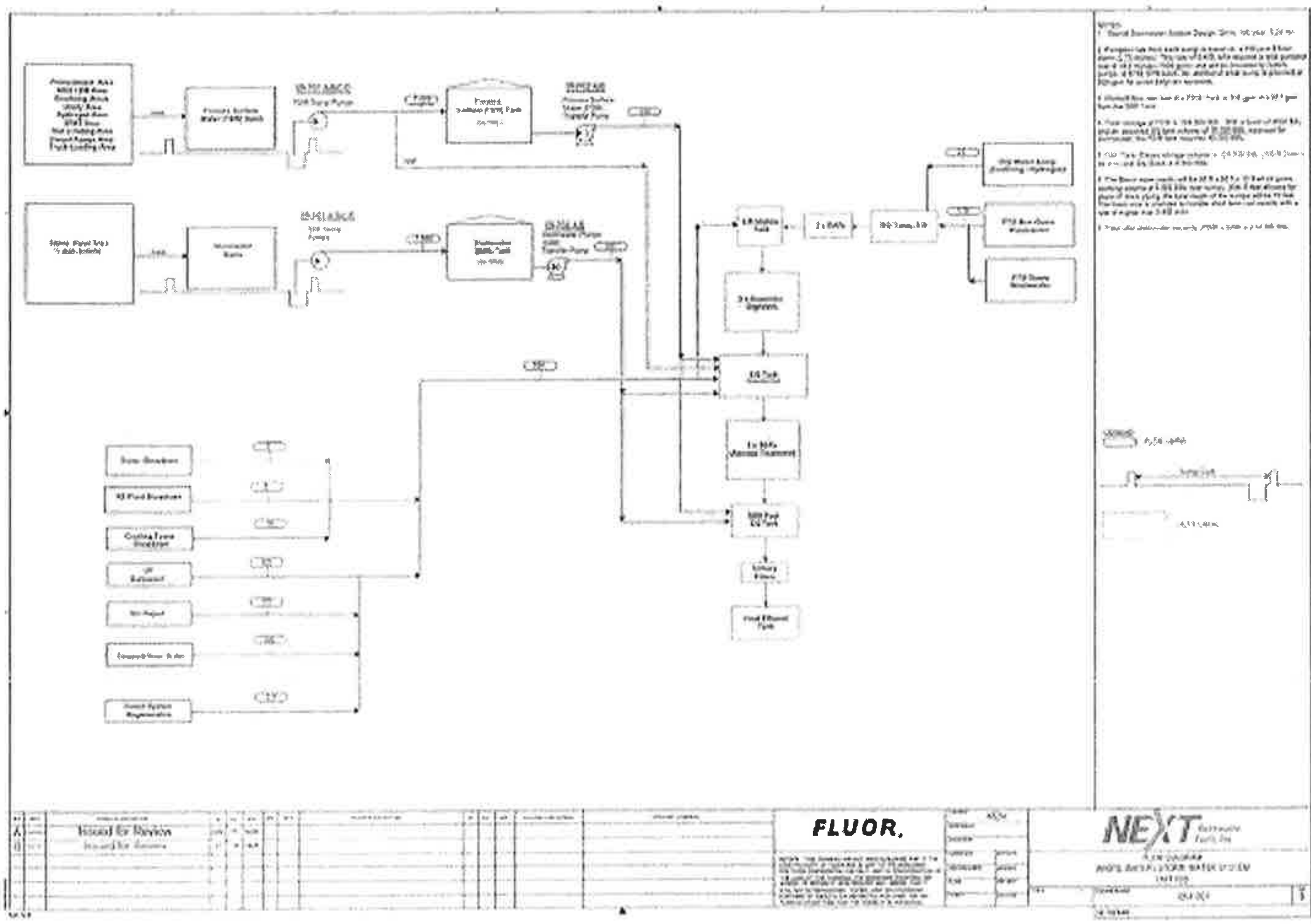
As part of the Green Diesel project, NEXT will install grassroots Wastewater and Storm Water facilities to ensure compliance with the Port of Columbia's NPDES permit and Oregon DEQ's 1200-Z Industrial Stormwater permit.

II. Overview

The NEXT Renewable Fuels facility is a designing a Wastewater Treatment facility to process wastewater and a portion of stormwater produced from processing 50,000 BPD of vegetable oil (VO) and animal fats (AF) to produce Renewable Diesel. The unique feedstocks provide some waste treatment challenges.

The Design for the Wastewater / Stormwater Treatment facilities is outlined in Figure 2:

Figure 2: Wastewater / Stormwater Block Flow



The NEXT wastewater / stormwater effluent will discharge to the existing Port Westward discharge outfall. The effluent qualities will be required to comply with the Port Westward NPDES permit for wastewater discharge (Attachment 1). To ensure compliance with the NPDES permit, the NEXT WWT effluent design specifications, shown in Table 1, are more stringent than required.

Table 1: NEXT WWT/SW Effluent Specifications

WWT Specifications	Spec	Comment
Temperature DT:	0 °F	Temp delta is influent raw water - WWT effluent
COD:	N/A	
BOD5:	≤ 20 mg/L	
FOG:	≤ 20 mg/L	
TSS:	≤ 10 mg/L	
Total Nitrogen:	≤ 50 mg/L	
Phosphorus-P:	≤ 5 mg/L	
Alkalinity:	≥ 50 mg/L	
pH:	6.6 – 8.5	
Free Chlorine:	≤ 0.15 mg/L	

Wastewater Treatment

The Renewable Diesel facility provides some unique waste treatment challenges. As Figure 2 highlights, the WWT flow scheme has been designed to segregate and optimize the treating of the various stream contaminants.

The VO/FA pretreat facility will produce two streams, High and Low Strength. The high strength will contain a high COD load from the degumming section and the low strength will be a lower COD stream made up of several sources. These streams will be segregated and processed differently. The low strength COD stream will be comingled with the normal process water stream and processed through the DAF for separation of oily float, solids and water. The high strength COD stream will be comingled with the DAF float and processed in the Anerobic Digestions system.

The treated products from the DAF and the Anerobic Digestor will flow to the Equalization Tank where they will be comingled with low COD water from several sources, RO Reject, Boiler Blowdown, Stripped Sour Water, and Cooling Tower Blowdown and if necessary, Storm Water from either the process area or the general storm water system. These waters will be mixed and charged to the Aeorbic Sequentual Batch Bioreactors (SBR) for further processing.

The water from the SBR's will flow to the Post Equalization Tank for further oxidation and clarification before being sent to the Tertiary Filtration to substantially reduce solid content. Storm Water from the facility will be comingled with the Post Equalization water and be processed through the Tertiary Filters.

The final step in processing for the wastewater and stormwater is cooling of the streams to ensure compliance with NPDES discharge specifications. A heat exchanger will be used to cool the wastewater/stormwater effluent against incoming plant raw water.

Storm Water System

The storm water system will be designed to collect and process water for a 24 hour 100 year rain event. The design storms used for the project are based on the Columbia County Stormwater Ordinance, Appendix E, using the rainfall depth for Clatskanie.

Table 1: Columbia County Design Storm Rainfall Depths

Storm Event	Water Quality (SLOPES V)	2-yr 24-hr	5-yr 24-hr	10-yr 24-hr	25-yr 24-hr	100-yr 24-hr
Rainfall Depth	1.40"	2.8"	3.4"	3.9"	4.5"	5.4"

Storm facilities for this project are designed with the assumption that infiltration is negligible. The runoff numbers for the site soils are selected for hydrologic soil group C and D to reflect the low-infiltration conditions, as follows.

Table 2: Runoff Efficiency

Surface Coverage	Runoff Collection
Paved Roadway, Building Roof, and Sidewalks	98
Gravel Surfacing and Roadways	92
Proposed Landscaping	78
Existing Grass or Vegetated Field	80

The NEXT facility storm water's will be segregated and provided with several different types of drainage systems based on concerns of potential contamination and appropriate opportunities for treatment. These include:

- Systems for disposal of storm water from outside the process and utility unitpaved areas.
- Systems for collection and transfer for treatment of storm water within the process and utility areas.
- An oily water system for drains from equipment and vessels.

Storm water within the property boundaries will come from several different areas. Listed below are various areas and their proposed drainage:

- **Process & Utility Areas**
  These areas are routed to the Process Surface Water System (PSW) as described below. This area also includes pump manifolds, flaredrum areas, etc.

- **Paved Areas other than Process & Utilities**
  These areas will be routed to the Storm Water System (SW) as described below. This includes paved roads and the swale along each side of the roads as well as parking areas

- **Unpaved Areas**
  These areas will be routed to SW.

- **Inside Tank Dikes**
  Storm water will be contained inside the dike and normally allowed to evaporate. If a diked area

needs to be drained the water will be tested prior to draining. If contaminated, the water will be collected with vacuum trucks or other methods and transported to the WastewaterTreatment Plant.

Process Surface Water (PSW) System:

The PSW (Process Surface Water) drain is a single contained system for collecting storm water and water wash-downs from the paved process unit and utility areas of the facility.This water is considered to be contaminated and requires treatment.

Surface waters from the various process areas are collected through a network of underground pipes and gravity flow to lift stations. Water is pumped from the lift stations to the PSW Tank and is then pumped at a controlled rate to the Wastewater Treatment (WWT).

Surface waters from smaller pump manifolds, flare drum areas, loading/unloading areas,etc. which are remote from the process areas will be collected in a similar manner and routed to one or more centralized lift stations to be pumped to the PSW Tank.

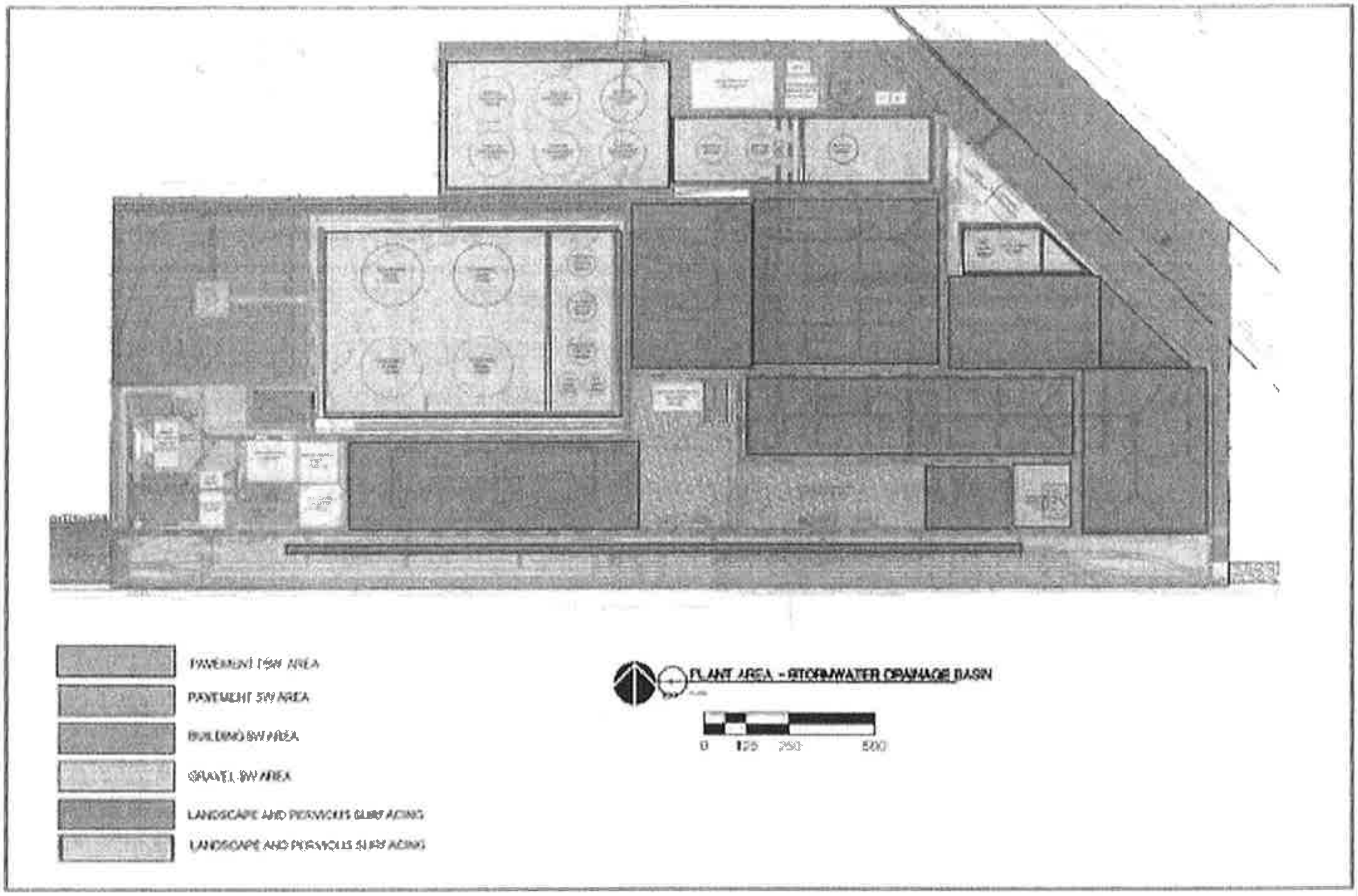
Storm Water (SW) Drain System:

In the SW (Storm Water) drain system, rain water falling outside the process areas is channeled through open trenches or underground pipes to a storm water tank. Runoff from the paved roads is collected through a network of underground pipes and gravity flows beyond the paved areas to storm water collection areas where it is pumped to the Storm Water tank. Storm water from the SW Tank will be tested and if complies with NPDES permit specifications will be pumped at a controlled rate to the WWT where it will be comingled with the WWT effluent and then processed through Tertiary Filtration before flowing to the Port Outfall.

Outlined below is the acreage attributed to each storm water basins. Figure 3 is the geographic area's that make up the different storm water basins.

NEXT Storm Water Basin Acreage	
	Acre's
Process Storm Water Area	32
Storm Water Area	30
Other Pervious Surface Area	38
Total Surface Area	100

Figure 3: Storm Water Basin Geographic Definition



Utilizing the 24 hour 100 year rain of 5.4 in., the storm water system will need to contain ~200,000 bbl of process and storm water. The water will be contained in various tanks within the Process and Storm Water systems and pumped back to the process facility over a 2 week period. The current design has the following water storage capacity.

Storm Water Tankage

Source	PSW Tk (bbls)	Eq 1 (bbls)	Eq 2 (bbls)	Sump (bbls)	Total (bbls)
PSW	50,000	40,000	15,000	12,468	117,468
	SW Tk				
SW	100,000				100,000
Total Water Storage					217,468

III. DESIGN BASIS / CONSIDERATIONS

WWT INFLUENT WATER SPECIFICATIONS

Pretreat Unit Summary Table		without high COD stream to WWTP		high-COD stream only		If all streams combined
Flow Rate	GPM	118		53		171
Flow Rate	pph	58962		26233		85194
COD (estimated)	ppm	19824		282697		100767
BOD5	ppm	11498		163965		58445
Lyso-Phospholipids	ppm	335		28631		9048
Phospholipids	ppm	0		0		0
Fats & Oils	ppm	9058		52771		22518
Inorganic Chlorides	ppm	15		655		212
Citric Acid	ppm	0		9772		3009
Phosphorus	ppm	43		1661		542
TSS (insoluble impurities + Fats & Oils entrained with insoluble impurities)	ppm	0		93814		28887
TDS (not considering TDS of process water provided by client)	ppm	102		3099		1025
Sulfur (rough estimate)	ppm	51		807		284
Nitrogen (rough estimate	ppm	130		2039		717

	Cooling Tower Blowdown	RO Reject
Dissolved Oxygen, ppm O2		
pH	7.5-8.0	
Total Hardness as CACO3, ppm	345	590
Alkalinity – Bicarbonate, ppm	266.5	506
Total Iron, ppm FE	0.21	0
Total Copper, ppm Cu	0.0035	
Total Dissolved Solids, ppm	450	850
Total Suspended Solids, ppm	73.5	127
Silica ppm as SiO2	63	117
Conductivity, microohms/cm at 68F	725	1250
Turbidity	-	
Chloride	23.5	
Fluoride	0.5	
Nitrate as N	2.5	
Sulfate	48.5	
Calcium	79.5	
Magnesium, Total	23	
Potassium, Total	6	
Sodium, Total	33	
Temp	70-120 F	70-90 F

Typical Boiler Blowdown Properties		
Boiler pressure	psig	600 - 750
Iron Concentration	(ppm)	0.025
Copper Concentration	(ppm)	0.02
Hardness $CaCO_3$	(ppm)	0.2
Silica Concentration	(ppm)	30
Alkalinity $CaCO_3$	(ppm)	400
Total Dissolved Solids	(ppm)	1000
Specific Conductivity	(µS/cm)	4000
Temp		400-450 F

Sour Water Effluent Stream	
Ammonia ppm	50
H2S ppm	<5
Phenol ppm	30
BOD ppm	120
COD ppm	514
TOC ppm	160
pH	5-7

Olly Water Effluent		
pH		6-9
COD, ppm		750
BOD, ppm		300
TSS, ppm		250
TOC, ppm		150
Alkalinity, ppm		125
Ammonia, ppm		0
Temp		100 F

APPENDIX F

GEOTECHNICAL ENGINEERING REPORT

Harza Engineering Company
Fossil Power Business Unit
Two Honey Creek Corporate Center
115 South 84th Street, Suite 200
Milwaukee, Wisconsin

GEOTECHNICAL EVALUATION
SUMMIT/WESTWARD ENERGY PROJECT
CLATSKANIE, OREGON

May 2001

REGISTERED PROFESSIONAL ENGINEER
58371PE
OREGON
JULY 13, 1999
MITCHELL F. SCHAUB
Expires: 06/30/02

Mitchell F. Schaub, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
9057
OREGON
DEC. 17, 1976
GERALD L. JACKSHA
EXPIRES 06/30/02

Jerry L. Jacksha, P.E.
Principal Engineer

Arlan H. Rippe, P.E.
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Squier Associates, Inc.
4260 Galewood Street
Lake Oswego, Oregon 97035
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201578

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GEOTECHNICAL EVALUATION
SUMMIT/WESTWARD ENERGY PROJECT
CLATSKANIE, OREGON

1.0 INTRODUCTION

This geotechnical evaluation report has been completed for the proposed Summit/Westward Energy Project, which includes a new natural gas-fired combined-cycle combustion turbine generation facility located near Clatskanie, Oregon. The project site is located on Port of St. Helens property located in Columbia County approximately seven miles northeast of Clatskanie, Oregon. The Vicinity Map, Figure 1, shows the location of the project site.

The purpose of this evaluation was to present findings regarding the geologic and seismic setting of the project site; assess the nature of the subsurface conditions and materials which underlie the project site including site specific seismic evaluation; develop preliminary conclusions concerning the key geotechnical aspects of the project, such as foundations for the turbines/generators and other settlement sensitive facilities; seismic design considerations; and related site geotechnical issues. This report also contains "site specific geological and soil stability assessment" information pertinent to site certificate application, Exhibit H, requirements by Oregon Department of Energy, Energy Facility Siting Council.

2.0 LIMITATIONS

The scope of the geotechnical evaluation presented herein is limited to the assessment of geologic site-specific conditions and evaluation of the subsurface conditions related to the proposed facilities for the Summit/Westward Energy Project near Clatskanie, Oregon. This report has been prepared to aid Harza Engineering Company, Milwaukee, Wisconsin and the project owner in the evaluation of the site and application for site certificate for the proposed facility in accordance with generally accepted engineering geologic and geotechnical engineering practices. No other warranty, based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

Our description of the project represents our understanding of the significant aspects of the project relevant to the general arrangement of the project and the proposed site layout provided by Harza Engineering. In the event that any changes in the proposed locations of the structures as outlined in this report are planned or occur, we recommend that a geotechnical review of the changes be made to affirm in writing the conclusions of this report.

The scope of our services reported herein included environmental field screening of the near surface soils to a depth of 15 feet below the current ground surface for the presence of certain soil contaminants. Any statement in this report or on the boring logs regarding odors noted or unusual or suspicious items or conditions observed are solely for the information of our client.

The analyses and conclusions represented in this report are based on the data obtained from the borings made at the locations indicated on the Boring Location Site Map, (Figure 2) and from other information discussed herein. This report is based on the assumption that the subsurface conditions across the site are not significantly different from those revealed by the borings. However, variations in soil conditions may exist between the borings locations. The nature and extent of the variations may not become evident until further investigations are made at the site during the design phase or during construction.

The exploratory activities, laboratory testing, and preliminary analysis are consistent with those normally used in conceptual or preliminary geotechnical evaluations and for site characterizations to develop budgets for future design and construction. When concepts have been better defined, additional explorations and analyses will be necessary to complete the geotechnical analysis and to provide design recommendations.

3.0 BACKGROUND INFORMATION

3.1 Topography

The site is located in the Oak Point 7½-minute quadrangle (U.S. Geological Survey, 1985). The proposed generation facilities site is a relatively flat, vegetation covered pasture land with shallow drainage ditches containing water generally to the south and east of the proposed main plant facilities. The greatest relief on the site is related to the existing drainage ditches, which are less than 10 feet deep with associated spoil piles from the ditch excavations. The ground surface on the site varies between elevations 5 and 10 feet, based on North American Vertical Datum (NAVD) 1988, according to the contours shown on Figure 2. The topography north of the site remains flat for a distance of approximately 2000 feet to a levee that bounds Bradbury Slough, a side channel of the Columbia River.

3.2 Project Description

The proposed project layout map is shown on Figure 2. Along the northeast border of the property are existing gas lines and power lines and a railroad spur. We understand that the generation facility will contain the following major components:

- Combustion Turbines and Generators;
- Heat Recovery Steam Generators (HRSGs);
- A Steam Turbine, Condenser and Generator;
- Main Power Transformers;
- Miscellaneous Buildings;
- Multi-cell Cooling Tower Complex;
- Water Storage Tanks;
- In-plant Substation and Switch Yard; and
- Pipes, Conduits, and Pipe Racks.

We understand there also will be numerous buried utilities and associated underground vaults constructed across the proposed plant site to depths up to 20 feet. Large diameter underground pipelines will be installed between the cooling tower and the steam generator. We understand the orientation of the structures shown on Figure 2 could change, but the general spacing or relative location will remain similar.

4.0 GEOLOGIC SETTING

The information in this section represents a summary of the geologic setting information presented in Appendix D.

4.1 Regional and Site Geology

The Summit/Westward Energy Project site and its related/supporting facilities are located on the Columbia River alluvial valley within the Coast Range physiographic province of northern Oregon and southern Washington. A physiographic province is a region of similar geologic history and composition. The Coast Range province is broadly upwarped, forming a low mountain range located between the Pacific Ocean and coastline on the west and Willamette Valley-Puget Sound Lowlands on the east. The general geology in the vicinity of the project area is shown in Appendix D, Geology Map, Figure H3 (Walsh and others, 1987 and Walker and MacLeod 1991). The region is underlained from oldest to youngest: basement rock of

Eocene epoch age volcanic sea floor basalt and island volcanic centers; a thick marine sedimentary sequence of younger Oligocene to Miocene; Miocene epoch Columbia River Basalt lava flows; and local younger alluvial deposits along the Columbia River, coastal rivers and bays. The Eocene volcanic rock basement is estimated to be about 20 miles thick under the Oregon Coast Range (Orr and Orr, 1996). The overlying marine sedimentary sequence is at least 5,000 feet thick and the Columbia River Basalt 1,400 feet thick in the northern Oregon Coast Range (Beaulieu, 1973). The alluvial sediments may be about 350 feet thick.

Following the cessation of Columbia River Basalt volcanism, the Coast Range began to uplift. Concurrently, the eastern and western margins began to subside and sedimentation resumed along the eastern and western margin of the uplift. As the uplift continued, the erosive power of the Columbia River was able to maintain its course through the growing mountain range.

During the Pleistocene (2 million years) (Orr and Orr, 1996), major continental glaciers periodically formed over much of Canada and Europe. At glacial maximums, vast quantity of water was locked up in glacial ice, which caused 300 to 450 feet lowering of sea level (Balwin, 1964). During these times, the Columbia River eroded a deep channel. The eroded Pleistocene Columbia River channel was probably greater than 350 feet deep at the project site.

During glacial maximums, glacial ice advance blocked the Clark Fork River in northern Idaho and northwestern Montana. Water backed up behind the ice-dam until the dam became unstable and failed, releasing a vast flood of water (Trimble, 1963). These floods are known as the Pleistocene floods or "Bretz Floods". These floods scoured and redeposited sands and gravels in the Pleistocene river channel. At the site, the Pleistocene channel at the time of the floods was probably greater than 350 feet below the present day ground surface. Consequently, the Pleistocene flood deposits are not exposed at the surface in the lower Columbia River valley but are probably present at depths below 300 feet.

At the end of each glacial period, including the latest, sea level rose rapidly as the glacial ice melted. This rise in sea level caused a general flooding and formation of an estuary environment in the lower Columbia River. The base level of the Columbia River rose concurrently, resulting in rapid sedimentation of alluvium along the river. This alluvium consists of sand deposit along the river channel and silt, clay, and organic soils in the overbank (flood plain) deposit.

The geologic structure within the vicinity of the project area is complex. Overall, the area is dominated by the broad north-south upwarp of the Coast Range. The amount of upwarping is uneven, with both the Tillamook highlands to the south and Willapa Hills to the north, uplifted higher than the area in between along the lower Columbia River. Geologic mapping shows the older rocks exposed in the core of the uplifted areas are extensively faulted (Walker and MacLeod, 1991 and Walsh and others, 1987). Faults are generally oriented northwest-southeast and northeast-southwest. Most of these faults, however, appear to be restricted to the older rocks suggesting that they are related to the older tectonism and were not active after the deposition of the younger sedimentary rocks. Therefore, they are not active now.

Superimposed on the broad uplift are numerous small secondary folds. In the vicinity of the project, these secondary folds are oriented northwest-southeast (Walker and MacLeod, 1991 and Walsh and others, 1987). The nearest mapped secondary fold is a syncline that trends through Quincy, beneath the project site and into the state of Washington.

4.2 Seismic Setting

The site is located in the seismic region known as the Cascadia Subduction Zone (CSZ), which extends from Northern California to British Columbia. A more in depth discussion of the CSZ is presented in Appendix D. In the CSZ, just off the coast of Oregon and Washington, the oceanic Juan de Fuca Plate is being forced under the North American Plate. Much of the Pacific Northwest's topographic relief, including the Coast Ranges and Cascade Mountains and the region's seismicity, can be attributed to the plate tectonics of the region. Three types of earthquakes are known to occur within the CSZ: shallow crustal, deeper subcrustal intraplate, and the large interface. The most seismically active area occurs in the Puget Sound region, 60 miles to north.

Earthquakes are sized using two fundamentally different scales: Modified Mercalli scale and magnitude scales. The following definitions are based on Rogers, Walsh, Kockelman, and Priest (1996) definitions. The Modified Mercalli scale was developed before the advent of mechanical means of measuring earthquakes. It is a subjective numerical index describing the severity of an earthquake in terms of its effects on the Earth's surface and on humans and their structures. The index scale spans from Roman Number I, felt by few, to XII, total destruction. Unless specifically stated, Modified Mercalli intensity is the maximum observed at the epicenter of an earthquake.

Magnitude scale is a measured number that characterizes the relative size of an earthquake. It is based on measurement of the maximum motion recorded by a seismograph corrected for attenuation to a standardized distance. Several magnitude scales have been defined, but the most commonly used are 1) local magnitude (M_L), commonly referred to as "Richter magnitude," 2) surface-wave magnitude (MS), 3) body-wave magnitude (m_b), and movement magnitude (M_W). The first three scales have limited range and applicability and do not satisfactorily measure the largest earthquakes. The moment magnitude (M_W) scale is based on the concept of seismic moment, and is uniformly applicable to all sizes of earthquakes. Conceptually, all magnitude scales can be cross-calibrated to yield the same value for any given earthquake. In practice, however, this has only been proved to be approximately true. For engineering purposes, the scales are similar enough that the differences are not significant. Historically, most of the earthquakes recorded in the Pacific Northwest were reported in local magnitude M_L scale. For this report, magnitudes are expressed as **M** without attempting to convert between the various scales.

Shallow crustal earthquakes take place typically between depths of 10 km and 20 km. Several earthquakes between estimated **M**4 and **M**5 have occurred within 31 miles (50 km) of the site over the past 150 years. The most significant event is the estimated **M**5.2, 1962 Portland-Vancouver earthquake located approximately 46 miles east-southeast of the site. Earthquake recurrence relationship suggests a magnitude **M**6.0 event with about a 500-year recurrence and a magnitude **M**6.5 event with about a 5000-year recurrence.

The second major type of earthquake that could affect the site is a deeper subcrustal intraplate earthquake occurring within the subducting Juan de Fuca Plate at depths between 40 km to 60 km. The 1949 Olympia and the 2001 Nisqually earthquakes were deep subcrustal events. An intraplate earthquake could potentially occur directly below the site (depth 50 km). The maximum expected magnitude for an intraplate earthquake is between **M**7.0 and **M**7.5. An earthquake recurrence relationship extrapolated to large magnitudes based on smaller magnitude subcrustal earthquakes suggests that an **M**7.0 event may occur in the region once in 1000 years. The distance that this possible event could have ranges between 0 to more than 30 miles (0 to 50 km). For hazard analysis purposes, a **M**7.0 occurring directly beneath the site (distance 0 km, depth 50 km) and a larger **M**7.5 event occurring at a distance of 30 miles (50 km) were considered.

The third major type of earthquake that potentially could affect the site is an interface, or subduction zone, earthquake, which could take place at the boundary of the Juan de Fuca and the North American plates. Although a subduction zone earthquake has not been historically recorded off the coast of Oregon or Washington, geologic data suggests that a **M9+** earthquake is possible from an interface event. The best estimate for the most likely size ranges between magnitudes M8 to M9 depending upon the length that ruptures. Recurrence for a subduction zone interface earthquake ranges from 350 to 600 years, with a mean recurrence of about 450 years. The last event occurred 300 years ago. The nearest approach of a CSZ interface earthquake would be about 30 miles (50 km) west of the site.

A literature review was also conducted to identify known geologically active or potentially active faults within 62 miles (100 km) of the site. The results are presented in Appendix D. Primary reference sources reviewed include Seismic Design Mapping: State of Oregon (Geomatrix Consultants, 1995), National Seismic Hazard Maps (Frankel, et al., 1996) and Wong and others (2000). The review shows that there are at least eleven geologic faults or fault zones with or suspected with greater than 50 percent probability of having Quaternary movement (movement within the last two million years). In addition, the CSZ is active and underlies the site at depth.

4.3 Geologic Hazards

Potential geologic hazards for the site were evaluated. The results are presented in Geologic and Soil Stability Assessment, Appendix D. Based on the geologic history, the alluvial soil is assumed to extend down to about 350 feet below sea level. Deep alluvial soils at the site strongly affect seismic ground response at the surface. The assessment identified the primary geologic and soil stability issues are associated with seismic hazards: primarily strong ground shaking, the potential for liquefaction of some of the subsurface materials, and seismically induced settlement. The analysis indicates that seismic waves would be significantly dampened and deamplified as they traverse up through the deep soil column. In addition, the analysis suggests that some of the loose sandy silt and sand strata may be susceptible to liquefaction during a subduction zone earthquake event. The occurrence of liquefaction could result in loss of foundation bearing capacity of the near surface soils and/or settlement. Consequently, heavy structures and structures sensitive to settlement probably will be founded on deep piles driven to below identified liquefiable zones to provide adequate support.

Other geologic hazards, in our current opinion, are not significant at the site. The site is flat and there are no landslide or slope stability issues. Also, there is little risk of fault displacement at the site. In addition, the site is located behind flood control levees that provide 100-year flood protection with 4.7 feet of freeboard. Since the site is level and over 2000 feet from Bradbury

Slough, the potential for lateral spreading is not considered a hazard. Also, the site is too far from the ocean to be affected by tsunami.

5.0 FIELD EXPLORATIONS AND LABORATORY TESTING

The subsurface conditions beneath the site were investigated with eight borings that were advanced between April 16 and April 25, 2001. Laboratory and field soil tests consisting, of among others, photoionization, soil classification, seismic compression and shear wave, and soil resistivity tests were performed. Presented in the following sections is a discussion of tests performed at the site during the field exploration and laboratory testing that were performed on the samples returned to our office.

5.1 Field Explorations

The locations of the borings, designated B-1 through B-8, are shown on the Borehole Location Map, Figure 2. The borings were advanced to between 80 and 150 feet from the ground surface using a combination of track and truck-mounted drill rigs owned and operated by Geo-Tech Explorations of Tualatin, Oregon. A total of 852 feet lineal feet was drilled, sampled, and logged.

During the drilling, disturbed samples were obtained at about every 2.5 feet in the upper 25 feet, and about 5 feet thereafter using the Standard Penetration Test (SPT) ASTM D1586. During the Standard Penetration Test, the N-value blow counts required to advance the sampler with a 140-pound weight dropped 30 inches was recorded. The N-value, expressed as blows per foot, is used to provide a measure of the relative density of granular soils such as sand, and the consistency of cohesive soils such as silt and clay. In addition, thin-wall Shelby tube samples of relatively undisturbed soil were obtained at selected depths.

Two piezometers, consisting of a slotted PVC pipe backfilled with clean free draining sand were installed in Borings B-4 and B-7 at the site to allow for future measurements of a ground water level. At the ground surface, each piezometer pipe was placed inside a flush mounted monument cover set in concrete. All the other borings were backfilled with bentonite up to the ground surface at the completion of drilling, except for B-3 that also contained the downhole testing PVC pipe, described below.

Presented in Appendix A is a description of the procedures used in making the borings, including the details of the piezometer installations and the techniques utilized in obtaining the various types of soil samples. Table A1 in Appendix A presents the terminology used to describe the soils. Presented on Figure A1 of Appendix A is information related to the symbols, soil and well material graphics, and soil property data presented on the boring logs. The logs of the borings are presented in figures A2 through A9.

5.2 Photoionization Testing

Environmental screening for the presence of volatile vapors in the upper 15 feet of each boring was analyzed by use of a Photoionization detector (PID). The PID measures vapors released from chemical volatilization of organic compounds in parts per million (ppm). For the purpose of environmental screening, a lower limit threshold was set to 10 ppm for this project based on typical industry standards, before further environmental analysis was considered necessary. Additional information on this testing is contained in Appendix A.

5.3 Laboratory Testing

Laboratory tests were performed on selected soils returned to our laboratory to evaluate the soil index properties and provide data related to the strength and settlement characteristics of the soil. The testing program adopted for this investigation includes soil visual examinations, moisture content, grain-size analyses, Atterberg limits, and unit weight measurements. In addition, two unconfined compressive strength and a soil consolidation test were also performed. Presented in Appendix B of this report is a description of the laboratory tests that were performed and the testing results.

5.4 Downhole Seismic Tests

A downhole seismic wave velocity survey for S and P waves was conducted at the project site in Boring B-3 on April 22, 2001. The test was performed by Northwest Geophysical Associates, Corvallis, Oregon, and the results are presented in Appendix C. In general, the test measures the time required for shear (S) and compression (P) waves propagation through soils over a range of distances from a surface energy source. By measuring the arrival time of shear waves at incremental depths in the borehole, a profile of shear wave velocity is developed. Changes in shear wave velocity with depth in the borehole were used to predict differences in soil types, soil properties and soil behavior. Shear wave velocity in the soils was used in the seismic analyses of the site and an evaluation of the range of the level of ground shaking during the controlling earthquake event.

5.5 Soil Resistivity

Soil resistivity measurements were made at the site on May 3, 2001 to determine the soil resistance to an electric current. We understand this information will be used to evaluate the grounding potential of the soils at the site. The resistivity of the soil was measured using the four-point Wenner method with tests performed by Northwest Geophysical Associates. The results of the test are presented in Appendix E.

6.0 DISCUSSION OF SUBSURFACE CONDITIONS

6.1 Soils

Figure 3 through Figure 5 present general geologic cross sections, which show in a generalized manner, the interpreted subsurface conditions disclosed by the borings at various locations at the site. The Cross Sections are designated A-A', B-B', and C-C' and their location and orientation are shown on the Site Plan, Figure 2. The geologic Cross Sections are interpretive in nature and the contacts between soil units may be gradational. Further, variations in soil conditions may exist between the locations of the borings.

As shown on the geologic Cross Sections, the subsurface materials encountered at the site can be divided into two general soil units within the depth of our explorations, based on their engineering characteristics and stratigraphic position. The subsections that follow present a description of the two soil units, including the subsurface conditions and materials present across the site. A more detailed description of the soils is described on the Boring Logs, Figures A2 through A9 (Appendix A).

6.1.1 Upper Fine-Grained Alluvium

An upper fine-grained alluvium unit was encountered in all the borings and consists generally of very soft silt with various minor amounts of fine sand. The upper alluvium was encountered up to depths between 25 to 60 feet from the ground surface. Blow counts or N-values, observed during the Standard Penetration Test (SPT) varied from 0 to 11 blows per foot. In general, the predominantly silt soils, which constituted a majority of the unit, had N-values between 0 and 2. Higher N-values between 5 and 11 were observed in the silt soils containing, in general, a higher percentage of sand. Organics, including isolated pieces of plant and wood fiber, were generally observed in estimated amounts between 5 to about 15 percent (based on volume) of the soil samples. The moisture content of the unit ranged between 40 to 70 percent. Some higher moisture contents were observed within the soils containing a larger percentage of organic matter.

The plasticity characteristics of the soil unit, as measured in Atterberg Limits Tests, indicate a Liquid Limit (LL) between 53 and 73 percent, and a Plastic Limit (PL) between 35 and 41 percent. These values are influenced, in our opinion, because of the presence of organic matter, as described previously. The Plasticity Index (PI) ranged between 0 percent (non-plastic) to 34 percent, with a majority of the test results below 15 percent. Locally within the unit, some minor amounts of clay were apparent, up to estimates of about 5 percent, by weight of the sample. Classification tests performed on the silt, including dry strength, dilatancy and toughness, performed in general accordance with ASTM D-2488, indicate a range of plasticity between non-plastic to medium plasticity, with a majority of the results ranging from non-plastic to low plasticity.

In general, as indicated by a majority of the "N"-values between 0 and 2, the silty soil was classified as either "very loose" or as "soft", depending upon its apparent plasticity. The condition of the silt, together with a high ground water level at the site, and the presence of organic matter, in our opinion, contributes to a moderate to high potential of settlement within the unit. A consolidation test was performed on a sample of the upper fine-grained alluvium with results discussed under Section 7.3.

Measurements of shear strength were performed on selected samples of the soil unit and consisted of unconfined compressive strength test, pocket penetrometer, and torvane strength tests. The results of the unconfined tests indicate undrained shear strength of between .18 and .25 ton per square foot (tsf), correlating to very soft. Pocket penetrometer tests and torvane tests performed on Shelby tube samples returned to our laboratory indicate a range of undrained shear strength between 0 and .25 tsf.

6.1.2 Lower Sandy Alluvium

Below the upper fine-grained alluvium, we encountered a lower sandy alluvium unit consisting mostly of fine-grained poorly graded sand with varying amounts of silt. All of the borings were terminated in this soil unit. N-values varied between 4 to 60 blows per foot, with most of the values between 20 to 35 blows per foot. The lower N-values within this unit were generally observed in the sand soils that contained a higher percentage of silt. The moisture content of the unit ranged between about 30 to 50 percent. Organics, although observed in this unit, were generally less abundant than observed in the upper fine-grained alluvium.

6.2 Ground Water

Ground water was measured at depths between 2 to 4 feet from the ground surface in Borings B-3, B-4 and B-5 during and immediately after drilling. A ground water level was not observed in the other borings and is in general, difficult to measure when a mud-rotary system is used. Based on our analyses and our experience, we believe that the ground water level at the site should be expected at elevations closely related to the surface water level in the Columbia River, located to the north of the site.

6.3 Photoionization Results

Photoionization results on soil samples in the upper 15 feet of each boring ranged from 0 to 8 ppm. Boring B-3 at 10 feet registered 8 ppm, while all other results in the other seven borings registered no more than 0.1 ppm. Since all results were below the minimum threshold, 10 ppm, previously described, no samples required additional analytical analysis.

7.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

7.1 General Findings

The field explorations disclosed that deep soft alluvial sediments exist across the site. The conditions observed in the borings suggest that the upper 50 feet of soils is relatively loose to very soft, and potentially liquefiable during the design earthquake. In addition, ground water occurs at a relatively shallow depth. During periods of flooding, water level in the river is higher than the ground surface. High ground water is currently controlled by a drainage ditch system managed by the Beaver Drainage District. In our opinion, the upper relatively soft soils in their existing condition are not suitable for the support of settlement sensitive equipment, heavily loaded mat foundations, and building foundations. Pile supporting structures or ground modification techniques will be discussed in later sections.

7.2 Site Preparation/Earthwork/Ground Water Control

The following issues are considerations for future design and construction activities.

7.2.1 Clearing and Stripping

There are scattered trees that will need to be cleared and grubbed. The pasture land vegetation cover and topsoil should be stripped under settlement sensitive facilities and other areas where organics left in-place would be a detriment to long-term performance.

7.2.2 Well Abandonment

Regarding subsurface features, we became aware of an existing shallow water well that would need to be abandoned by a State of Oregon licensed water well driller. Similarly, the two soil borings containing the standpipe piezometers and the one boring containing the grouted pipe for the downhole seismic tests will need to abandoned according to Oregon Department of Water Resources regulations.

7.2.3 Working Pad (Site Fill)

Due to the relatively very loose and soft nature of the shallow subsurface materials and the high ground water levels, working pads or mats are advisable for the construction period. Typically, a pad constructed of imported granular material, preferably well-graded, free-draining crushed rock placed on a heavy non-woven geotextile would be used. The material specifications, thickness, and placement methods would depend on how the working pad would be incorporated into the design of the various foundation systems, roadway subgrade preparation, and buried piping. Based on discussions with the site grading consultant, we understand that site filling throughout most of the area will be less than 3 feet. The exception would be areas requiring special treatment. Since site filling would cause some settlement, we have assumed a site fill thickness of 3 feet in our analysis discussed in Section 7.4.

7.2.4 Drainage Ditches

There are at least two fairly deep drainage ditches that intersect the footprint of the plant facilities that will need to be dealt with during site preparation. We understand that these ditches are part of the Beaver Drainage District.

7.2.5 Softer Surface Areas

In the southwest portion of the site in the vicinity of the existing barn, we noticed that the ground surface was generally softer than the rest of the plant site area. Additional stripping or other treatment may be required if facilities area placed in this area.

7.2.6 Ground and Subsurface Modification

To decrease the long-term settlement of the deep, soft and loose subsurface materials for static and seismic loading conditions, various ground improvement methods may be needed as part of the overall site preparation. More discussion related to this is mentioned in sections below.

7.2.7 Earthwork and Ground Water Control

For the various earthwork activities, heavy earthwork equipment and loaded dump trucks most likely will have difficulty operating on the existing ground surface. During our explorations, truck-mounted soil exploration drill rigs were breaking through the vegetative cover and were stuck several times. For the excavations that extend below the shallow ground water, we anticipate that lowering ground water levels with positive control dewatering systems would be needed. Use of sump systems is generally not feasible for these types of soil. The use of excavated material from above and below the ground water levels for structural fill or backfill most likely is not feasible. Potential uses of the excavation spoils may be for landscaping or grading for surface drainage improvements. Grading this material with its high moisture content will be difficult.

7.2.8 Other Related Issues

There other site preparation issues adjacent to the site, such as construction of an access roadway embankment to change grade from the existing road on the levee adjacent to the slough, crossing of the raised grade railroad tracks, and preparation of subsurface for utilities coming into or leaving the site.

7.3 Soil Parameters for the Site

Soil parameters are provided for the project site to assist in the preliminary project site evaluation. Based on the subsurface conditions and the laboratory testing, the recommended soil parameters are presented in Table 1, below. Descriptions of the various parameters follow Table 1.

Table 1
Soil Parameters for the Site

Soil parameter	Very loose silt to sand	Very soft silt	Lower Sandy Alluvium
Poisson's ratio	0.2	0.3	0.25
Modulus of elasticity	100,000 psf	10,000 psf	250,000 psf
Shear modulus	300,000 psf	340,000 psf	900,000 psf
Subgrade modulus	30 pci	25 pci	100 pci
Moist unit weight	105 pcf	100 pcf	120 pcf

psf = pounds per square foot
pcf = pounds per cubic foot
pci = pounds per square inch per inch

7.3.1 Poisson's Ratio

Poisson's ratio, μ, is defined as the ratio of axial compression to lateral expansion strains. Poisson's ratio is both nonlinear and stress-dependent. The range of Poisson's ratio is relatively small for the same types of soil at the site; therefore, we estimated Poisson's ratio based on the soil classifications. The estimated Poisson's ratio values are presented on Table 1. The Poisson's ratio for the very soft silt is estimated for drained condition.

7.3.2 Modulus of Elasticity

The modulus of elasticity, E_0, is the initial slope of soil stress-strain curve. It is often estimated by correlation from field tests, such as the Standard Penetration Test (SPT) and Cone Penetration Test (CPT). For this project, we used the field SPT N-values and laboratory test results to estimate the Modulus of Elasticity for both the very loose silt to sand and very soft silt. The modulus of elasticity of the very soft silt is estimated for drained condition. The estimated modulus of elasticity values are shown in Table 1. Estimates of E_o were based on information from EPRI, 1990.

7.3.3 Shear Modulus

The shear modulus, G, is defined as the slope of the shear stress-strain curve. For soil seismic evaluation purposes, the shear modulus is often estimated by using shear wave velocity measurements, v_s. The relationship between shear modulus and shear wave velocity is: $G = \rho v_s^2$, where ρ is the mass density of the soil. The shear modulus estimated using the above method is a low-strain shear modulus. The shear modulus for the project site were estimated by using the measured shear wave velocity data obtained using a downhole technique in Boring B-3.

Appendix C provides additional background data related to the downhole shear wave velocity values. The estimated shear modulus values are shown in Table 1.

7.3.4 Subgrade Modulus

The subgrade modulus, k_{s1}, is defined as the ratio of stress to deformation for a 1-foot by 1-foot square plate or 1-foot wide beam resting on the subgrade. The subgrade modulus is generally dependent on the relative density of the native soil and the thickness of the compacted foundation structural fill above the native material. The estimated subgrade modulus for the native soils is shown in Table 1. The estimated subgrade modulus values in Table 1 are based on an assumption that footings directly are founded on the native soils. Therefore, in the final design phase, the subgrade modulus should be modified based on the thickness of the compacted working pad and foundation structural fill above the native soils.

7.3.5 Consolidation Settlement Parameters

A one-dimensional consolidation test was performed on a sample of the upper fine-grained alluvium layer, specifically from boring B-6, at a depth of 15 feet. The test sample was classified as soft silt (ML) with trace fine sand and scattered organics. An Atterberg Limits Test resulted LL = 53.6%, PL = 40.8 %, and PI = 13.9%.

The percent strain in the sample was plotted versus the applied test load. Since the interpreted apparent pre-consolidation pressure was slightly above the present overburden pressure, the sample was judged to be essentially normally consolidated. From the strained based consolidation test, soil was judged to normally consolidated based on a reconstructed curve to adjust for potential sample disturbance. The following parameters were estimated based on the results of the consolidation test and our experience:

$C_{c\varepsilon}$	=	0.12
$C_{r\varepsilon}$	=	0.0008
$C_{\alpha\varepsilon}$	=	0.002
Pre-consolidation pressure	=	1,700 psf
OCR	=	slightly over 1

where $C_{c\varepsilon} = \dfrac{C_c}{1 + e_0}$.

For definition of terms, we recommend referring to Holtz and Kovacs, 1981. In our experience with silty soil with organics along the Columbia River, we have seen $C_{c\varepsilon}$ values range from approximately 0.10 to 0.20, depending on the soil consistency and amount of organics.

7.3.6 Coefficient of Sliding Resistance

The lateral loads on the various power facilities, including lateral earth pressures, earthquakes, and wind can be resisted by sliding resistance of the foundation and partial soil passive pressure, which should be estimated in the final design. The coefficient of sliding resistance for concrete on granular materials generally ranges between 0.3 to 0.4. For this site, it is not feasible to place concrete foundations directly on the native soil.

7.3.7 CBR and Resilient Modulus

The native soil subgrade at the plant site is predominately very low strength non-plastic silt to sand with relatively high natural moisture content. For design of flexible pavement sections, we estimate a California Bearing Ratio (CBR) of 1 percent. Also, for use in design of flexible pavement sections, we estimate a resilient modulus (M_R) value of 1,500 psi. The CBR value was estimated by past experience on these types of soils, and use of the soil classification tests performed on the near surface soils. The M_R value was estimated by the commonly used expression (1500 x CBR) presented in AASHTO Guide for Design of Pavement Structures (1993).

7.3.8 Hydraulic Conductivity of Native Soil

Hydraulic conductivity tests have not been conducted on the native soils. However, based on visual soil classification, experience in similar soils along the Columbia River, and comparison to the consolidation test time rates, hydraulic conductivity is expected to be low. The upper silt and silty fine sand is estimated to have a hydraulic conductivity of about 10^{-5} to 10^{-3} cm/sec. The hydraulic conductivity of the underlying very soft silt is estimated to be in the range of 10^{-6} to 10^{-4} cm/sec.

7.3.9 Seismic Soil Profile Type

The seismic soil profile type represents the average condition of the upper 100 feet beneath the site. The Uniform Building Code, 1997 Edition (UBC-97) Soil Profile Type for the site is S_F because the soil is vulnerable to potential failure due to liquefaction occurring in the medium dense silty sand. The designation S_F means that a site-specific evaluation must be conducted.

From our site evaluation, the site is underlain by about 50 feet of loose sandy silt and medium dense silty sand that is susceptible to liquefaction and 20 to 30 feet of very soft silt (PI <20).

7.3.10 Site Response

Site response spectra for the site is presented in Appendix D. The site is classified as a seismically soft site with potential for soil liquefaction to occur above elevation -50 feet. The foundation support system should consider this risk.

7.4 Foundation Alternative Evaluation

To compare foundation support alternatives for the non-heavily loaded structures planned for the site, we have completed a preliminary evaluation of two different support alternatives using two site soil models. These consist of 1) shallow mat foundations, and 2) pile-supported deep foundations. The two different soil models and types of planned structures are:

- Main Plant Area – Typical water tanks planned for construction in the north central portion of the site.
- Cooling Tower Area – A series of multi-cell cooling towers planned near the southeast corner of the site.

Presented below is an estimate of static settlement and seismically induced post-liquefaction settlement for the shallow foundation system. With large amounts of settlements anticipated for these structures, piles for most of the structures may be warranted. A discussion of estimated pile capacities is presented in a later section. Also discussed are possible mitigation measures to reduce settlement.

We have assumed the heavily loaded structures such as turbines, generators, HRSGs, and other settlement sensitive structures would be placed on pile-supported foundations.

7.4.1 Shallow Foundations Main Plant Area

To analyze a typical shallow foundation support alternative, we have assumed a mat foundation with a plan area of 40 feet by 40 feet and a static dead and sustained live load of 500, 1000, 2000, and 3000 psf. A preliminary soil analytical model was developed for this area based on the interpreted subsurface soil conditions, and the results of laboratory tests. A detail of the soil model for the main plant area is presented in Figure 6. For these settlement estimates, the lower sandy alluvium is considered non-compressible.

For static dead load and sustained live loads, estimates of total settlement, including estimates of secondary settlement, are:

For 500 psf:	1 to 2 inches
For 1,000 psf:	3 to 6 inches
For 2,000 psf:	6 to 10 inches
For 3,000 psf:	10 to 15 inches

Settlement at the site may also occur due to earthquake induced post-liquefaction settlement. The extent and level of liquefaction in general, will depend on the severity of ground shaking at the site. Figure 6 shows approximated soil zones that would liquefy during the design level magnitude earthquake that was selected based on the site-specific earthquake and hazard analyses described in Appendix D. We estimate that between 10 and 15 inches of post-liquefaction induced settlement may occur.

Based on these estimates of static and seismic induced settlement, settlement mitigation will be necessary to prevent damage to the structures. For mitigation of static and seismically induced settlement, we suggest supporting the structures on piles. Preloading could mitigate excessive static settlement; however, in our opinion, typical schedule constraints for fast-track power plant projects cannot accommodate the time necessary for conventional preloading approaches. Based on our analysis and experience, we estimate that a preload fill without installing vertical drains in the subsurface should remain in place a minimum of 3 to 4 months to induce the consolidation settlement. Installing vertical wick drains could substantially speed up the time for settlement to occur. Since preloads generally cannot mitigate for seismically induced liquefaction settlement, ground modification construction techniques should be evaluated to densify the sandy liquefiable materials.

7.4.2 Shallow Foundations Cooling Tower Area

To analyze the shallow foundation support alternative for the cooling tower area, we have assumed a mat foundation with a plan area of 40 feet by 450 feet and a static dead load and sustained live load of 500, 1,000, 2,000, and 3,000 psf. A soil analytical model was developed for this area based on the interpreted subsurface soil conditions and the results of laboratory tests. A detail of the soil model for the cooling tower area is presented in Figure 7. For these settlement estimates, lower sandy alluvium is considered non-compressible.

For static dead load and sustained live loads, estimates of total settlement, including estimates of secondary settlement, are:

For 500 psf:	4 to 6 inches
For 1,000 psf:	8 to 12 inches
For 2,000 psf:	12 to 18 inches
For 3,000 psf:	18 to 24 inches

Figure 7 shows our estimate of the soil zones that would liquefy under the same seismic event described in Appendix D. We estimate that between 12 and 18 inches of soil liquefaction induced settlement may occur.

Settlement mitigation will again be necessary to prevent structural damage to the structures. The settlement mitigation measures described above also apply to this area.

7.4.3 Deep Foundations for the Site

As previously discussed, the preliminary analytical soil models presented on Figures 6 and 7 show a layer of very soft compressive silt, and layers of very loose to medium dense liquefiable sandy silt to sand up to a depth of 60 feet below the existing ground surface. Since this surface condition results in very large estimated settlements, pile-supported foundations should be considered for all the settlement sensitive plant facilities or the seismically designed facilities. We recommend that the minimum pile embedment be 80 feet which includes at least 20 feet below the bottom of the potentially liquefiable layers to account for variability of subsurface conditions at the site. We recommend additional subsurface explorations including use of the Cone Penetration Test (CPT) to better define the thickness of the compressible soil layers.

For preliminary evaluation, we analyzed piles consisting of 12¾-inch and 16-inch diameter driven closed-end, steel pipe piles. Pipe piles should conform to the requirements of ASTM A252, Specifications for Welded and Seamless Steel Pipe Piles. We assumed the pipe piles would be fitted with a welded flat plate.

The allowable compressive and uplift capacities of the driven closed-end, steel pipe piles were evaluated under both static and seismic conditions with capacity estimates in Table 2. For the static compression condition, a nominal soil shaft friction was used for the upper 60-foot compressible zone. The allowable compressive values have a factor of safety equal to or slightly greater than 3. For the seismic compression condition, the upper 60-foot compressible zone was assumed to provide no soil shaft friction resistance and apply no downdrag or negative skin friction to the pile. The allowable seismic compressive values have a factor of safety equal to or slightly above 2. For the allowable static uplift capacities shown in Table 3, the 60-foot compressible zone was treated in the same manner as for compression. The factor of safety for the static allowable uplift condition is equal to or greater than 3. The factor of safety for the seismic allowable condition is equal to or greater than 1.

Table 2
Allowable Compressive Pile Capacities

Pile Depth (ft)	12¾-inch Dia. (kips)		16-inch Diameter (kips)	
	Static	Seismic	Static	Seismic
70	80	65	120	100
80	100	85	150	130
90	125	110	190	170

Table 3
Allowable Uplift Pile Capacities

Pile Depth (ft)	12¾-inch Dia. (kips)		16-inch Diameter (kips)	
	Static	Seismic	Static	Seismic
70	50	30	75	55
80	65	45	95	75
90	85	65	120	100

The above compressive and uplift capacities with the pile embedment lengths shown should result in less than ½-inch settlement. The allowable capacities assume no reduction for group effects and that all piles are driven no closer than 3 pile diameters center-to-center. Also, to maintain spacing, we assume piles would be driven with a maximum deviation from vertical of not more than 3 percent (1.5 inches in 4 feet).

The proposed structures will be subject to lateral loads due to wind and earthquake forces. The lateral load capacities of these pipe piles were evaluated for both static and seismic loading conditions. The laterally loaded pipe pile analyses were performed with the aid of the computer program "LPILE". Two pile sections, PP12¾ X 0.375 and PP16 X 0.375, under a free-pile head condition were evaluated. For these values a reduction for group action was not considered and no lateral resistance was assumed form passive resistance from an embedded pile cap. Based upon our evaluation, the single pipe piles, PP12¾ X 0.375 and PP16 X 0.375, can provide 4 kips and 6 kips, allowable lateral capacities, respectively, under static loading condition and horizontal deflection of approximately ½-inch. Included is a factor of safety equal to about 2.0. Under seismic loading conditions, the allowable lateral capacities of the piles should be reduced to about 50 percent of the static condition. The results of the computer analyses showed an approximate depth to fixity below the top of the pile as follows:

PP12¾ X 0.375	25 feet
PP16 X 0.375	30 feet

7.4.4 Settlement Sensitive Pipes, Pipe Racks, and Conduits

We estimate that differential static settlement between pipe racks, utility conduits and pipelines (i.e., linear facilities) may occur between structures with different foundation support systems. In addition, seismic induced liquefaction settlement could have a significant impact on settlement sensitive linear facilities. If these facilities cannot tolerate the settlement magnitudes estimated, we suggest deep foundation be considered. If linear facilities are allowed to settle, we recommend evaluating special pipe joints and connections, sleeves, shorter pipe lengths, and other methods to help mitigate such settlement and possible infrastructure damage. Also, we recommend that settlement analyses based on the type, depth, and difference in settlement tolerance between the planned structures be completed to evaluate the impact on these type of structures.

7.4.5 Lateral Earth Pressure

Lateral earth pressure on retaining walls depend on the type of wall (i.e., yielding or non-yielding), the type and method of placement of backfill against the wall, the magnitude of surcharge during construction or permanent loads on the ground surface adjacent to the wall, the slope of the backfill, location of the ground water level, use of positive drainage systems behind wall, and the design criteria such as static or seismic condition, and combination loading conditions. Based on the nature of the native soil at the site, it is our opinion that the native soil should not be used for backfill, and backfill material should be imported. For retaining wall backfill, import material consisting of free-draining, crushed rock would be the most desirable.

7.4.6 Roadways

Construction staging areas, roadways, and parking areas constructed on these loose and soft subsurface materials will require special consideration for subgrade stabilization. The subgrade bearing values for the native materials are estimated to be extremely low; therefore the use of geotextile, geogrids, and free-draining imported crushed rock should be considered to develop an adequate zone of subbase strength. Also, the consideration of maintaining drained subbase base material should also be considered.

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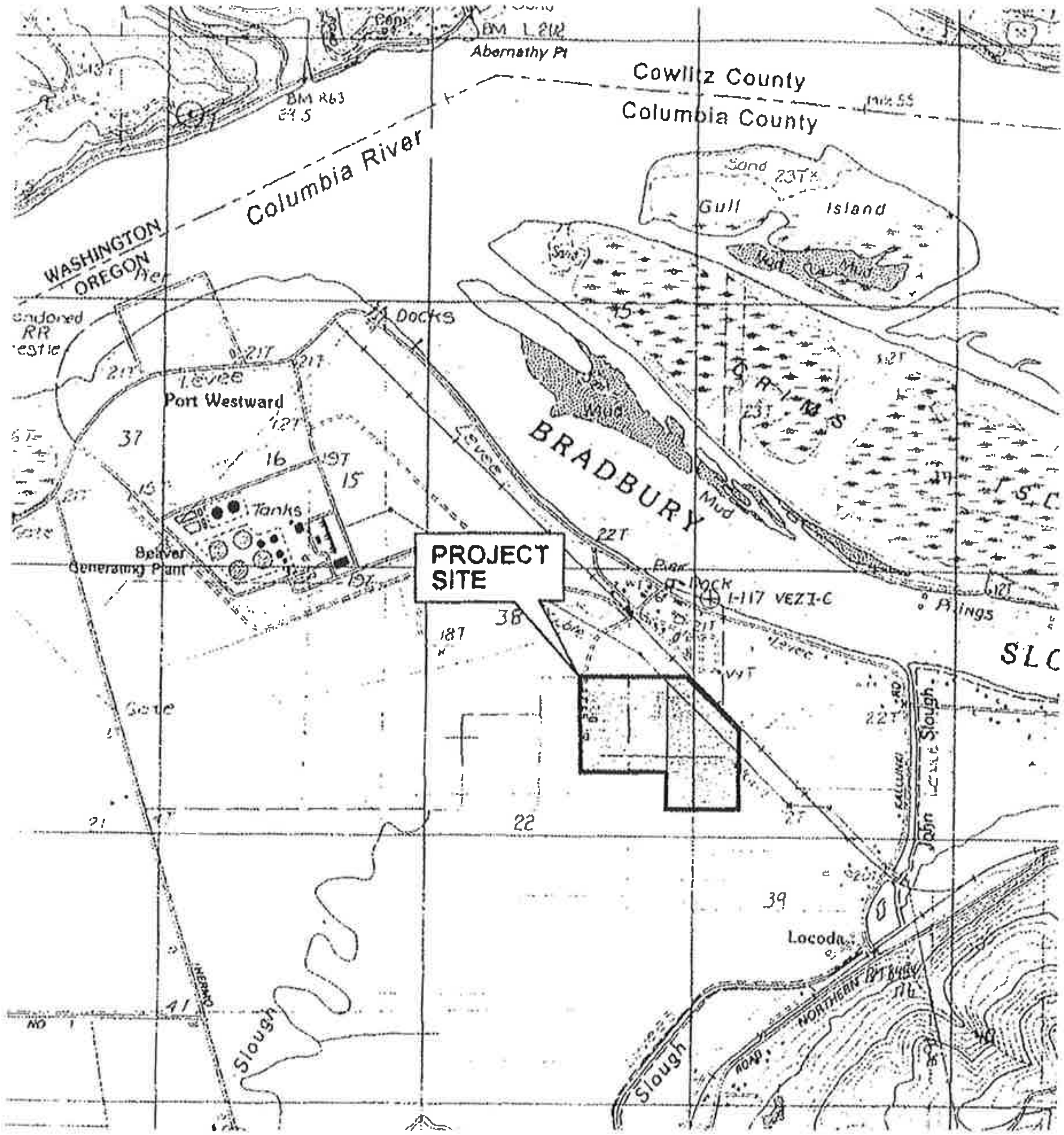
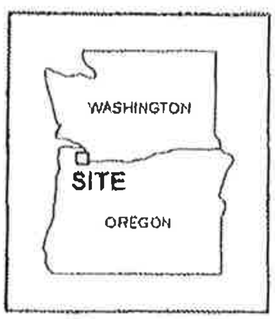
Note: A more comprehensive list of references for the Site Specific Geological and Soil Stability Assessment is contained in Appendix D.





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SCALE IN FEET

0 2000 4000

Source:
USGS 7.5 minute quadrangle map.
"OAK POINT, WASH,-OREG."

SUMMIT/WESTWARD ENERGY PROJECT
CLATSKANIE, OREGON

VICINITY MAP

SQUIER ASSOCIATES

FIGURE 1

NOTE: Features shown are for illustrative purposes only and are approximate.

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TABLE 2

DC Resistivity Models
Summit/Westward Energy Project
Clatskanie, Oregon

Models

Sounding	Layer 1 Resistivity (ohm-m)	Layer 1 Depth (Feet)	Layer 2 Resistivity (ohm-m)	Layer 2 Depth (Feet)	Layer 3 Resistivity (ohm-m)	Layer 3 Depth (Feet)	Layer 4 Resistivity (ohm-m)	Model Misfit % Error
R-1	204	1.5	105	15.4	18	40	52	3.1%
R-2	161	4.7	67	18.1	19	46	57	5.2%
R-3	122	2.9	98	14.3	18	39	57	1.9%
R-4	102	3.2	57	14.5	17	36	51	2.3%
R-5	148	1.5	87	7.3	35	32	49	0.7%
R-6	213	3.4	72	15.8	21	40	54	1.9%

TABLE 1

a-spacing (feet)	Apparent Resistivity (Ohm-m)	Apparent Resistivity (Ohm-ft)	V/I (Ohms)	Error	Current (mA)
Sounding R-5					
Boring B-6	E-W Sounding offset 20 feet north of B-6(new)				
3.0	111.3	319.9	1.70E+01	0.1%	100
4.0	99.2	285.1	1.13E+01	0.0%	100
5.0	91.0	261.5	8.32E+00	0.0%	100
7.0	79.8	229.2	5.21E+00	0.0%	100
10.0	67.2	193.2	3.08E+00	0.0%	100
15.0	52.6	151.0	1.60E+00	0.0%	100
20.0	45.0	129.2	1.03E+00	0.0%	100
25.0	42.8	123.0	7.83E-01	0.0%	100
30.0	42.0	120.8	6.41E-01	0.1%	100
40.0	41.4	119.0	4.74E-01	0.0%	100
50.0	42.3	121.5	3.87E-01	0.1%	100
70.0	43.3	124.4	2.83E-01	0.0%	100
100.0	45.3	130.1	2.07E-01	0.0%	100
130.0	46.6	134.0	1.64E-01	0.0%	100
160.0	47.4	136.3	1.36E-01	0.0%	100
Sounding R-6					
Boring B-4	E-W Sounding offset 10 feet north of B-4				
3.0	181.2	520.8	2.78E+01	0.0%	100
4.0	170.2	489.0	1.95E+01	0.0%	100
5.0	147.0	422.5	1.34E+01	0.0%	100
7.0	115.8	332.9	7.57E+00	0.1%	100
10.0	87.4	251.1	4.00E+00	0.0%	100
15.0	66.4	190.8	2.02E+00	0.0%	100
20.0	52.4	150.5	1.20E+00	0.0%	100
25.0	46.1	132.5	8.43E-01	0.1%	100
30.0	40.5	116.3	6.17E-01	0.0%	100
40.0	35.1	100.7	4.01E-01	0.1%	100
50.0	34.5	99.1	3.15E-01	0.1%	100
70.0	36.4	104.6	2.38E-01	0.0%	100
100.0	41.6	119.6	1.90E-01	0.0%	100
130.0	44.6	128.0	1.57E-01	0.0%	100
160.0	47.7	137.2	1.36E-01	0.6%	100
200.0	46.2	132.8	1.06E-01	0.0%	100

END

TABLE 1

a-spacing (feet)	Apparent Resistivity (Ohm-m)	Apparent Resistivity (Ohm-ft)	V/I (Ohms)	Error	Current (mA)
Sounding R-3					
B-7 offset	E-W Sounding offset 100 feet east of B-7				
3.0	120.2	345.4	1.83E+01	0.3%	100
4.0	113.3	325.6	1.30E+01	0.0%	100
5.0	107.3	308.3	9.81E+00	0.0%	100
7.0	98.2	282.3	6.42E+00	0.0%	100
10.0	91.4	262.7	4.18E+00	0.0%	100
15.0	77.2	221.8	2.35E+00	0.1%	100
20.0	60.8	174.6	1.39E+00	0.0%	100
25.0	48.4	139.1	8.86E-01	0.0%	100
30.0	41.9	120.3	6.38E-01	0.0%	100
40.0	33.9	97.4	3.88E-01	0.0%	100
50.0	32.6	93.6	2.98E-01	0.0%	100
70.0	35.5	101.9	2.32E-01	0.0%	100
100.0	40.1	115.2	1.83E-01	0.0%	100
130.0	43.6	125.2	1.53E-01	0.0%	100
160.0	45.4	130.5	1.30E-01	0.2%	100
Sounding R-4					
	E-W Sounding offset 300 feet east of B-7				
3.0	92.0	264.4	1.40E+01	0.0%	100
4.0	86.3	248.0	9.87E+00	0.0%	100
5.0	82.8	238.0	7.57E+00	0.0%	100
7.0	69.5	199.8	4.54E+00	0.0%	100
10.0	58.8	169.0	2.69E+00	0.0%	100
15.0	48.7	139.9	1.48E+00	0.0%	100
20.0	40.5	116.3	9.26E-01	0.1%	100
25.0	35.8	103.0	6.55E-01	0.0%	100
30.0	31.9	91.7	4.87E-01	0.0%	100
40.0	28.5	81.9	3.26E-01	0.0%	100
50.0	31.7	91.2	2.90E-01	0.1%	100
70.0	32.3	93.0	2.11E-01	0.0%	100
100.0	37.5	107.8	1.72E-01	0.0%	100
130.0	41.3	118.8	1.45E-01	0.0%	100
160.0	42.0	120.7	1.20E-01	0.0%	100

TABLE 1

DC Resistivity Soundings

Wenner Array
Summit/Westward Energy Project
Clatskanie, Oregon

a-spacing (feet)	Apparent Resistivity (Ohm-m)	Apparent Resistivity (Ohm-ft)	V/I (Ohms)	Error	Current (mA)
Sounding R-1					
Boring B-5	N-S Sounding offset 10 feet west of B-5				
3.0	138.2	397.1	2.11E+01	0.0%	20
4.0	135.7	389.8	1.55E+01	0.0%	20
5.0	112.9	324.4	1.03E+01	0.0%	20
7.0	103.5	297.4	6.76E+00	0.0%	20
10.0	96.2	276.3	4.40E+00	0.0%	20
15.0	84.6	243.0	2.58E+00	0.0%	20
20.0	69.1	198.5	1.58E+00	0.0%	20
25.0	54.3	156.1	9.94E-01	0.0%	20
30.0	44.5	127.9	6.79E-01	0.0%	50
40.0	35.5	102.1	4.06E-01	0.0%	50
50.0	33.4	96.0	3.06E-01	0.3%	50
70.0	34.4	98.8	2.26E-01	0.0%	50
100.0	38.0	109.3	1.74E-01	0.0%	100
150.0	42.2	121.3	1.29E-01	0.0%	100
Sounding R-2					
Boring B-3	N-S Sounding offset 10 feet east of B-3				
3.0	139.4	400.7	2.13E+01	0.7%	100
4.0	136.0	390.7	1.55E+01	0.8%	100
5.0	146.6	421.2	1.34E+01	1.0%	100
7.0	113.3	325.5	7.40E+00	0.2%	100
10.0	84.7	243.5	3.87E+00	0.1%	100
15.0	70.6	203.0	2.15E+00	0.1%	100
20.0	59.0	169.4	1.35E+00	0.0%	100
25.0	47.2	135.5	8.63E-01	0.2%	100
30.0	41.0	117.8	6.25E-01	0.0%	100
40.0	34.2	98.2	3.91E-01	0.1%	100
50.0	33.0	95.0	3.02E-01	0.0%	100
70.0	34.5	99.3	2.26E-01	0.0%	100
100.0	39.5	113.5	1.81E-01	0.1%	100
130.0	42.7	122.6	1.50E-01	0.2%	100
160.0	44.6	128.2	1.28E-01	0.0%	100

Geophysical Services

Environmental • Groundwater • Geotechnical

D.C. Resisitivity

INTRODUCTION

D.C. resistivity (electrical resistivity) techniques measure earth resistivity by driving a direct current (D.C.) signal into the ground and measuring the resulting potentials (voltages) created in the earth. From the data the electrical properties of the earth (the geoelectric section) can be derived. In turn, from those electrical properties we can infer geologic properties of the earth.

In geophysical and geotechnical literature, the terms "electrical resistivity" and "D.C. resistivity" are used synonymously. The term "vertical electric sounding" (VES) is also used to refer to soundings using the D.C resistivity method. The terms "resistivity" or "electrical" are often used to refer to the same methods or techniques, although "electrical" is sometimes used to encompass a broader range of techniques including the electromagnetic methods.

APPLICATIONS

Electrical resistivity of soils and rocks correlates with other soil/rock properties which are of interest to the geologist, hydrogeologist, geotechnical engineer and/or quarry operator. Several geologic parameters which affect earth resistivity (and its reciprocal, conductivity) include:

- clay content,
- groundwater conductivity,
- soil or formation porosity, and
- degree of water saturation.

D.C. resistivity techniques may be used in the profiling mode (dipole-dipole surveys) to map lateral changes and identify near-vertical features (e.g., fracture zones), or they may be used in the sounding mode (e.g., Schlumberger soundings) to determine depths to geoelectric horizons (e.g., depth to saline groundwater).

Common applications of the D.C. resistivity method include:

- delineation of aggregate deposits for quarry operations
- measuring earth impedance or resistance for electrical grounding circuits or for cathodic protection,
- estimating depth to bedrock, to the water table, or to other geoelectric boundaries, and
- mapping and/or detecting other geologic features.

D.C. resistivity and electromagnetic (EM) techniques both measure electrical properties of the earth, and hence both are used for many of the same applications. Conductivity, which is often reported by EM instruments, is the reciprocal of resistivity.

THEORY OF OPERATION

Figure 2 is a schematic diagram showing the basic principle of D.C. resistivity measurements. Two short metallic stakes (electrodes) are driven about 1 foot into the earth to apply the current to the ground. Two additional electrodes are used to measure the earth voltage (or electrical potential) generated by the current.

Depth of investigation is a function of the electrode spacing. The greater the spacing between the outer current electrodes, the deeper the electrical currents will flow in the earth, hence the greater the depth of exploration. The depth of investigation is generally 20% to 40% of the outer electrode spacing, depending on the earth resistivity structure.

(Continued Next Page)

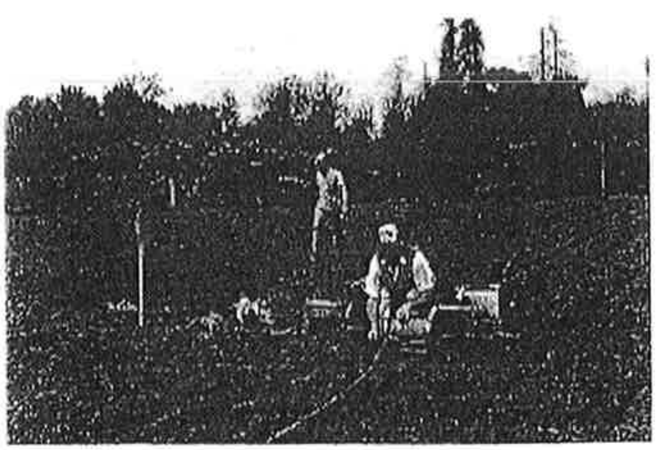

Figure 1 - D.C. Resistivity Crew In Operation In The Willamette Valley of Oregon

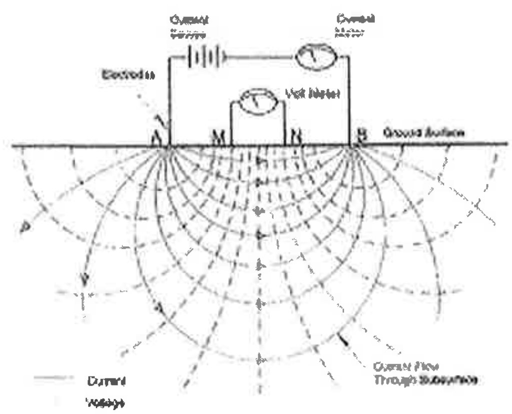

Figure 2 - Schematic Illustrating Basic Concept Of Electrical Resistivity Measurement

...hwest Geophysical Associates, Inc.
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D.C. Resitivity

ATA ANALYSIS & INTERPRETATION

Apparent Resistivity:
Instrument readings (current and voltage) are generally reduced to "apparent resistivity" values. The apparent resistivity is the resistivity of the homogeneous half-space which would produce the observed instrument response for a given electrode spacing. Apparent resistivity is a weighted average of soil resistivities over the depth of investigation.

For soundings a log-log plot of apparent resistivity versus electrode separation is obtained. This is sometimes referred to as the "sounding curve."

Modeling:
Resistivity data is generally interpreted using the "modeling" process: A hypothetical model of the earth and its resistivity structure (geoelectric sections) is generated. The theoretical electrical resistivity response over that model is then calculated. The theoretical response is then compared with the observed field response and differences between observed and calculated are noted. The hypothetical earth model is then adjusted to create a response which more nearly fits the observed data. When this iterative process is automated it is referred to as "iterative inversion" or "optimization."

iqueness
..esistivity models are generally not unique; i.e., a large number of earth models can produce the same observed data or sounding curve. In general, resistivity methods determine the "conductance" of a given stratigraphic layer or unit. The conductance is the product of the resistivity and the thickness of a unit. Hence that layer could be thinner and more conductive or thicker and less conductive, and produce essentially the same results. Hence constraints on the model, from borehole data or assumed unit resistivities, can greatly enhance the interpretation.

Deliverables
The end product from a D.C. resistivity survey is generally a "geoelectric" cross section showing thicknesses and resistivities of all the geoelectric units or layers. If borehole data or a conceptual geologic model is available, then a geologic identity can be assigned to the geoelectric units.

A two-dimensional geoelectric section may be made up of a series of one-dimensional soundings joined together to form a two-dimensional section, or it may be a continual two-dimensional cross section. The type of section produced depends on the acquisition parameters and the type of processing applied to the data.

Figure 3 is a two dimensional geoelectric section from a dipole-dipole survey in Alaska. The resistivity survey, part of a water resources investigation, was conducted in order to identify fracture zones with increased porosity. The geophysical objective was to locate conductive fracture zones in the more resistive bedrock. The zone with lower resistivities (1500 to 2000 ohm-meters), which is seen in Figure 3 between 90m and 100m, is indicative of increased water content due to higher fracture porosity in that region.

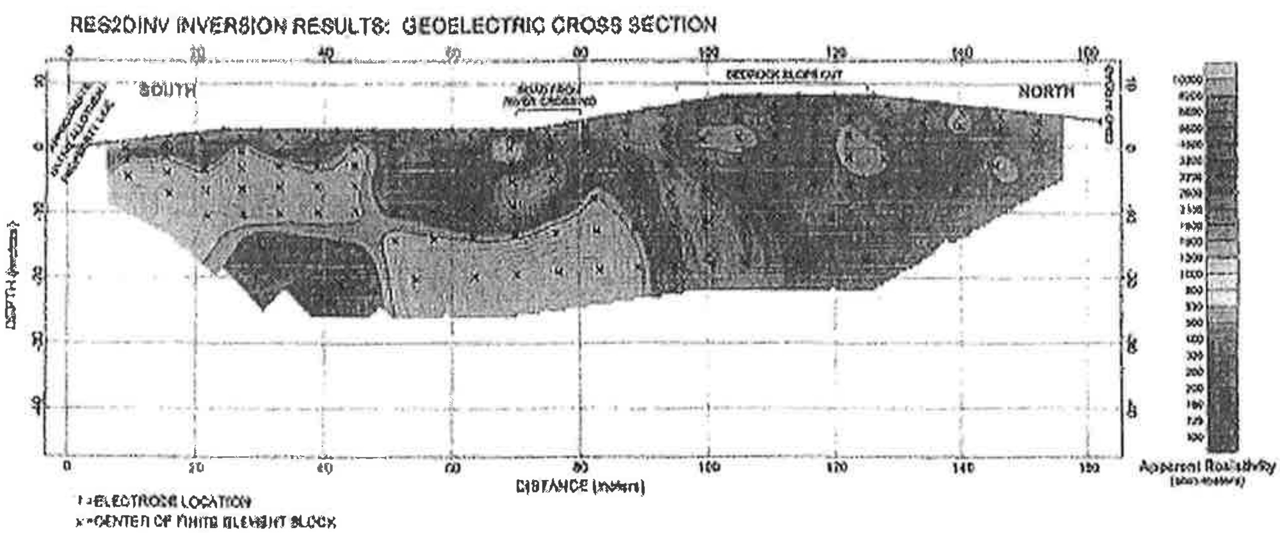

Figure 3 - Geoelectric Model From Dipole-Dipole Resistivity Survey