Permit Storm Water Report

For

Lincoln Sands Center Building

Lincoln, Oregon

Date: November 9th, 2022

Prepared for:
Open Concept
Architecture and
O' Brien & Company LLC

Prepared by:
Humber Design Group, Inc.
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Portland, OR 97214
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(503) 946-5358
Report By: Min Chan Song
Supervised By:
Kristian McCombs

City File No:
Humber Design Group, Inc. No: OCA006

Engineer's Certification

The technical information and data contained in this report were prepared under the direction and supervision of the undersigned, whose seal, as a professional engineer licensed as such, is affixed below.

87835PE
OREGON
SEPT 12, 2017
KRISTIAN ANN MCCOMBS

EXPIRES 12-31-2022

Humber
Design
Group,
Inc.

Table of Contents

Project Overview

Purpose of Report

This purpose of this report is to analyze the impact the proposed development will have on the existing downstream stormwater conveyance system, and document the criteria used to design the proposed stormwater facilities. Source information used to define the different features of the site is also provided.

Project Description and Location

The Lincoln Sands Center Building project is located in the Lincoln City, Oregon along the intersection of NW Inlet Ave and NW 5th Ct (Tax Lots 8800, 10200, 10300, and 10400). The total site area is approximately 0.39 acres.

0.27 acres of the 0.39-acre development area will be considered impervious surface. This evaluation will demonstrate that the proposed water quality systems will adequately treat any on-site runoff, and the downstream stormwater facility will meet the code requirements for water conveyance outlined in the Lincoln City Design Standards Chapter 3 – Stormwater Systems.

Existing Conditions

The property is located approximately 500 feet east of the Pacific Ocean and approximately 1500 feet west of Devils Lake. The site contains hotel, private residence building, and pavement parking area. The site drains to either a catch basin connected to the city storm system at NW 5th Ct, or drain though internal catch basins and storm pipes to the beach and Pacific Ocean.

Per the Geotechnical Engineering Report provided by NV5 on August 19th, 2022, infiltration testing result is 35 in/hr at a depth of 5 feet BGS. Subsurface soils were sampled by drilling two borings to depths of 20 and 66.5 feet BGS, respectively. Sand was encountered below the ground surface, with varying proportions of silt. Groundwater was measured at approximately 10.9 and 15.2 feet BGS.

Developed Conditions

The proposed development will be a new 5 story hotel with sidewalk and new access road from NW 5th Ave to existing hotel parking area.

Offsite

There will be no off-site improvements.

Downstream Conveyance

Any stormwater leaving the site will be conveyed south of the site into the existing storm system along NW 5th Court Ave.

Regulatory Design Criteria

Stormwater Quantity Management-Design Criteria

The onsite stormwater facilities will be designed to meet the requirements listed in Chapter 3 – Stormwater System of the Lincoln City Design Standards. The design will be sized using the 2-

year, 10-year, and 25-year, 24-hour events. According to these requirements, post-development flow rates will be designed to be no greater than pre-development outflow during the 2-year, 10-year, and 25-year storm events. Pre-developed conditions are considered the configuration of the site immediately prior to development. In addition to outflow control, stormwater treatment of 50% for the 2-year event is required.

The design of the stormwater quantity facilities used the following criteria to analyze the performance of the system:

Storm Event	Rainfall Depth (in)
2-yr	4.2
10-yr	5.7
25-yr	6.5

- A Tc of 5 minutes was used in calculations involving the post-developed site conditions.
- The Santa Barbara Urban Hydrograph (SBUH) method was used to estimate the stormwater runoff for the site. See HydroCAD Calculations in Appendix E.
- According to the USDA soil survey, 100% of the soil on the proposed site consists of is Winema-Fendall silt loams, 3 to 15 percent slopes
- All impervious, and pervious areas use runoff curve numbers (CN) of 98, and 79 respectively.

Potential Site Pollutants

The Department of Environmental Quality (DEQ) recognizes sediments, metals, various petroleum products, nutrients, pesticides, herbicides, and fungicides as common pollutants found in residential developments.

Maintenance Plan

All stormwater facilities on-site will be the responsibility of the property owner to maintain. The property owner will also agree to any maintenance standards set forth by the Lincoln City.

Design Methodology

To meet the Lincoln City water quality design standards, on-site water quality will be designed to treat at least 50% of the 2-year, 24-hour event.

Design Parameters

Existing Site Conditions

The site contains Asphalt pavement, concrete curb, retaining wall, and mixed vegetation bush and grasses.

Soil Type

According to the Geotechnical Engineering Report provided by NV5 on August 19th, 2022, subsurface soils were sampled by drilling two borings to depths of 20 and 66.5 feet BGS, respectively. Sand was encountered below the ground surface, with varying proportions of silt. Groundwater was measured at approximately 10.9 and 15.2 feet BGS.

Post Developed Site Conditions

The proposed development will be a new 5 story hotel with sidewalk and new access road from NW 5th Ave to existing hotel parking area.

Calculation Methodology

HydroCAD version 10.00 was used to calculate all stormwater runoff quantities. The Santa Barbara Urban Hydrograph was used in conjunction with the SCS Type 1A 24- hour storm region.

Proposed Stormwater Conduit Sizing and Inlet Placement

All stormwater line sizes will be calculated using Manning’s equation for a SBUH 25- year storm event.

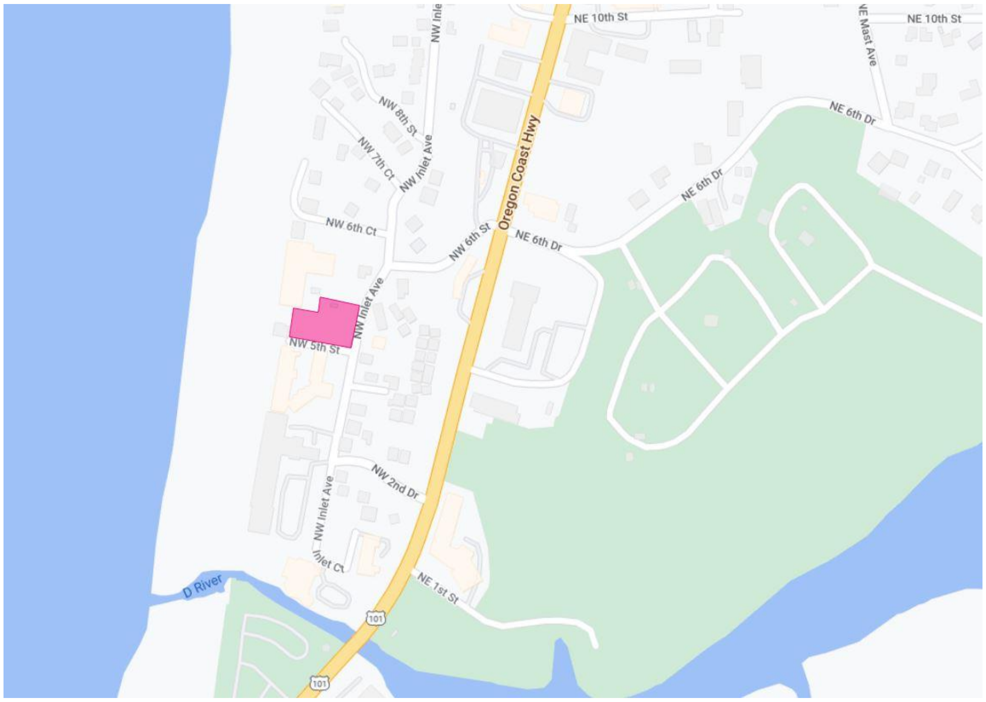
Proposed Stormwater Quantity Control Facility Design

The post-developed outflow rates for the 2-year, the 10-year, and the 25-year storm events are equal to or less than the pre-developed outflow of both the on-site and off-site over detention per Lincoln City /NOAA design standards. Refer to Appendix D for water quantity calculations.

Infiltration System	**25-Year (cfs)**	**Area (sqft)**	**Catchment**	**Destination**
Storm Planter	0.11	280	3,304 SF	Ground
Rock Infiltration	0.22	500	8,436 SF	Ground

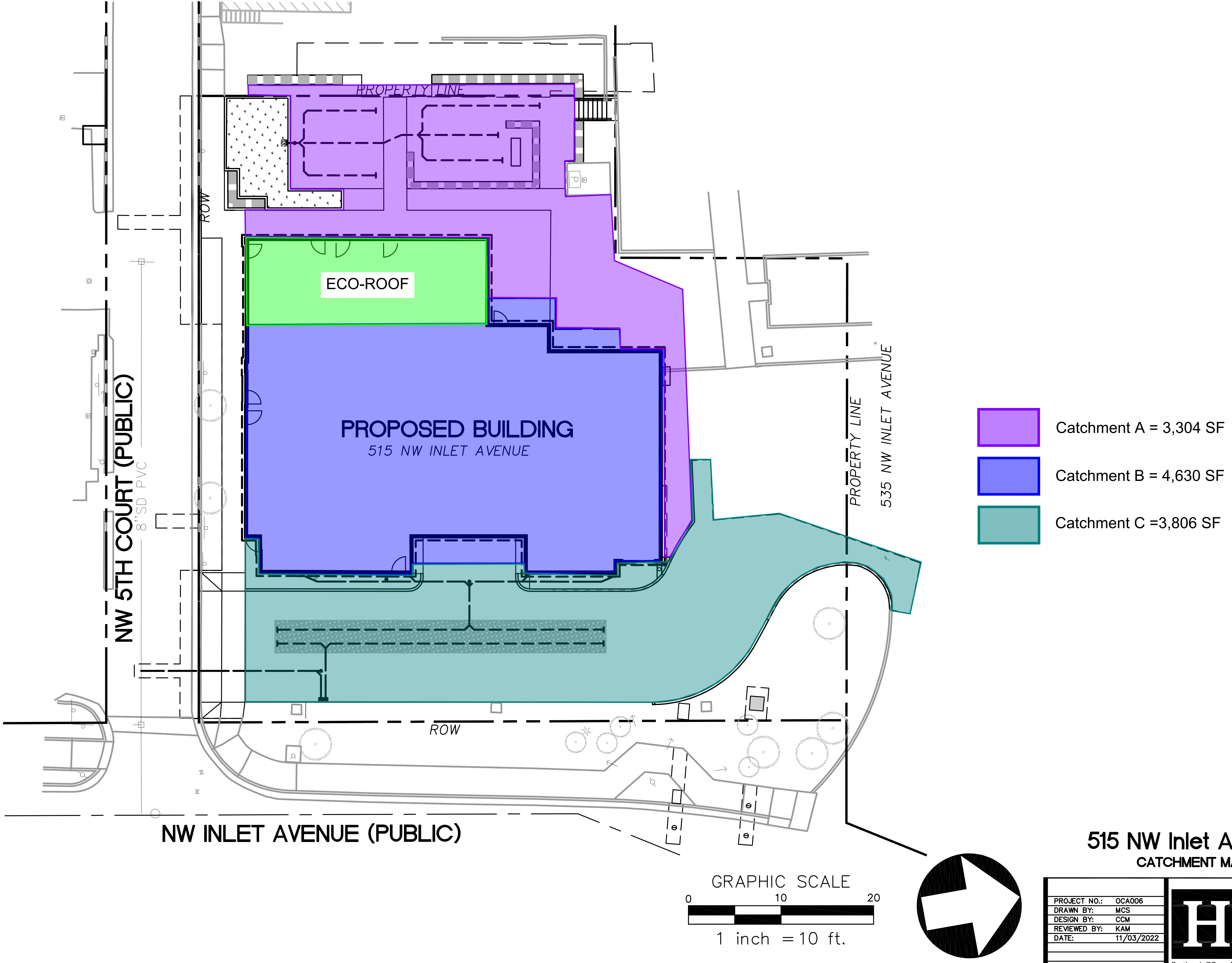
Appendix A

Vicinity Map



Appendix B

Postdeveloped Basin Map



515 NW Inlet Avenue

CATCHMENT MAP

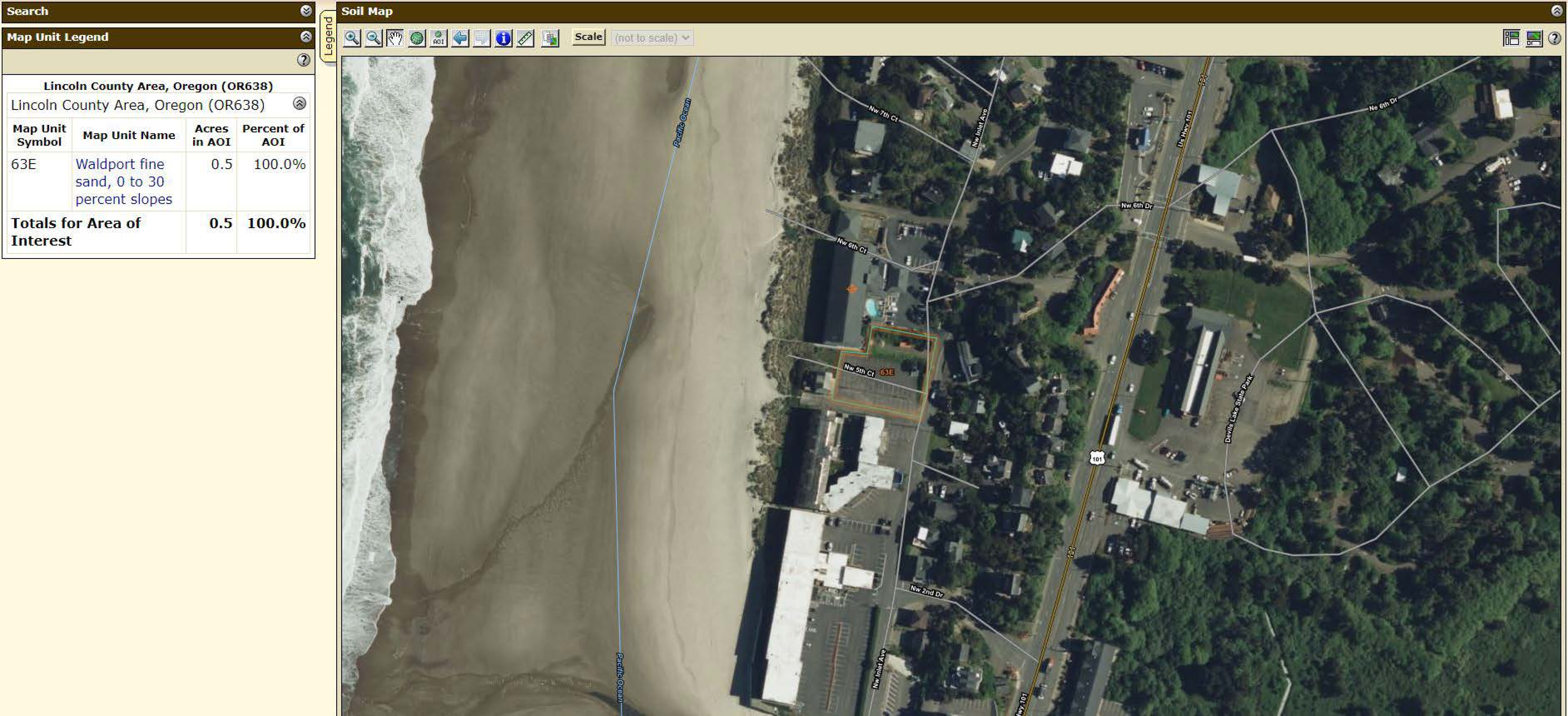
PROJECT NO.:	OCA006
DRAWN BY:	MCS
DESIGN BY:	CCM
REVIEWED BY:	KAM
DATE:	11/03/2022

Humber Design Group, Inc.

Portland, OR ▪ 503.946.6690 ▪ hdgpdx.com

Appendix C

Soils Information



Lincoln County Area, Oregon (OR638)

Lincoln County Area, Oregon (OR638)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
63E	Waldport fine sand, 0 to 30 percent slopes	0.5	100.0%
Totals for Area of Interest		**0.5**	**100.0%**

Appendix D

Water Quantity Calculations

Summary for Subcatchment A: Catchment A

[49] Hint: Tc<2dt may require smaller dt

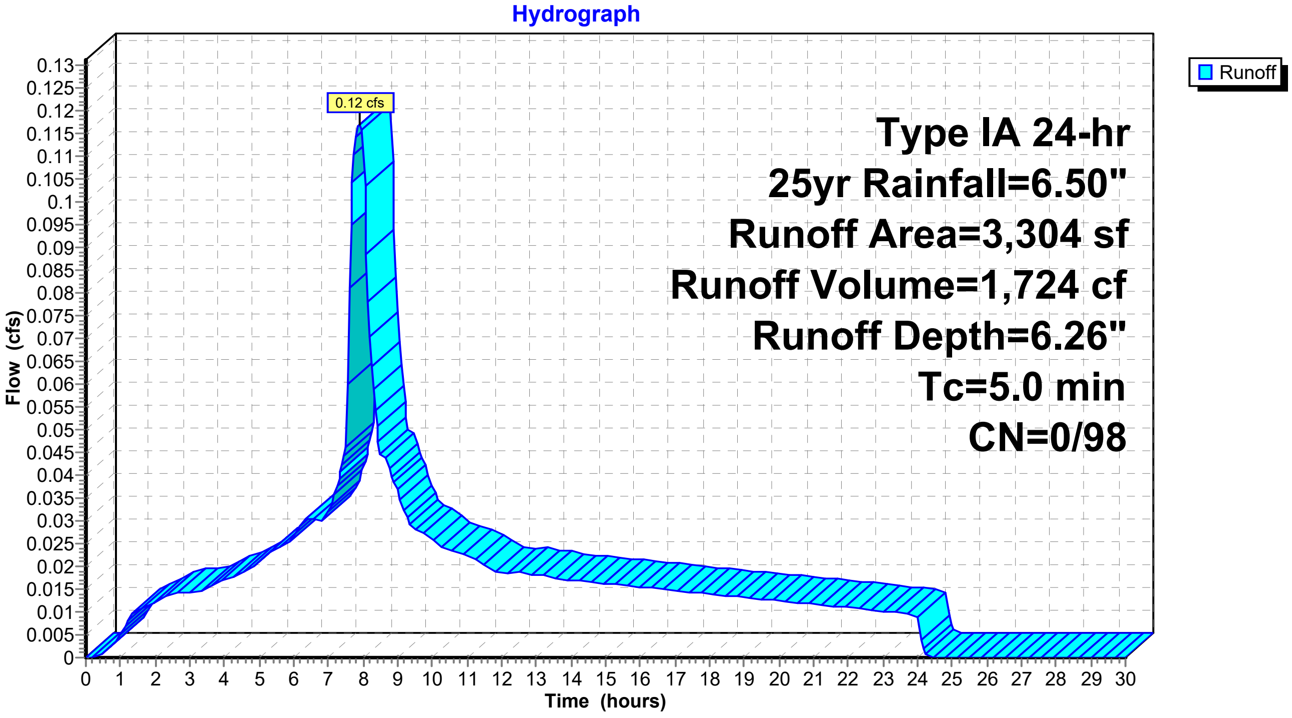
Runoff = 0.12 cfs @ 7.89 hrs, Volume= 1,724 cf, Depth= 6.26"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 25yr Rainfall=6.50"

	Area (sf)	CN	Description
*	3,304	98	
	3,304		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry,**

Subcatchment A: Catchment A



Summary for Subcatchment 3S: Catchment B

[49] Hint: Tc<2dt may require smaller dt

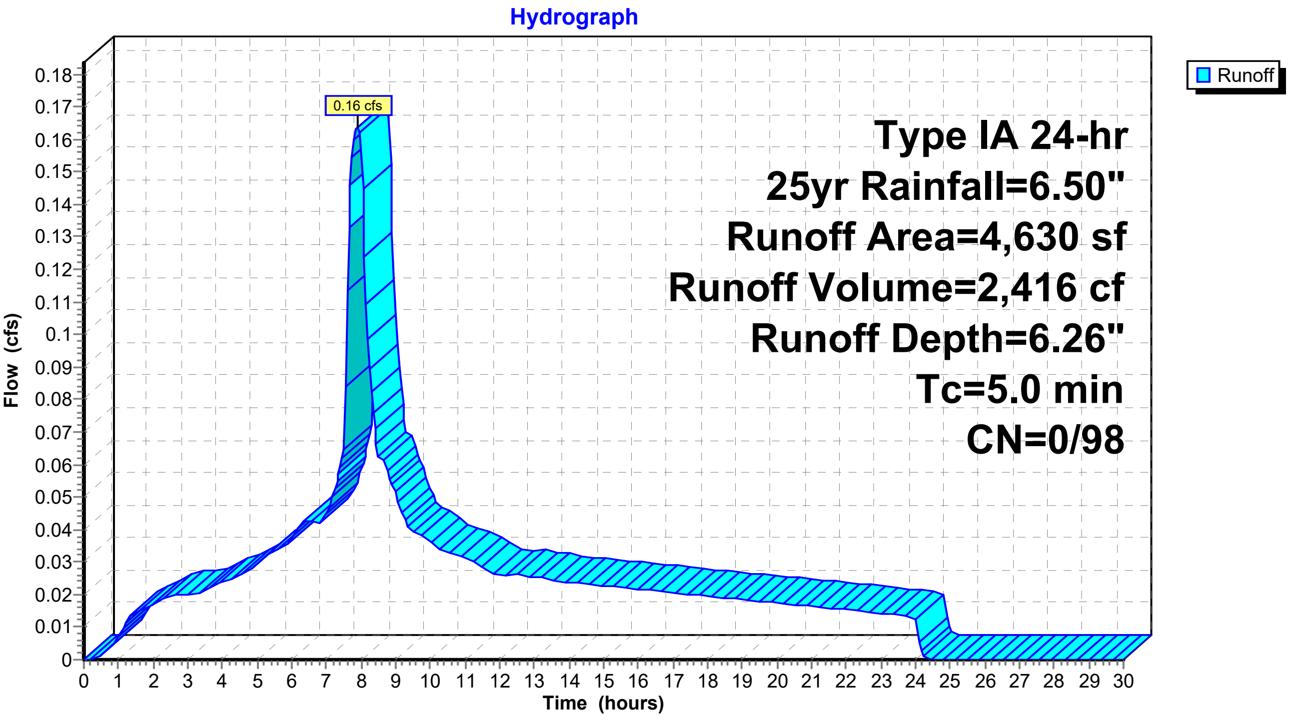
Runoff = 0.16 cfs @ 7.89 hrs, Volume= 2,416 cf, Depth= 6.26"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 25yr Rainfall=6.50"

	Area (sf)	CN	Description
*	4,630	98	
	4,630		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry,**

Subcatchment 3S: Catchment B



Summary for Subcatchment 6S: Catchment C

[49] Hint: Tc<2dt may require smaller dt

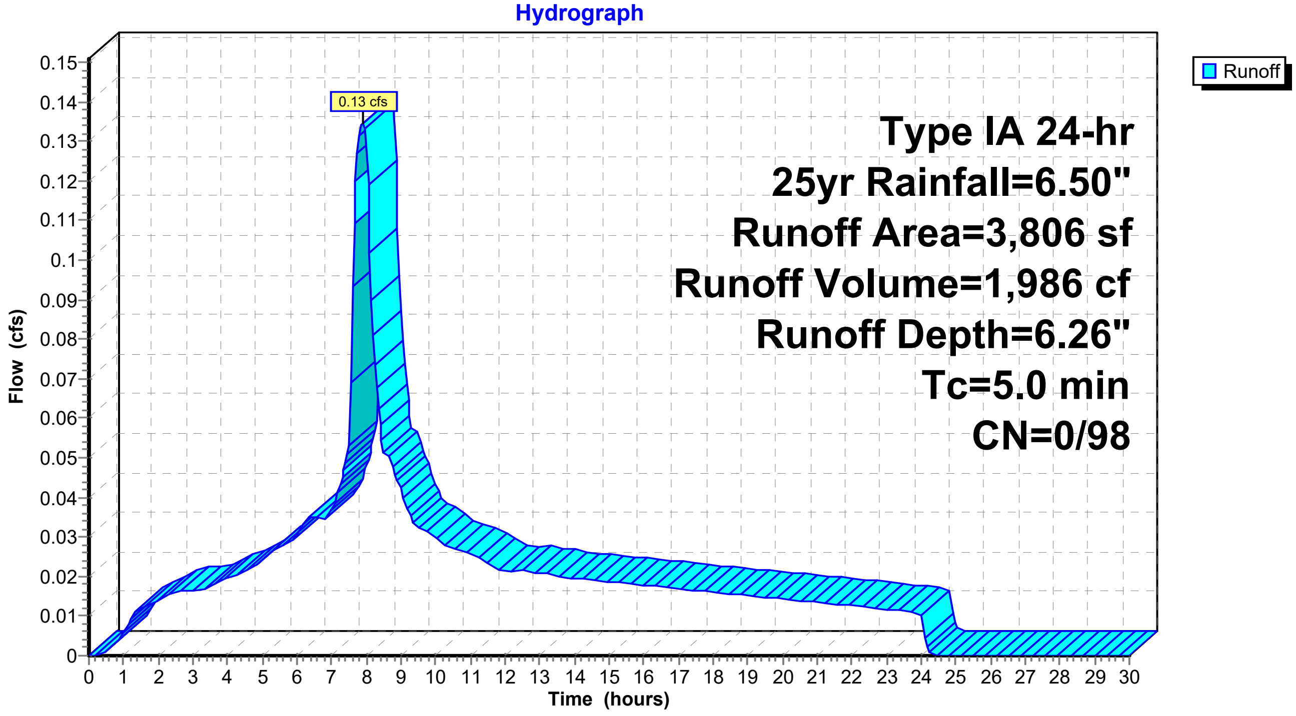
Runoff = 0.13 cfs @ 7.89 hrs, Volume= 1,986 cf, Depth= 6.26"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 25yr Rainfall=6.50"

	Area (sf)	CN	Description
*	3,806	98	
	3,806		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry,**

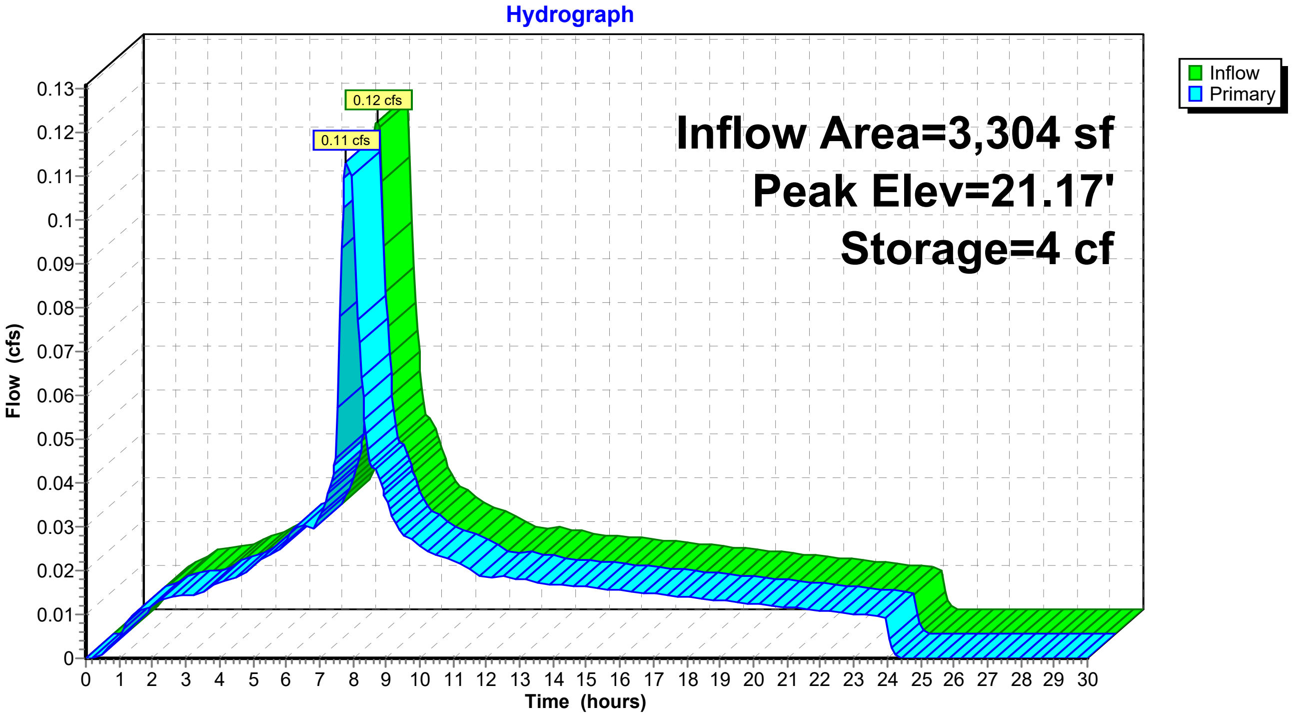
Subcatchment 6S: Catchment C



Summary for Pond 1P: Outdoor Planter (custom)

```
Inflow Area =          3,304 sf,100.00% Impervious,  Inflow Depth =  6.26"    for 25yr event
Inflow      =       0.12 cfs @   7.89 hrs,  Volume=           1,724 cf
Outflow     =       0.11 cfs @   7.80 hrs,  Volume=           1,724 cf,  Atten= 3%,  Lag= 0.0 min
Primary     =       0.11 cfs @   7.80 hrs,  Volume=           1,724 cf

Routing by Stor-Ind method, Time Span= 0.00-30.00  hrs, dt= 0.05 hrs
Peak Elev= 21.17' @ 7.99 hrs     Surf.Area= 280 sf   Storage= 4 cf

Plug-Flow detention time= 0.4 min calculated for 1,721 cf (100% of inflow)
Center-of-Mass det. time= 0.4 min ( 651.6 - 651.2 )

Volume       Invert    Avail.Storage   Storage Description
   #1        21.12'          511 cf    Custom Stage Data (Prismatic)Listed below (Recalc)

 Elevation        Surf.Area   Voids         Inc.Store        Cum.Store
    (feet)          (sq-ft)     (%)      (cubic-feet)     (cubic-feet)
     21.12              280     0.0                 0                0
     22.62              280    33.0               139              139
     23.95              280   100.0               372              511

Device   Routing         Invert   Outlet Devices
   #1    Primary         21.12'   17.500 in/hr Exfiltration over Surface area

Primary OutFlow  Max=0.11 cfs @ 7.80 hrs   HW=21.15'    (Free Discharge)
└─1=Exfiltration   (Exfiltration Controls 0.11 cfs)
```

Pond 1P: Outdoor Planter (custom)



Summary for Pond 4P: Rock Infiltration

Inflow Area = 8,436 sf,100.00% Impervious, Inflow Depth = 6.26" for 25yr event
Inflow = 0.30 cfs @ 7.89 hrs, Volume= 4,402 cf
Outflow = 0.22 cfs @ 8.11 hrs, Volume= 4,402 cf, Atten= 25%, Lag= 12.9 min
Primary = 0.22 cfs @ 8.11 hrs, Volume= 4,402 cf

Routing by Stor-Ind method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Peak Elev= 18.34' @ 8.11 hrs Surf.Area= 500 sf Storage= 116 cf

Plug-Flow detention time= 1.2 min calculated for 4,394 cf (100% of inflow)
Center-of-Mass det. time= 1.2 min (652.4 - 651.2)

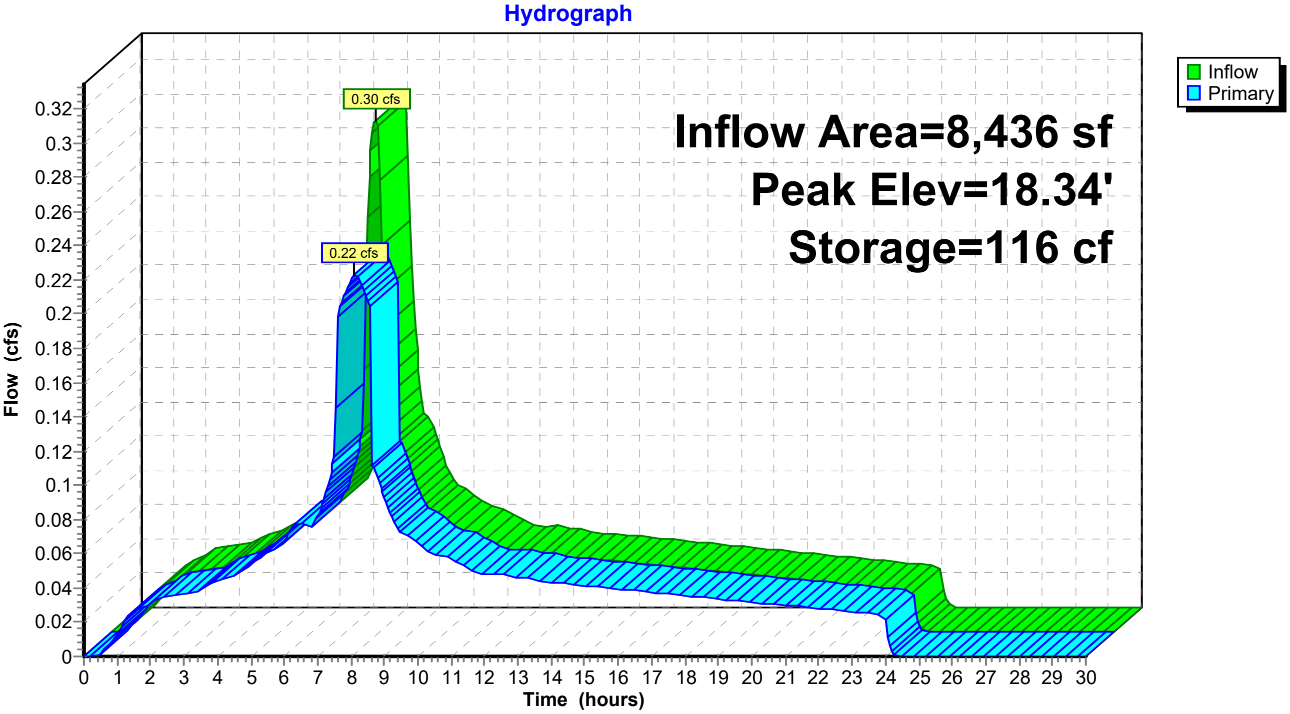
Volume	Invert	Avail.Storage	Storage Description
#1	17.70'	540 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
17.70	500	0.0	0	0	500
20.70	500	36.0	540	540	738

Device	Routing	Invert	Outlet Devices
#1	Primary	17.70'	**17.500 in/hr Exfiltration over Wetted area**

Primary OutFlow Max=0.22 cfs @ 8.11 hrs HW=18.34' (Free Discharge)
↑—**1=Exfiltration** (Exfiltration Controls 0.22 cfs)

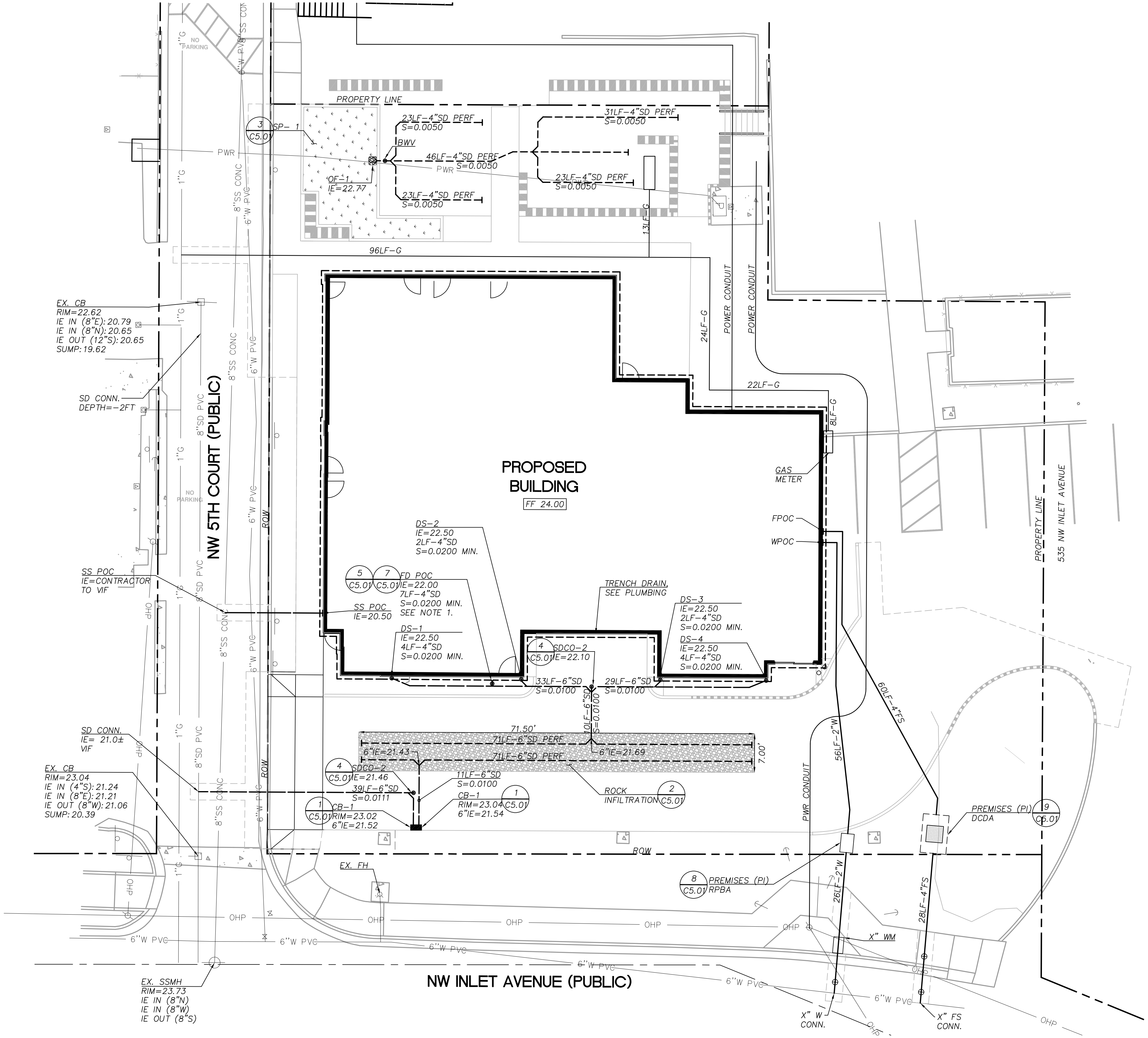
Pond 4P: Rock Infiltration



Appendix E

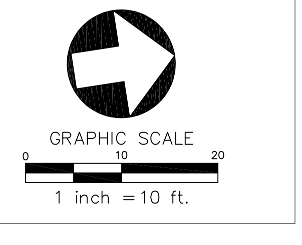
Utility Plan
Grading Plan

SHEET SIZE : 22" x 34"



UTILITY PLAN

SCALE: 1"=10'



SHEET LEGEND

ITEM	DESCRIPTION	REFERENCE
SD	STORM DRAIN	
SS	SANITARY DRAIN	
W	DOMESTIC WATER	
FS	FIRE WATER	
PWR	POWER	
G	GAS	
■	CATCH BASIN	1 C5.01
●	CLEANOUT	4 C5.01
●	BWV	7 C5.01
▦	GRATE	2 C5.01

SHEET NOTES

1. INSTALL 4" PERFORATED FOUNDATION DRAINAGE PIPE AT BASE OF FOOTING AROUND PERIMETER OF BUILDINGS PER DETAIL 5/C5.01. CONNECT PIPE TO SOLID PIPE WITH CLEANCHECK BACKFLOW PREVENTOR PER DETAIL 7/C5.01

OPEN CONCEPT ARCHITECTURE

201 NW 21ST STREET
PORTLAND, OREGON 97209

REGISTERED PROFESSIONAL ENGINEER
87835PE
OREGON
SEPT 12, 2017
KRISTIAN ANN MCCOMBS
EXPIRES 12-31-2022

LINCOLN SANDS CENTER BUILDING

515 NW Inlet Ave, Lincoln City, OR 97367

LINCOLN ASSET MANAGEMENT

UTILITY PLAN

KEY PLAN

REVISION LIST #

#	DATE	ISSUE

DATE : 11-04-2022
DRAWN : CM, SY
CHECKED : KM

PROJECT #: 20-055

C4.00

CONSTRUCTION DOCUMENTS

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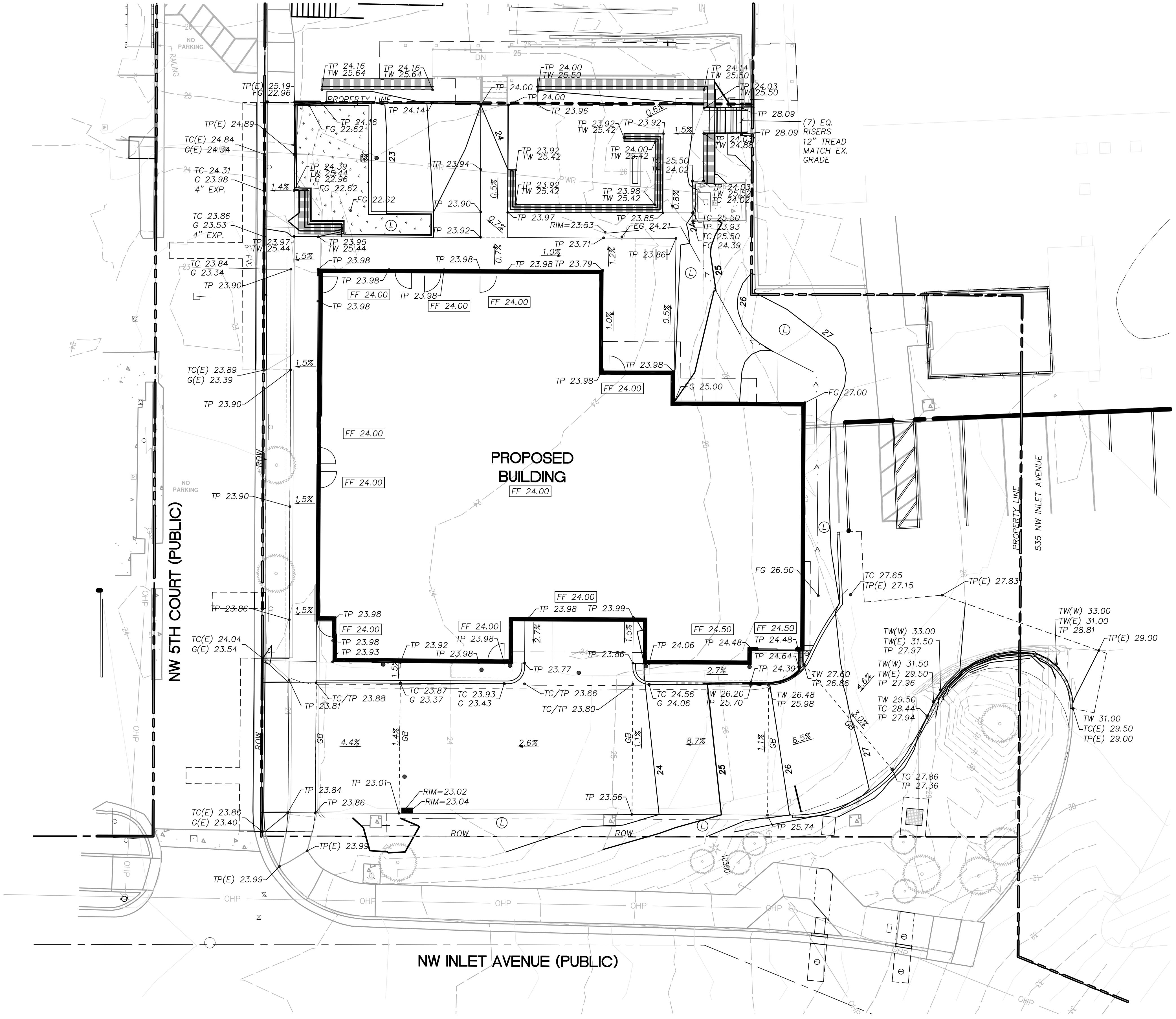
SHEET SIZE : 22" x 34"



GRADING PLAN

SCALE: 1"=10'

GRAPHIC SCALE

0 10 20

1 inch = 10 ft.

SHEET LEGEND

ITEM	DESCRIPTION	REFERENCE
— — — 25 — — —	EXISTING CONTOUR	
25	PROPOSED CONTOUR	
- - - - - - - -	GRADE BREAK (GB)	
— — — — — —	SAWCUT	
—— ·<· —— ·<· ——	FLOW LINE	
Ⓛ	LANDSCAPE	
TC XXX.XX TP XXX.XX TS XXX.XX TW XXX.XX FF XX.XX FG XXX.XX EG XXX.XX X.X% CT	GRADE AT TOP OF CURB GRADE AT TOP OF PAVEMENT GRADE AT TOP OF STEP GRADE AT TOP OF WALL FINISH FLOOR ELEVATION FINISHED GRADE EXISTING GRADE SLOPE ARROW CURB TAPER	

SHEET NOTES

1. 24-HOUR EMERGENCY CONTACT: XXXX

OPEN CONCEPT ARCHITECTURE

201 NW 21ST STREET
PORTLAND, OREGON 97209

REGISTERED PROFESSIONAL ENGINEER
87835PE
OREGON
SEPT 12, 2017
KRISTIAN ANN MCCOMBS

EXPIRES 12-31-2022

LINCOLN SANDS CENTER BUILDING

515 NW Inlet Ave, Lincoln City, OR 97367

LINCOLN ASSET MANAGEMENT

GRADING PLAN

KEY PLAN

REVISION LIST

#	DATE	ISSUE

DATE : 11-04-2022

DRAWN : CM, SY

CHECKED : KM

PROJECT #: 20-055

C3.00

CONSTRUCTION DOCUMENTS

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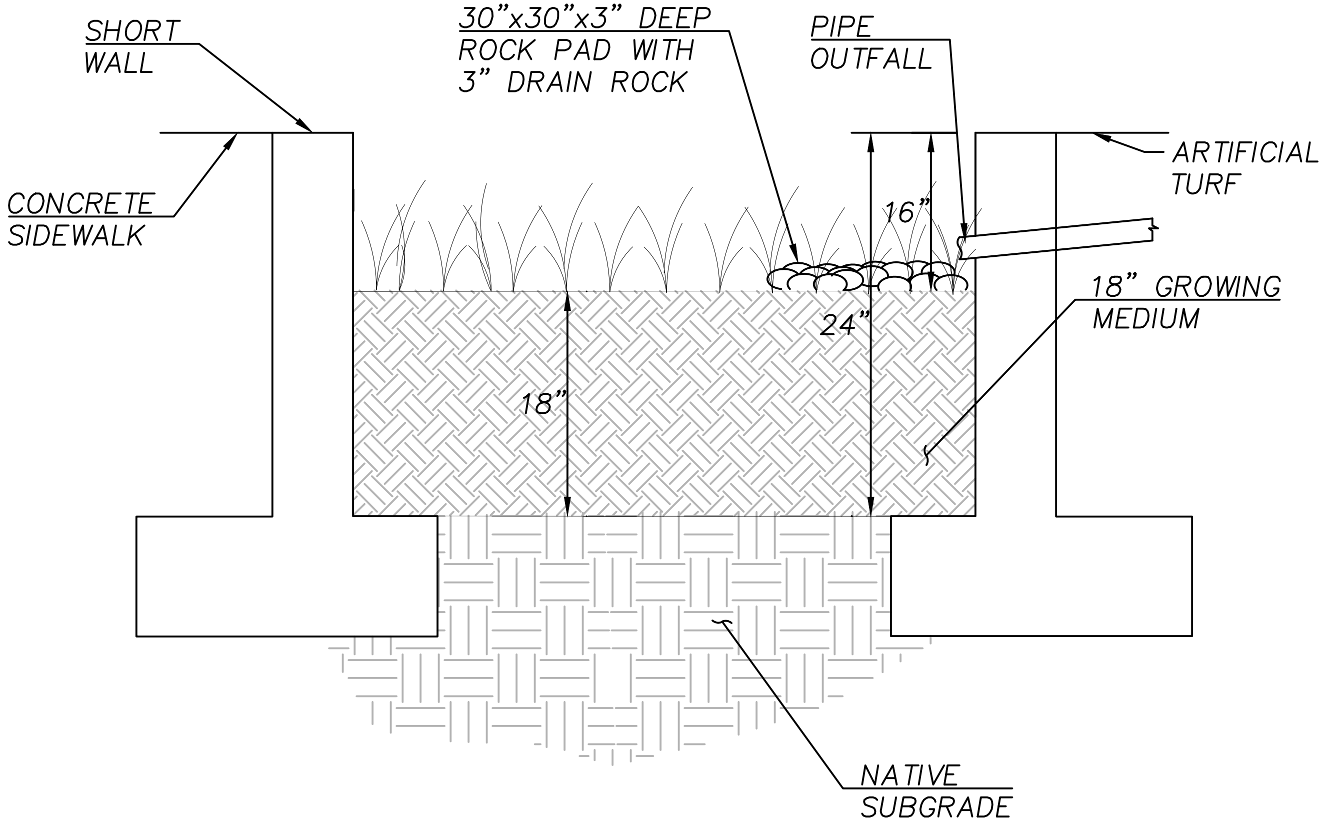
Appendix F

Detail Drawings

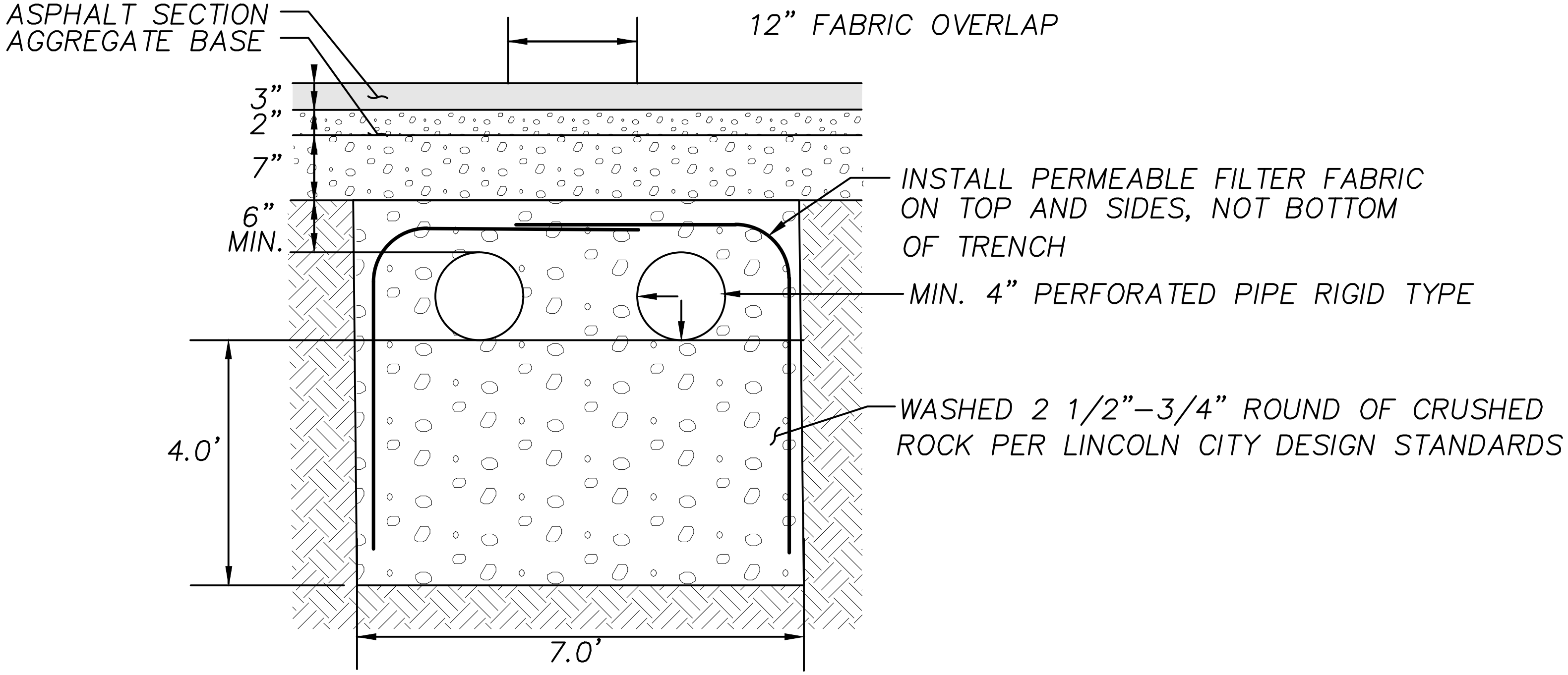


NOTES:

1. PLANTING PER LANDSCAPING PLANS.
2. GROWING MEDIUM PER SPECIFICATIONS.
3. CONCRETE PLANTER BOX SHALL BE POURED MONOLITHICALLY WITH NO COLD JOINTS TO AVOID THE REQUIREMENT FOR A LINER.

1 STORMWATER PLANTER

NTS



2 ROCK INFILTRATION

NTS

Appendix G

Geotechnical Report (Separate PD

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Lincoln Sands
1067 NW 5th Court
Lincoln City, Oregon

For
Lincoln Asset Management
August 19, 2022

Project: LincolnAM-3-01

August 19, 2022

Lincoln Asset Management
15924 Quarry Road
Lake Oswego, OR 97035

Attention: Torre Morgal

Report of Geotechnical Engineering Services
Lincoln Sands
1067 NW 5th Court
Lincoln City, Oregon
Project: LincolnAM-3-01

NV5 is pleased to present this report of geotechnical engineering services for the proposed new Lincoln Sands hotel in Lincoln City, Oregon. The site is located at the northwest corner of NW 5th Court and NW Inlet Avenue. Our services have been provided in general accordance with our proposal dated June 23, 2022.

We appreciate the opportunity to be of service to you. Please contact us if you have questions regarding this report.

Sincerely,

NV5

Shawn M. Dimke, P.E., G.E.
Principal Engineer

cc: Jeremy Cogdill, Open Concept Architecture

ZMR:SMD:kt
Attachments
One copy submitted
Document ID: LincolnAM-3-01-081922-geor.docx

EXECUTIVE SUMMARY

We understand proposed development plans include construction of a new five-story hotel building and associated access road, utilities, landscaping, and patio. The following provides a summary of pertinent geotechnical considerations. The report should be referenced for a thorough description of the subsurface conditions and geotechnical recommendations.

- The site is currently a parking lot for the existing hotel. Existing pavement, curbs, and any other structures will need to be demolished from the development area and backfilled, as necessary, with structural fill.

- The on-site soil is suitable for support of the proposed structure. We recommend compacting the surface of foundation and building subgrades to 95 percent of the maximum dry density, as determined by ASTM D1557, or to a firm and dense condition. Sand subgrades are easily disturbed when dry. A thin layer of crushed rock can be placed and compacted until “well keyed” over the sand to help prevent disturbance. Any disturbed soil should be removed or moisture conditioned and recompacted prior to pouring foundations.

- The native sand is prone to raveling, sloughing, and caving. Trench cuts and footing excavations will likely need to be laid back, shored, or formed to avoid sloughing and caving.

- Groundwater was encountered in boring B-1 at a depth of 15.2 feet BGS and a static water table of approximately 10.9 feet BGS was estimated using pore pressure dissipation testing data from CPT-1 at the time of our explorations on July 22, 2022. Increased caving, sloughing, and “running sand” are possible for excavations that extend below the depth to groundwater. If excessive caving, sloughing, or “running sand” is encountered, the excavation should be shored or flattened for stability and external dewatering may be required. In addition to safety considerations, caving, sloughing, “running sand,” or other loss of ground will increase backfill volumes and can result in damage to adjacent structures or utilities.

- The tested on-site infiltration rate indicates the native sand above the groundwater is suitable for infiltrating on-site stormwater. Based on the observed and measured groundwater depths, we recommend limiting infiltration facilities to the upper 6 feet BGS. We recommend that confirmation infiltration testing be completed at the time of construction to verify the design infiltration rates. Any infiltration facility should include an overflow that is connected to a suitable discharge point such as the public storm system or an acceptable overland flow away from buildings and slopes.

- We estimate up to 1.5 inches of liquefaction-induced settlement is possible at the ground surface as a result of a design-level seismic event and differential settlement will be up to one-half of the total settlement over a distance of 50 feet. Lateral spreading is not considered a hazard at the site.

TABLE OF CONTENTS

TABLE OF CONTENTS

APPENDICES (continued)

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	Asphalt Concrete Pavement
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CPT	cone penetration test
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
MCE	maximum considered earthquake
MSL	mean sea level
OSHA	Occupational Safety and Health Administration
OSSC	2021 Oregon Standard Specifications for Construction
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
psf	pounds per square foot
SLIDO	Statewide Landslide Information Database for Oregon
SPT	standard penetration test

1.0 INTRODUCTION

NV5 is pleased to submit this report of geotechnical engineering services for the proposed new Lincoln Sands hotel in Lincoln City, Oregon. The site is located at the northwest corner of NW 5th Court and NW Inlet Avenue. Figure 1 shows the site relative to existing physical features and streets. The site boundaries and exploration locations are shown on Figure 2.

We understand the proposed development will consist of a new five-story building and associated access road, utilities, landscaping, and patio. Foundation loads were unknown at the time of this report; however, discussions with the design team indicate column loads will likely be less than 100 kips and perimeter footing loads for walls will be less than 6 kips per lineal foot. We expect cuts and fills will be less than a few feet each.

Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for the proposed development. The specific scope of our services is summarized as follows:

- Reviewed geotechnical and geologic information provided for the site and information from our in-house project files for projects in the site vicinity.
- Called the one-call utility notification center and subcontracted a private subcontractor to locate subsurface utilities before beginning our subsurface exploration program.
- Explored subsurface soil and groundwater conditions for the proposed development by conducting the following explorations and testing:
  - Drilled two borings to depths of 20 and 66.5 feet BGS, using hollow-stem auger and mud rotary drilling techniques, respectively.
  - Performed one infiltration test at a depth of 5 feet BGS.
  - Advanced one CPT probe to a depth of 26.7 feet BGS. Shear wave velocity testing was performed at 2-meter intervals, and one pore water pressure dissipation test was performed.
- Maintained a detailed log of the borings and classified the material encountered in the borings in general accordance with ASTM D2488.
- Conducted a laboratory testing program consisting of the following:
  - Twenty-one moisture content determinations in general accordance with ASTM D2216
  - Nine particle-size analyses in general accordance with ASTM C117 or ASTM D1140
- Provided recommendations for site preparation, grading and drainage, stripping depths, fill type for imported material, compaction criteria, trench excavation and backfill, the use of on-site soil, and wet/dry weather earthwork.
- Provided recommendations for the preferred foundation type, including allowable capacity, settlement estimates, and lateral resistance or response parameters.
- Recommended design criteria for retaining walls, including lateral earth pressures, backfill, compaction, and drainage.
- Provided recommendations for preparation of floor slab subgrade.

- Provided recommendations for managing identified groundwater conditions that may affect the performance of structures or pavement.
- Provided recommendations for the AC pavement for access roads and parking areas, including subbase, base course, and AC paving thickness.
- Provided recommendations for ASCE 7-16 seismic coefficients and evaluated the risk of liquefaction and lateral spreading at the site.
- Prepared this geotechnical engineering report summarizing the results of our geotechnical evaluation.

3.0 BACKGROUND

We previously prepared a geologic bluff erosion evaluation report dated August 3, 2020 (GeoDesign, Inc., 2020). Our report indicates the planned new pool and deck adjacent to the existing building will not adversely affect the geologic conditions of the site and will have sufficient offset for erosion protection.

4.0 SITE CONDITIONS

4.1 GEOLOGIC SETTING

Lincoln City is located on the west flank of the Coast Range geo-anticline, a complex structural high with a predominant northerly trend but containing strong northeast-trending structural elements. Bedrock in the vicinity is composed of a wide variety of igneous and sedimentary rocks. Siletz River Volcanics, the oldest rocks in the area, are considered the core of the Coast Range (Early to Middle Eocene). A wide variety of younger sedimentary and igneous rocks flank the west side of the coast range, generally dipping to the west (Middle Eocene to Miocene). Faulting is extensive in these units and indicates a county-wide complex of northwest- and northeast-trending normal faults (Schlicker et al., 1973).

Surficial units at the site are mapped as Quaternary marine terrace deposits that represent uplifted, elevated former beaches. These deposits are indicative of much of Lincoln City shorelines between rocky headlands and drainages, with maximum thicknesses of up to 75 feet. Terrace deposits dominantly consist of beach sand of varying grain size, with localized concentrations of silt, clay, and organics and lenses of gravel (Schlicker et al., 1973).

A review of SLIDO-4.2 indicates that the site is not underlain by any known landslides (Burns and Watzig, 2014). The closest mapped landslide from SLIDO-4.2 is mapped approximately 0.7 mile northeast of the site.

4.2 SURFACE CONDITIONS

The site is bound by NW 5th Court to the south; NW Inlet Avenue to the east; the existing Lincoln Sands hotel building and parking lot to the north; and a two-story, residential-style building and the Oregon coast beach to the west. The site has historically been used as a parking lot for the hotel. At the time of our explorations, some of the existing AC and curbs had been removed and the area was being used as a staging area for ongoing construction projects at the hotel.

The site gently is relatively flat and slopes down from northwest to southeast and elevations on site range from 26 to 23 feet MSL.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

We drilled two borings (B-1 and B-2) to depths of 20 and 66.5 feet BGS, respectively, and advanced one CPT probe (CPT-1) to a depth of approximately 26.7 feet BGS. We conducted infiltration testing in boring B-1 at a depth of 5 feet BGS. The approximate exploration locations are shown on Figure 2. A description of the boring explorations and laboratory testing program, the boring logs, and results of laboratory testing are presented in Appendix A. The CPT results are presented in Appendix B.

4.3.2 AC

We encountered 2 inches of AC overlying 8 inches of aggregate base at our boring locations.

4.3.3 Native Sand

Below the ground surface, native soil consists of sand with varying proportions of silt. The relative density of the sand generally increases with depth, from loose near the surface to very dense at 30 feet BGS. Laboratory testing indicates that the moisture content of the native soil ranged between 4 and 41 percent and the fines content ranged between 1 and 26 percent at the time of our explorations.

4.3.4 Gravel

Medium dense to dense gravel with sand and varying proportions of silt was encountered within the sand at depths of 14.5 and 15 feet BGS. The gravel layers encountered are 2.5 to 4 feet thick. Laboratory testing indicates that the moisture content of the gravel ranged between 14 and 16 percent and the fines content ranged between 4 and 5 percent at the time of our explorations.

4.3.5 Groundwater

Groundwater was measured in boring B-1 at a depth of 15.2 feet BGS. In CPT-1, pore water pressure dissipation measured a static groundwater level of approximately 10.9 feet BGS at the time of our exploration. The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study. Based on the depths encountered, we do not anticipate groundwater to significantly impact design and construction of the proposed development.

4.4 INFILTRATION TESTING

Infiltration testing was completed to assist in the evaluation of potential stormwater infiltration facilities for the development. We conducted infiltration testing in boring B-1 at a depth of 5 feet BGS. Infiltration testing was performed using the encased falling head method using a 6-inch-inside diameter casing and approximately 18 to 24 inches of water head.

A representative soil sample was collected below the infiltration test depth to determine the percent fines content. Table 1 summarizes the unfactored infiltration test results and the fines content at the depth of the infiltration test.

Table 1. Unfactored Infiltration Rates

Location	Depth (feet BGS)	Material	Infiltration Rate (inches/hour)	Fines Content[1] (percent)
B-1	5	SAND (SP), trace silt	35	1

1. Fines content: material passing the U.S. Standard No. 200 sieve

4.5 SEISMIC HAZARDS

4.5.1 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Low plasticity, sandy silt may be moderately susceptible to liquefaction under relatively high levels of ground shaking.

We performed a liquefaction analysis using the results of the CPT and SPT blow counts from B-2, which was drilled using mud rotary methods. We assumed a design high groundwater elevation of 10 feet BGS. Our analysis indicates the medium dense sand below the groundwater depth down to the dense gravel layer encountered in our explorations is potentially liquifiable from a design-level seismic event. We estimate up to 1.5 inches of liquefaction-induced settlement at the ground surface as a result of a design-level seismic event. We estimate differential settlement will be up to one-half of the total settlement over a distance of 50 feet. The estimated settlement is typically within allowable design tolerances for structures. If the estimated settlement exceeds allowable tolerances, ground improvement can be conducted to mitigate the liquefaction potential. NV5 can be contacted to provide a discussion on potential ground improvement methods if requested.

4.5.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement.

Considering the lack of a steep face below the groundwater depth and limited zone of potentially liquefiable soil, lateral spreading is not considered a risk at the site.

5.0 DESIGN RECOMMENDATIONS

5.1 SEISMIC DESIGN CRITERIA

Seismic design is prescribed by ASCE 7-16. Table 2 presents the design parameters prescribed by ASCE 7-16 for the site. Due to the presence of potentially liquefiable soil, the Site Class is F; however, the design parameters for Site Class D, provided below, can be used per ASCE 7-16, provided the fundamental period of structure is 0.5 second or less.

Table 2. Seismic Design Parameters

Parameter	Short Period (T_s)	1 Second Period (T_1)
MCE Spectral Acceleration, S	S_s = 1.334 g	S_1 = 0.691 g
Site Class	F*	
Site Coefficient, F	F_a = 1.0	F_v = 1.7
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.334 g	S_{M1} = 1.175 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.889 g	S_{D1} = 0.783 g

* The above parameters provided for Site Class D can be used, provided structure has a fundamental period of 0.5 second or less per ASCE 7-16 Section 20.3.1 and the seismic response coefficient (C_s) is determined according to the exception in ASCE 7-16 Section 11.4.8 or else a site-specific response analysis will be required.

5.2 FOUNDATION SUPPORT

5.2.1 General

Based on the results of our explorations, laboratory testing, and analysis, it is our opinion that the site soil is capable of supporting the proposed structure on the native sand with surface subgrade compacted to 95 percent of the maximum dry density, as determined by ASTM D1557, or to a dense condition. As an alternative to compacting the sand subgrade, the upper 12 inches can be removed and replaced with crushed rock compacted as recommended for structural fill. Continuous wall and isolated spread footings should be at least 12 and 18 inches wide, respectively. The bottom of exterior column or continuous footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab.

5.2.2 Bearing Capacity

Column and continuous footings established on compacted native soil or structural fill over undisturbed native soil and prepared as recommended should be sized based on an allowable bearing pressure of 3,000 psf. The weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and can be increased by one-half for short-term loads such as those resulting from wind or seismic forces.

5.2.3 Settlement

Based on our analysis and experience with similar soil, total post-construction consolidation-induced settlement under static conditions should be less than 1 inch, with differential settlement of less than ½ inch between footings bearing on similar soil types.

5.2.4 Resistance to Sliding

Lateral loads on foundations can be resisted by passive earth pressure on the sides of the structure and by friction on the base of the foundation. Our analysis indicates that the available passive earth pressure for footings confined by on-site soil and structural fill is 350 pcf, modeled as an equivalent fluid pressure. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive equivalent fluid pressure of 275 pcf. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on passive resistance, a minimum of 10 feet of horizontal clearance must exist between the face of the footings and any adjacent downslopes.

For foundations in contact with native soil, a coefficient of friction equal to 0.35 should be used when calculating resistance to sliding. This value can be increased to 0.40 for foundations established on at least 4 inches of imported granular soil.

5.3 FLOOR SLABS

Satisfactory subgrade support for building floor slabs supporting up to 100 psf areal loading can be achieved on the existing native soil or on structural fill. All sand subgrade should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557. A minimum 6-inch-thick layer of aggregate base should be placed and compacted over the prepared soil subgrade. Imported granular material placed beneath building floor slabs should meet the requirements for aggregate base rock as described in the “Structural Fill” and “Fill Placement and Compaction” sections. A subgrade reaction modulus of 200 pci can be used to design floor slabs that bear on the native soil.

Vapor barriers are often required by flooring manufacturers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier (if needed) should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

5.4 RETAINING STRUCTURES

5.4.1 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) the walls consist of conventional, cantilevered retaining walls; (2) the walls are less than 10 feet in height; (3) the backfill is drained and consists of imported granular material; and (4) the backfill has a slope flatter than 4H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

5.4.2 Wall Design Parameters

For unrestrained retaining walls, an active equivalent fluid pressure of 35 pcf should be used for design. Where retaining walls are restrained from rotation (such as basement walls), an at-rest equivalent fluid pressure of 55 pcf should be used for design. A superimposed seismic lateral force should be calculated based on a dynamic force of $11.5H^2$ pounds per lineal foot of wall, where H is the height of the wall in feet, and applied as a distributed load with the centroid located at a distance of 0.6H from the base of the wall.

If surcharges (e.g., retained slopes, structure foundations, vehicles, steep slopes, terraced walls, etc.) are located within a horizontal distance from the back of a wall equal to the height of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads. The base of the wall footing excavations should extend a minimum of 12 inches below the lowest adjacent grade. The wall footings should be designed in accordance with the “Foundation Support” section.

5.4.3 Wall Drainage and Backfill

The above design parameters have been provided assuming back-of-wall drains will be installed to prevent buildup of hydrostatic pressures behind all walls. If a drainage system is not installed, our office should be contacted for revised design forces.

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular wall backfill meeting the requirements described in the “Structural Fill” section. Alternatively, the native soil can be used as backfill material, provided a minimum 1-foot-wide column of angular drain rock wrapped in a geotextile is placed against the wall and the native soil can be adequately moisture conditioned for compaction. The rock column should extend from the perforated drainpipe to within approximately 1 foot of the ground surface. The angular drain rock should meet the requirements provided in the “Structural Fill” section. All wall backfill should be placed and compacted as recommended for select granular wall backfill in the “Structural Fill” and “Fill Placement and Compaction” sections.

Perforated collector pipes should be placed at the base of the granular backfill behind the walls. The pipe should be embedded in a minimum 1-foot-wide zone of angular drain rock. The drain rock should meet specifications provided in the “Structural Fill” section. The drain rock should be wrapped in a drainage geotextile fabric meeting the requirements in the “Geotextile Fabric” section. The collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the drainage system of the wall.

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed at least four weeks after backfilling of the wall, unless survey data indicates that settlement is complete prior to that time.

5.5 PAVEMENT

The proposed project includes the construction of new AC parking lots and access roads. We anticipate the pavement traffic will generally consist of light automobiles and pickup trucks. We anticipate traffic on light duty access roads and in drive isles will also include up to five delivery- or garbage-type trucks per day. We anticipate the pavement will not be used regularly by large trucks or construction-type equipment. Our recommended pavement sections are presented in Table 3.

Table 3. Recommended Standard Pavement Sections

Pavement Use	AC Thickness[1] (inches)	Aggregate Base Thickness[1,] (inches)
Parking lot stalls	2.5	7
Light duty	3	9

1. All thicknesses are intended to be the minimum acceptable values. Additional thickness will be necessary if construction traffic is allowed on the pavement.

All of the recommended pavement sections with subgrades prepared as recommended are suitable to support an occasional 75,000-pound fire truck.

The AC and aggregate base should meet the requirements outlined in the “Materials” section. The pavement sections recommended above are designed to support post-construction traffic. If construction traffic is allowed on new pavement, allowance for the additional loading and wear should be included in the design section.

5.6 DRAINAGE CONSIDERATIONS

5.6.1 Temporary

During mass grading at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the site, the contractor should keep all pads and subgrade free of ponding water.

5.6.2 Surface

Where possible, the finished ground surface around the building should be sloped away from the structure at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries the collected water to an appropriate stormwater system. Trapped planter areas should not be created adjacent to the building without providing means for positive drainage (e.g., swales or catch basins).

5.6.3 Subsurface

Assuming the site grades around the building will be sloped as discussed previously, it is our opinion that perimeter footing drains will not be required around the proposed building. However, the use of these drains should be considered in areas where landscaping planters are placed proximate to the foundations or where surface grades cannot be completed as outlined above.

If installed, the foundation drains should be constructed at a minimum slope of approximately ½ percent and pumped or drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. The foundation drains should consist of 4-inch-diameter, perforated drainpipe embedded in a minimum 2-foot-wide zone of crushed drain rock that extends to the ground surface. The invert elevation of the drainpipe should be installed at least 18 inches below the elevation of the floor slab.

The drain rock and geotextile should meet the requirements specified in the "Materials" section. The drain rock and geotextile should extend up the side of embedded walls to within a foot of the ground surface, geotextile wrapped over the top of the drain rock, as recommended in the "Retaining Structures" section.

5.7 INFILTRATION SYSTEMS

The results of our infiltration testing indicate that the on-site soil should be suitable for infiltrating stormwater collected on site. We observed or measured groundwater at depths of approximately 10.9 to 15.2 feet BGS at the exploration locations. Based on the observed and measured groundwater depths, we recommend limiting infiltration facilities to the upper 6 feet BGS. The infiltration rate shown in Table 1 is a short-term field rate and factors of safety have not been applied for the type of infiltration system being considered. Appropriate correction factors should be applied by the project civil engineer to determine long-term infiltration parameters. From a geotechnical perspective, we recommend a minimum factor of safety of 3 be applied to the field infiltration value presented in Table 1 to account for soil variability with depth. The infiltration system design engineer should determine and apply appropriate remaining correction factor values or factors of safety to account for the degree of in-system filtration, system maintenance, vegetation, potential for clogging, etc. We recommend the installation of infiltration facilities be observed by a qualified geotechnical engineer or representative under their supervision to evaluate if soil conditions are consistent with subsurface conditions encountered during our explorations. We also recommend confirmation testing at the infiltration facilities.

The infiltration flow rate of a disposal system will diminish over time as suspended solids and precipitates in the stormwater slowly clog the void spaces between the soil particles. Eventually the system may fail and need to be replaced. We recommend that the system include an overflow that is connected to a suitable discharge point such as the public storm system or an acceptable overland flow away from buildings and slopes. Finally, stormwater infiltration systems will cause localized high groundwater levels; therefore, they should not be located near basement walls, retaining walls, or other embedded structures, unless these are specifically designed to account for the resulting hydrostatic pressure.

5.8 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. New constructed fill slopes should be over-built by at least 12 inches and then trimmed back to the required slope to maintain a firm face.

Access roads and pavement should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

6.0 CONSTRUCTION

6.1 SITE PREPARATION

6.1.1 Demolition

We anticipate existing structures such as pavement and curbs will be demolished as part of site preparation activities. Demolition includes complete removal of existing site developments within 5 feet of areas to receive new pavement, buildings, retaining walls, or engineered fills. Underground vaults, tanks, wells, and other subsurface structures should be removed in areas of new improvements. Utility lines should be completely removed or grouted full if left in place. Existing basements, crawl spaces, or other voids resulting from removal of existing improvements should be backfilled with compacted structural fill, as discussed in the "Structural Fill" section. The bottom of such excavations should be excavated to expose a firm subgrade before filling and their sides sloped at a minimum of 1.5H:1V to allow for more uniform compaction at the edges of the excavations.

6.1.2 Grubbing and Stripping

Trees and shrubs should be removed from development areas. In addition, root balls should be grubbed out to the depth of the roots, which could exceed 3 feet BGS. Depending on the methods used to remove root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

Existing topsoil should be stripped and removed from all fill areas. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

6.1.3 Foundation Subgrade Observation and Protection

Neat-cut, formed footing excavations may slough material during placement of reinforcement steel prior to concrete placement. This material should be removed prior to pouring footings. Excessive sloughing will require that footing excavations be laid back, formed, or shored.

All footing subgrades should be evaluated by the project geotechnical engineer or their representative after subgrade compaction to confirm suitable bearing conditions. Observations should also confirm that all loose or soft material, organic material, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate deleterious material.

Sand subgrades are easily disturbed when dry. A thin layer of crushed rock can be placed and compacted until "well keyed" over the sand to help prevent disturbance. The contractor is responsible for the construction sequencing and methodology for footing excavation and construction. Any foundation subgrade soil that is disturbed should be removed or moisture conditioned and compacted until dense and "well keyed" prior to pouring foundations.

6.1.4 Subgrade Evaluation

A member of our geotechnical staff should observe exposed structural subgrades and foundation excavations after stripping and site cutting have been completed to determine if there are additional areas of unsuitable or unstable soil. Our representative should observe a proof roll of structural fill, pavement, and slab subgrades with a fully loaded dump truck or similar heavy, rubber tire construction equipment to identify soft, loose, or unsuitable areas. In areas not accessible to proof rolling equipment, the subgrade should be evaluated by probing. Areas identified as soft, unstable, or otherwise unsuitable should be over-excavated and replaced with compacted material recommended for structural fill. Areas that appear too wet or soft to support proof rolling or compaction equipment should be evaluated by probing and prepared in accordance with the "Construction Considerations" section.

6.2 CONSTRUCTION CONSIDERATIONS

Trafficability of the site could be difficult because of the sand that exists at the ground surface and the natural tendency of unconfined surface sand to fluff to a relatively loose state. Accordingly, the use of granular haul roads or staging areas could be necessary for support of construction traffic. However, compaction of the surface sand followed by placement of 4 to 6 inches of stabilization material should be sufficient for the staging areas, the basic building pad, and haul roads. The stabilization material should be placed in one lift over the prepared undisturbed subgrade and compacted using a smooth-drum, vibratory roller. The stabilization material should meet the requirements described in the "Structural Fill" section.

6.3 EXCAVATION

6.3.1 General

Excavations will be required to construct new foundations, utilities, pavement, and other improvements. Conventional earthmoving equipment in proper working condition should be capable of making the necessary excavations. We encountered or measured groundwater at depths greater than 10 feet BGS. If groundwater is encountered in excavations, sloughing and caving will likely occur and "running sand" is possible. Accordingly, the contractor should expect to flatten excavations or shore excavations as described below where water is encountered. In addition to safety considerations, caving and loss of ground will increase backfill volumes and can result in damage to adjacent structures or utilities.

Excavations made in sand may be prone to raveling. We recommend that any excavation made into the native sand use shoring or be sloped. Sloped excavations may be used to depths of 10 feet BGS and should have side slopes no steeper than 1.5H:1V, provided groundwater seepage does not occur. We recommend a minimum horizontal distance of 5 feet from the edge of existing improvements to the top of any temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If seepage, sloughing, or instability is observed, the slope should be flattened or shored. Shoring will be required where slopes are not possible. We can provide additional shoring recommendations if shoring will be used on this project. The contractor should be responsible for selecting the appropriate shoring system. Furthermore, we recommend that the contractor use formwork during preparation of shallow foundations.

Excavations should not be allowed to undermine adjacent improvements. If existing roads or structures are located near a proposed excavation, unsupported excavations can be maintained outside of a 1H:1V downward projection that starts 5 feet outside the base of the existing elements. Excavations that must be inside of this zone should be supported by temporary or permanent shoring designed for moment resistance for the full height of the excavation, including kick-out for the full buried depth of the retaining system.

While we have described certain approaches to performing excavations, it is the contractor's responsibility to select the excavation and dewatering methods, monitor the excavations for safety, and provide any shoring required to protect personnel and adjacent improvements.

6.3.2 Trenches and Shoring

Open excavation techniques may be used to excavate trenches, provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If excavations are left open for extended periods of time, caving of the sidewalls will occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

6.3.3 Temporary Dewatering

We anticipate that most excavations will be above the groundwater level. However, some perched water could still seep into the site excavations, especially after periods of heavy rain. We anticipate that dewatering methods consisting of pumping water from the excavation with sumps will generally be adequate. Water generated during dewatering operations should be treated, if necessary, and pumped to a suitable disposal point.

Where seepage occurs in excavations, we recommend placing at least 1 foot of stabilization material at the base of the excavations. The stabilization material should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious material.

We note that these recommendations are for guidance only. Dewatering of excavations is the sole responsibility of the contractor, as the contractor is in the best position to select the appropriate system based on their means and methods.

6.3.4 Safety

All excavations should be made in accordance with applicable OSHA requirements and regulations of the state, county, and local jurisdiction. While this report describes certain approaches to excavation and dewatering, the contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring (as required) to protect personnel and adjacent structural elements.

6.4 MATERIALS

6.4.1 Structural Fill

6.4.1.1 General

Fills should only be placed over a subgrade that has been prepared in conformance with the "Site Preparation" section. All material used as structural fill should be free of organic material or other unsuitable material. The material should meet the specifications provided in OSSC 00330 (Earthwork), depending on the application. Except as modified below, all structural fill should have a maximum particle size of 4 inches. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

6.4.1.2 On-Site Soil

The on-site material should generally be suitable for use as general structural fill, provided it is properly moisture conditioned; free of debris, organic material, and particles over 8 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material). Laboratory testing indicates that use of the native sand as structural fill will require moisture conditioning. Typically, generous amounts of water and compaction using a vibratory roller are required to achieve adequate compaction.

6.4.1.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.14 (Selected Granular Backfill) or OSSC 00330.15 (Selected Stone Backfill). The imported granular material should also be angular, should be fairly well graded between coarse and fine material, should have less than 6 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two fractured faces.

6.4.1.4 Stabilization Material

Stabilization material used in staging or haul road areas, in trenches, or for other applications should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.15 (Selected Stone Backfill). The material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious material. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

6.4.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D).

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill material that is free of organic material and material over 6 inches in diameter and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D).

6.4.1.6 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and should meet the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material). The material should be free of roots, organic material, and other unsuitable material; should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

6.4.1.7 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavement should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). The aggregate should have at least two mechanically fractured faces. In addition, the aggregate should have less than 6 percent by dry weight passing the U.S. Standard No. 200 sieve.

6.4.1.8 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular material that meets the specifications provided in OSSC 00510.12 (Granular Wall Backfill) or OSSC 00510.13 (Granular Structure Backfill). We recommend the select granular wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

The backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls.

6.4.2 Geotextile Fabric

6.4.2.1 Subgrade Geotextile

Subgrade geotextile should conform to OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

6.4.2.2 Drainage Geotextile

Drainage geotextile should conform to Type 2 material of OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

6.4.3 AC

6.4.3.1 ACP

On-site AC should be Level 2, ½-inch, dense ACP according to OSSC 00745 (Asphalt Concrete Pavement – Statistical Acceptance) and compacted to 91 percent of the maximum specific gravity of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses are 2.0 and 3.5 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better.

6.4.3.2 Cold Weather Paving Considerations

In general, AC paving is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress.

Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. In Oregon, the AC surface temperature during paving should be at least 40 degrees Fahrenheit for lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thickness between 2.0 and 2.5 inches.

If paving activities must take place during cold-weather construction as defined above, the project team should be consulted and a site meeting should be held to discuss ways to lessen low compaction risks.

6.5 FILL PLACEMENT AND COMPACTION

Fill soil should be compacted at a moisture content that is within 3 percent of optimum. The maximum allowable moisture content varies with the soil gradation and should be evaluated during construction. Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The maximum lift thickness will vary depending on the material and compaction equipment used but should generally not exceed the loose thicknesses provided in Table 4. Fill material should be compacted in accordance with the compaction criteria provided in Table 5.

Table 4. Recommended Uncompacted Lift Thickness

Compaction Equipment	Recommended Uncompacted Lift Thickness (inches)		
	Native Soil	Granular and Crushed Rock Maximum Particle Size ≤ 1½ Inches	Crushed Rock Maximum Particle Size > 1½ Inches
Hand Tools: Plate Compactor and Jumping Jack	4 to 8	4 to 8	Not recommended
Rubber Tire Equipment	6 to 8	10 to 12	6 to 8
Light Roller	8 to 10	10 to 12	8 to 10
Heavy Roller	10 to 12	12 to 18	12 to 16
Hoe Pack Equipment	12 to 16	18 to 24	18 to 24

The table above is based on our experience and is intended to serve only as a guideline. The information provided in this table should not be included in the project specifications.

Table 5. Compaction Criteria

Fill Type	Compaction Requirements in Structural Zones		
	Percent Maximum Dry Density Determined by ASTM D1557		
	0 to 2 Feet Below Subgrade (percent)	Greater Than 2 Feet Below Subgrade (percent)	Pipe Zone (percent)
Area Fill (Granular)	95	95	-----
Area Fill (Fine Grained)	92	92	-----
Aggregate Bases	95	95	-----
Trench Backfill[1,2]	95	92	90[1,2]
Retaining Wall Backfill	95[3]	92[3]	-----

1. Trench backfill above the pipe zone in non-structural areas should be compacted to 85 percent.
2. Or as recommended by the pipe manufacturer.
3. Should be reduced to 90 percent within a horizontal distance of 3 feet from the retaining wall.

6.6 EROSION CONTROL

The native soil at this site is eroded easily by wind and water; therefore, erosion control measures should be carefully planned and in place before construction begins. Measures that can be employed to reduce erosion include the use of silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads. All erosion control methods should be in accordance with local jurisdiction standards. During earthwork at the site, the contractor should be responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface.

7.0 OBSERVATION OF CONSTRUCTION

Satisfactory earthwork and foundation performance depend to a large degree on the quality of construction. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated. In addition, sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications.

8.0 LIMITATIONS

We have prepared this report for use by Lincoln Asset Management and members of the design and construction team for the proposed project. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Soil explorations indicate soil conditions only at specific locations and only to the depths penetrated. The soil explorations do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary. In addition, if design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

◆ ◆ ◆

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

NV5

Zane M. Rogers, P.E.
Project Engineer

Shawn M. Dimke, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
63114PE
OREGON
March 14, 2006
SHAWN DIMKE
EXPIRES: 12/31/23

REFERENCES

ASCE, 2016. Minimum Design Loads for Buildings and Other Structures. ASCE Standard ASCE/SEI 7-016. American Society of Civil Engineers.

Burns, W.J., and Watzig, R.J., 2014, Statewide Landslide Information Layer for Oregon, Release 3 (SLIDO-4.2), Oregon Department of Geology and Mineral Industries.

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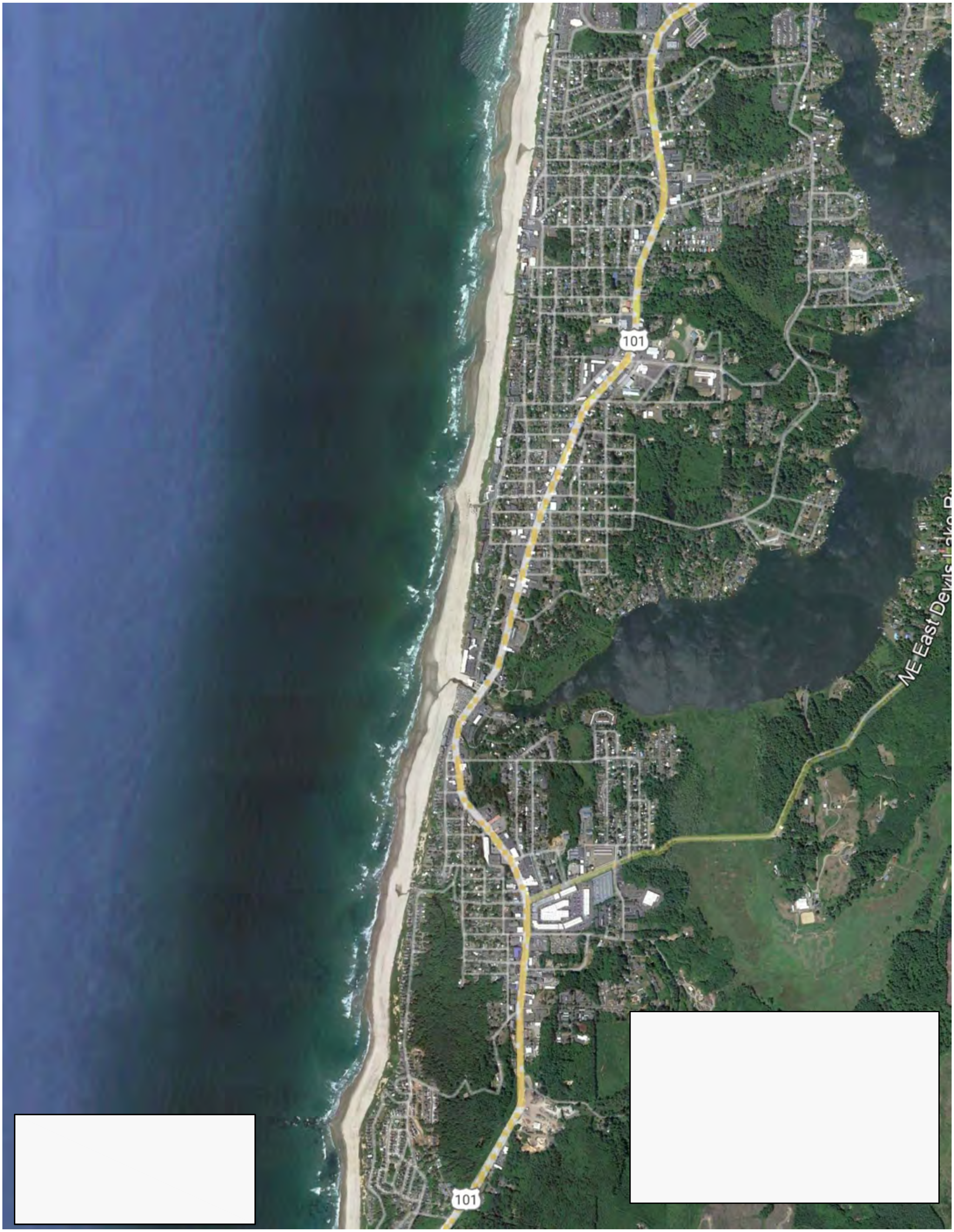
Schlicker, H.G., Deacon, R.J., Olcott, G.W., Beaulieu, J.D., 1973, Environmental Geology of Lincoln County, Oregon. Oregon Department of Geology and Mineral Industries, Bulletin 81.

FIGURES



NV5

LINCOLNAM-3-01

AUGUST 2022

VICINITY MAP

LINCOLN SANDS
LINCOLN CITY, OR

FIGURE 1

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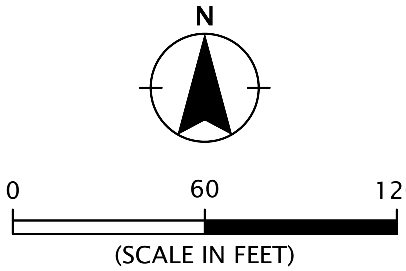
File Name: J:\E-L\LincolnAM\LincolnAM-3-01\Figures\CAD\LincolnAM-3-01-VM01.dwg | Layout: FIGURE 1

LEGEND:

— - —	SITE BOUNDARY
▨	APPROXIMATE PROPOSED BUILDING FOOTPRINT
B-1 ●	BORING
CPT-1 △	CPT



SITE PLAN BASED ON AERIAL PHOTOGRAPH DATED
JULY 24, 2019, OBTAINED FROM GOOGLE EARTH PRO.

FIGURE 2

SITE PLAN

LINCOLN SANDS
LINCOLN CITY, OR

LINCOLNAM-3-01

AUGUST 2022

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling two borings (B-1 and B-2) to depths of 20 and 66.5 feet BGS, respectively. The explorations were completed on July 22, 2022, by Western States Soil Conservation, Inc. of Hubbard, Oregon, using a truck-mounted drill rig with hollow-stem auger and mud rotary methods. The exploration logs are presented in this appendix.

The approximate exploration locations are shown on Figure 2. The locations were determined in the field by pacing or measuring from existing site features. This information should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

We collected representative samples of the various soil encountered during drilling for geotechnical laboratory testing. Samples were collected from the borings using 1½-inch-inside diameter, split-spoon SPT sampler in general accordance with ASTM D1586. The samplers were driven into the soil with a 140-pound automatic trip hammer free falling 30 inches. Each sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration logs, unless otherwise noted. Sampling methods and intervals are shown on the exploration logs.

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 85.5 percent. The calibration testing results are presented at the end of this appendix.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soil or its characteristics change, although the change could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications.

MOISTURE CONTENT

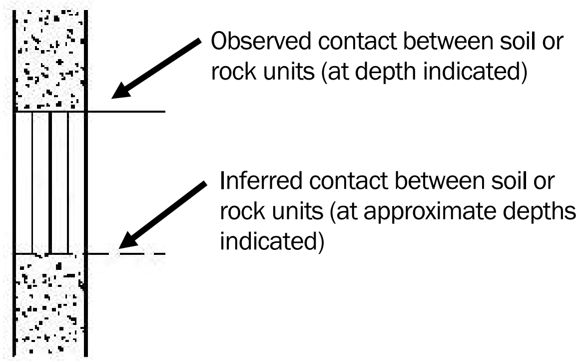
The natural moisture content of select soil samples was determined in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

Particle-size analysis was performed on select soil samples in general accordance with ASTM D1140. This test is a quantitative determination of the amount of material finer than the U.S. Standard No. 200 sieve expressed as a percentage of soil weight. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
◧	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test (SPT) with recovery
◧	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
▥	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▥	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
◩	Location of sample collected using 3-inch-outside diameter California split-spoon sampler and 140-pound hammer with recovery
☒	Location of grab sample
■	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

NV5

EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Test (SPT) Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Test (SPT) Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

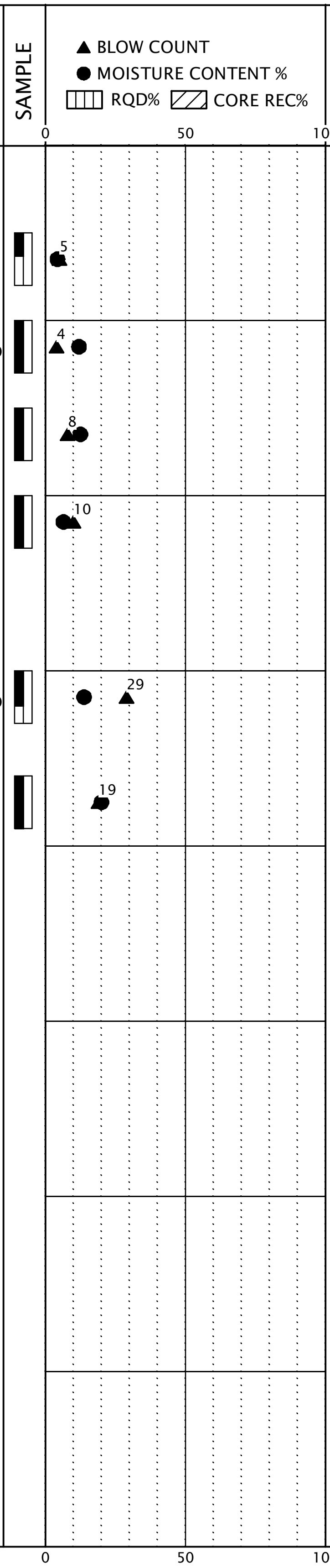
Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

NV5

SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (2.0 inches).	0.2		
	AGGREGATE BASE (8.0 inches).	0.8		
	Loose, brown-gray SAND (SP), trace silt; moist, sand is fine to medium.			
5	trace gravel; gravel is fine and subangular at 5.0 feet		P200	Infiltration test at 5.0 feet. P200 = 1%
	moist to wet at 7.5 feet			
10	loose to medium dense at 10.0 feet			
15	Medium dense, gray GRAVEL with silt and sand (GP-GM); wet, gravel is coarse and subangular, sand is fine to medium.	15.0	P200	▼ 15.2 feet, after drilling P200 = 5%
	Medium dense, gray SAND (SP), trace gravel and silt; wet, sand is fine to medium, gravel is fine and subangular.	17.5		Smooth drilling at 17.5 feet. Sand heave to 18.0 feet.
20	Exploration completed at a depth of 20.0 feet. Hammer efficiency factor is 85.5 percent.	20.0		Surface elevation was not measured at the time of exploration.
25				
30				
35				
40				



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: Z. Rogers
COMPLETED: 07/22/22

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 6 inches

NV5

LINCOLNAM-3-01

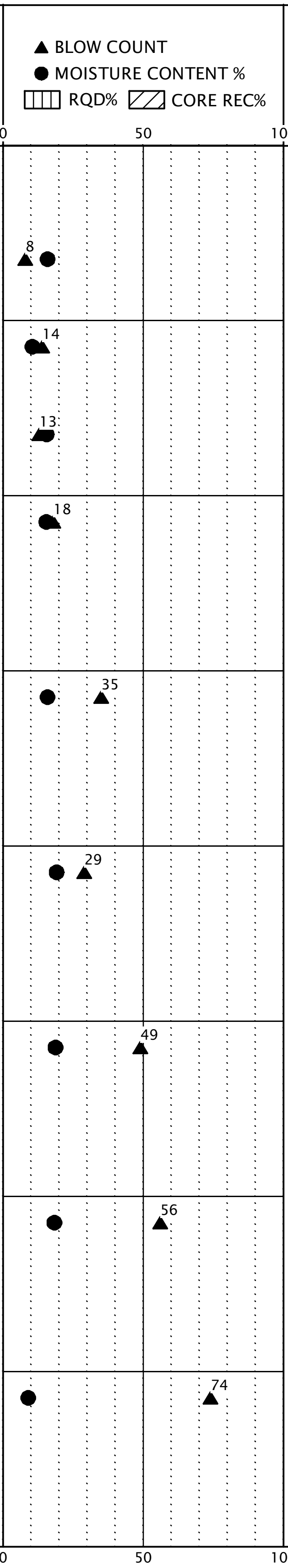
AUGUST 2022

BORING B-1

LINCOLN SANDS
LINCOLN CITY, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	▲ BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (2.0 inches).	0.2			
	AGGREGATE BASE (8.0 inches).	0.8			
	Loose, light brown SAND (SP), trace silt; moist, sand is fine to medium.			8	
5	medium dense at 5.0 feet			14	
				13	
10	minor gravel; gravel is fine and subangular at 10.0 feet			18	Switch to 3 7/8-inch tricone at 10.0 feet.
15	Dense, dark gray GRAVEL with sand (GP), trace silt; wet, gravel is fine to coarse and subangular, sand is fine to medium.	14.5	P200	35	Driller Comment: Gravel at 14.5 feet P200 = 4%
	Medium dense, gray SAND with silt (SP-SM), minor gravel; wet, sand is fine to medium, gravel is fine and subangular.	18.5			Driller Comment: Smooth drilling at 18.5 feet.
20			P200	29	P200 = 6%
25	dense, light brown-gray; sand is fine to coarse, gravel is fine to coarse and subrounded at 25.0 feet			49	
30	very dense, light brown with orange staining; gravel is fine and subrounded to subangular at 30.0 feet			56	
	Very dense, light brown, silty SAND (SM), minor gravel; wet, sand is fine to medium, gravel is fine and subrounded to subangular.	33.0			
35			P200	74	P200 = 13%
40					



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: Z. Rogers

COMPLETED: 07/22/22

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches and 3 7/8 inches

NV5

LINCOLNAM-3-01

AUGUST 2022

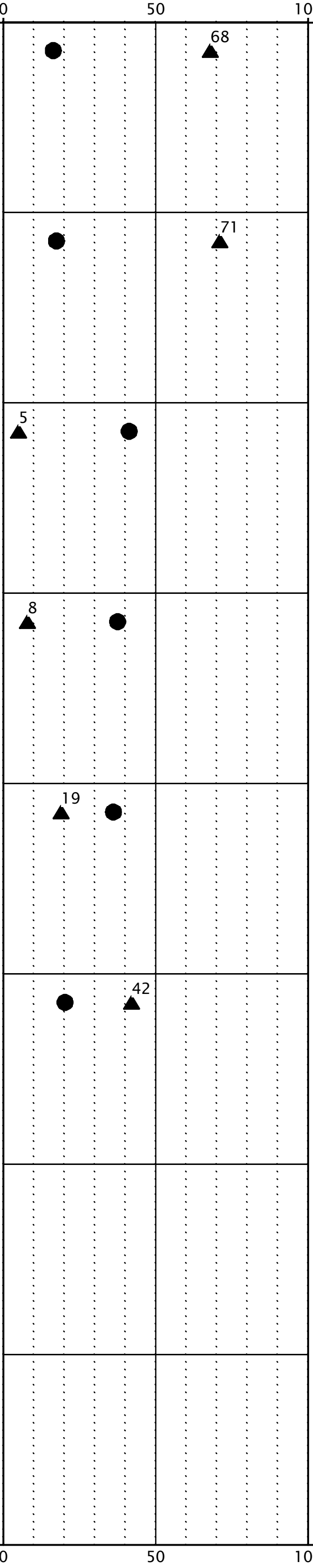
BORING B-2

LINCOLN SANDS
LINCOLN CITY, OR

FIGURE A-2

BORING LOG - NV5 - 1 PER PAGE LINCOLNAM-3-01-B1_2.GPJ GDI_NV5.GDT PRINT DATE: 8/19/22:SN:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
40	gravel is fine to coarse at 40.0 feet			
	Very dense, light brown SAND with silt (SP-SM); wet, sand is fine to medium, gravel is fine to coarse and subrounded to subangular.	43.0	P200	P200 = 8%
	Loose, dark brown, silty SAND (SM), minor gravel, trace organics (roots); moist to wet, sand is fine to medium, gravel is fine to coarse and subrounded to subangular, organics are less than 1/4 inch in diameter.	48.0	P200	Driller Comment: Soft material at approximately 48.0 feet. P200 = 26%
	Loose, brown SAND with silt (SP-SM); wet, sand is fine to medium.	53.0	P200	P200 = 11%
60	medium dense, brown-orange at 60.0 feet			
	Dense, light brown with orange stained, silty SAND (SM); wet, sand is fine to medium.	63.0	P200	P200 = 13%
	Exploration completed at a depth of 66.5 feet. Hammer efficiency factor is 85.5 percent.	66.5		Surface elevation was not measured at the time of exploration.



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: Z. Rogers

COMPLETED: 07/22/22

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches and 3 7/8 inches

NV5

LINCOLNAM-3-01

AUGUST 2022

BORING B-2
(continued)

LINCOLN SANDS
LINCOLN CITY, OR

FIGURE A-2

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	2.5		4							
B-1	5.0		12				1			
B-1	7.5		12							
B-1	10.0		6							
B-1	15.0		14				5			
B-1	18.0		20							
B-2	2.5		16							
B-2	5.0		10							
B-2	7.5		16							
B-2	10.0		15							
B-2	15.0		16				4			
B-2	20.0		19				6			
B-2	25.0		19							
B-2	30.0		18							
B-2	35.0		9				13			
B-2	40.0		16							
B-2	45.0		17				8			
B-2	50.0		41				26			
B-2	55.0		37				11			
B-2	60.0		36							
B-2	65.0		20				13			

NV5

LINCOLNAM-3-01

AUGUST 2022

SUMMARY OF LABORATORY DATA

LINCOLN SANDS
LINCOLN CITY, OR

FIGURE A-3

Summary of SPT Test Results

Project: WSSC-8-06, Test Date: 12/23/2021

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
60.00	**4-10-15**	**25**	**35**	50	14.7	52.0	308	88.1
60.00	**8-10-12**	**22**	**31**	49	13.9	51.9	299	85.4
60.00	**2-6-9**	**15**	**21**	46	15.2	51.8	304	86.8
60.00	**6-9-13**	**22**	**31**	46	13.9	52.2	286	81.7
			Overall Average Values:	48	14.4	52.0	299	85.5
			Standard Deviation:	6	1.6	2.3	33	9.5
			Overall Maximum Value:	54	15.7	70.0	316	90.4
			Overall Minimum Value:	0	1.0	41.0	2	0.6

APPENDIX B

APPENDIX B

CONE PENETROMETER TESTING

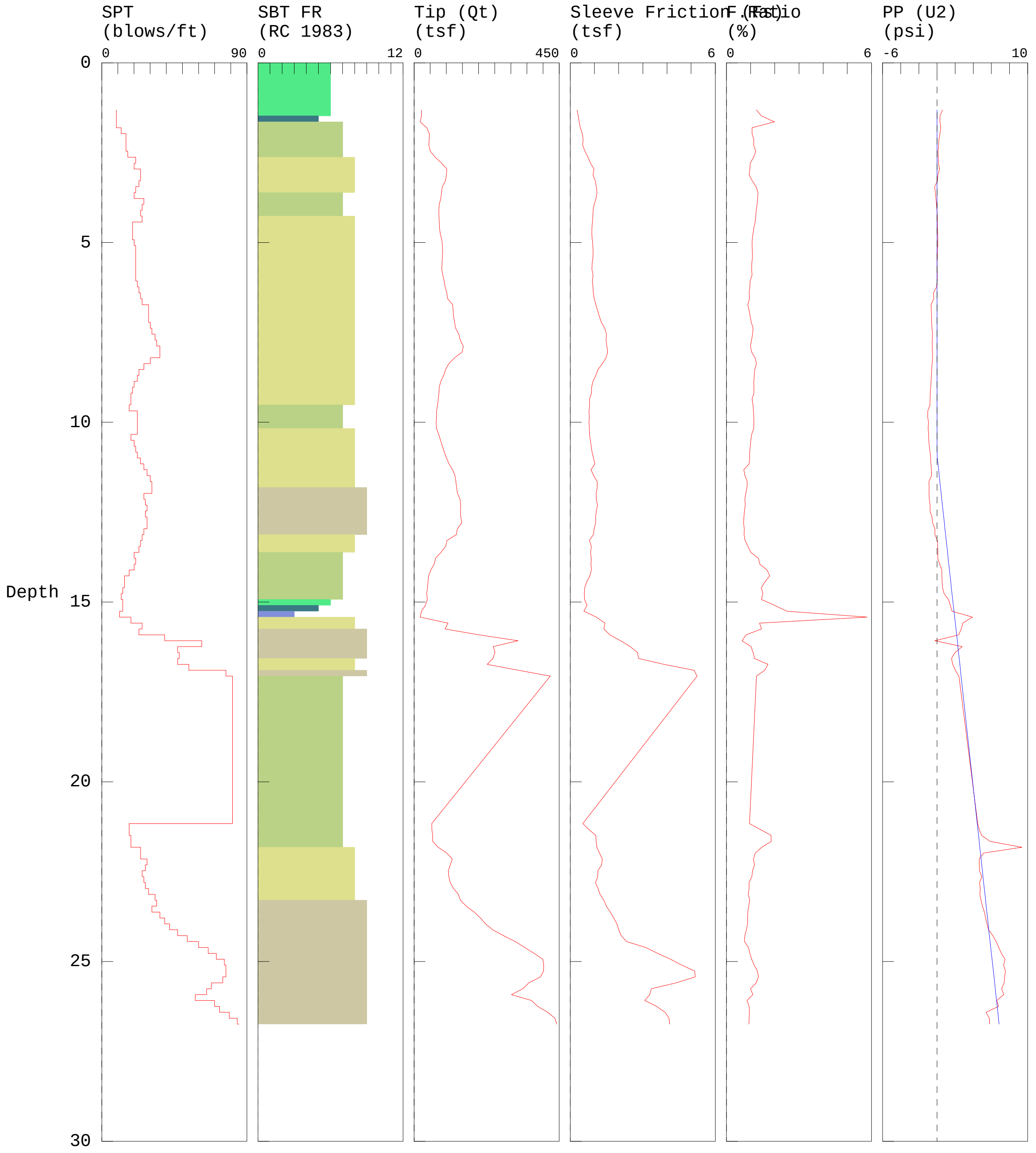
Our subsurface exploration program included advancing one CPT (CPT-1) to a depth of approximately 26.7 feet BGS. Figure 2 shows the location of the CPT. The CPT was conducted in general accordance with ASTM D5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon, on July 22, 2022. The results of the CPT are presented in this appendix.

The location of the exploration was determined in the field by pacing from existing site features. This information should be considered accurate the degree implied by the method used.

The CPT is an in-situ test that provides characterizes subsurface stratigraphy. The testing includes advancing a 35.6-millimeter-diameter cone equipped with a load cell and a friction sleeve through the soil profile. The cone is advanced at a rate of approximately 2 centimeters per second. Tip resistance, sleeve friction, and pore pressure at are typically recorded at 0.1-meter intervals. At select depths, the CPT advancement was suspended and pore water dissipation rates were measured. Shear wave velocity of the subsurface soil was also measured at variable increments.

NV5 / CPT-1 / 535 NW Inlet Ave Lincoln City

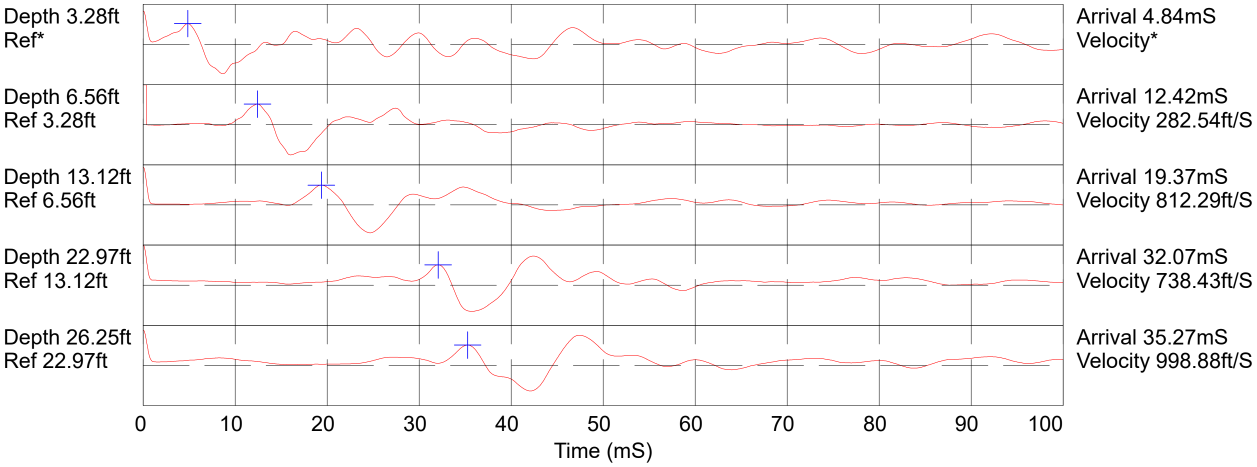
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 7/22/2022 10:31:56 AM



🟥 1 sensitive fine gra
▨ 2 organic materia
▨ 3 clay
🟦 4 silty clay to cl
▨ 5 clayey silt to silt
🟩 6 sandy silt to claye
▨ 7 silty sand to sandy
🟨 8 sand to silty sa
▨ 9 sand
🟧 10 gravelly sand to sand
▨ 11 very stiff fine grained (*)
▨ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: NV5 / CPT-1 / 535 NW Inlet Ave Lincoln City

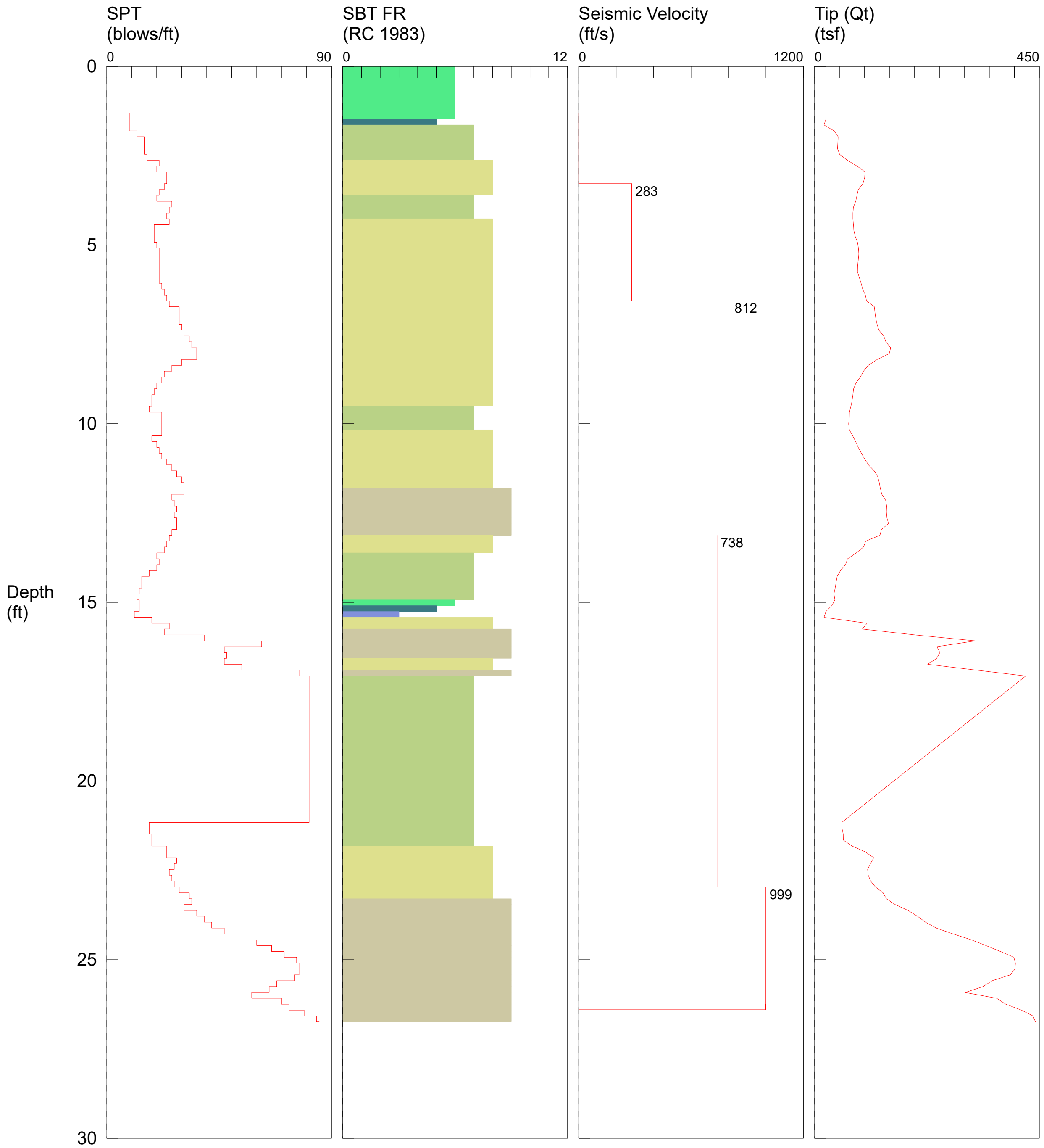


Hammer to Rod String Distance (ft): 5.58

* = Not Determined

NV5 / CPT-1 / 535 NW Inlet Ave Lincoln City

OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 7/22/2022 10:31:56 AM
TOTAL DEPTH: 26.739 ft

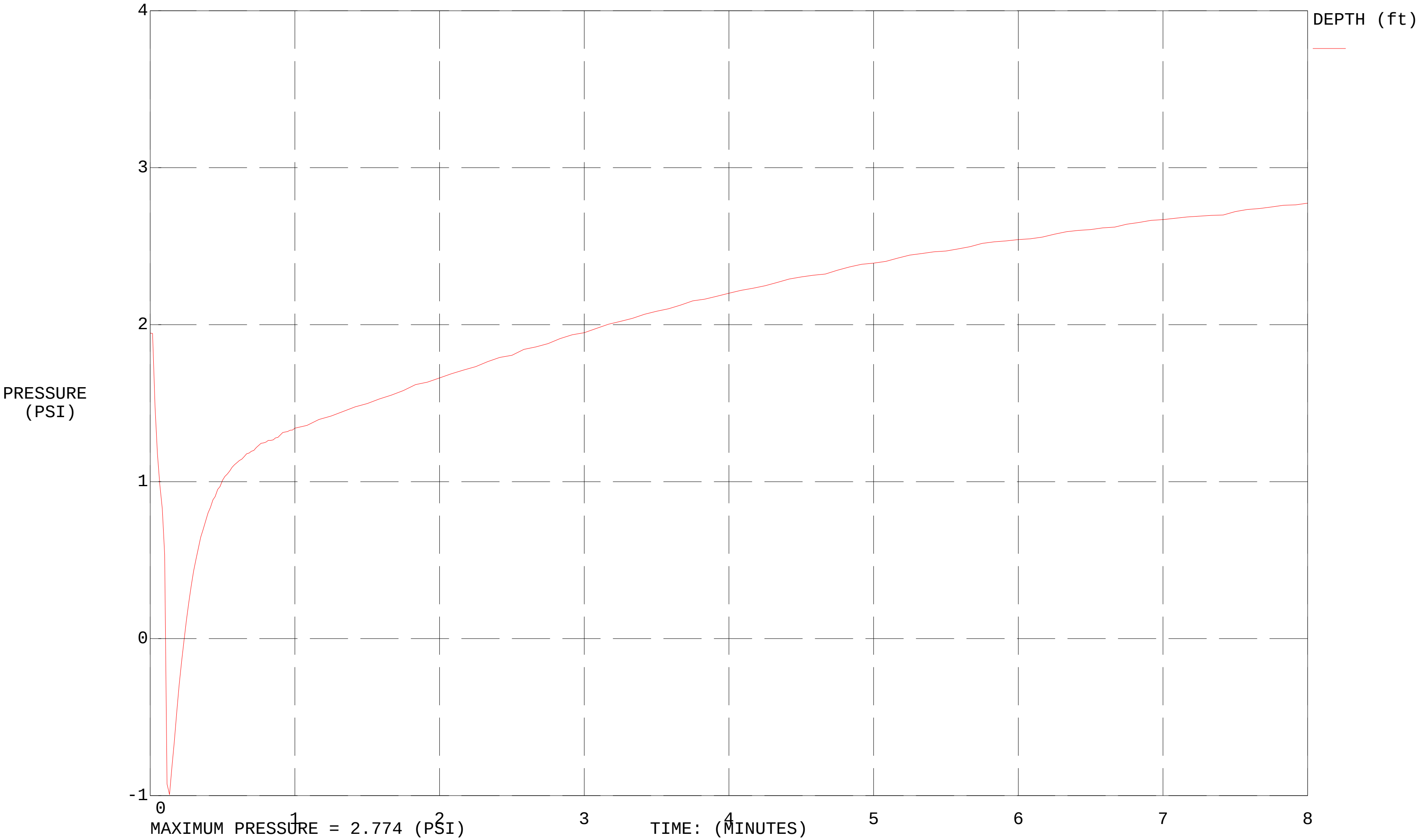


🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay
🟦 4 silty clay to clay
🟦 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟩 7 silty sand to sandy silt
🟨 8 sand to silty sand
🟫 9 sand
🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: NV5 / CPT-1 / 535 NW Inlet Ave Lincoln City

TEST DATE: 7/22/2022 10:31:56 AM

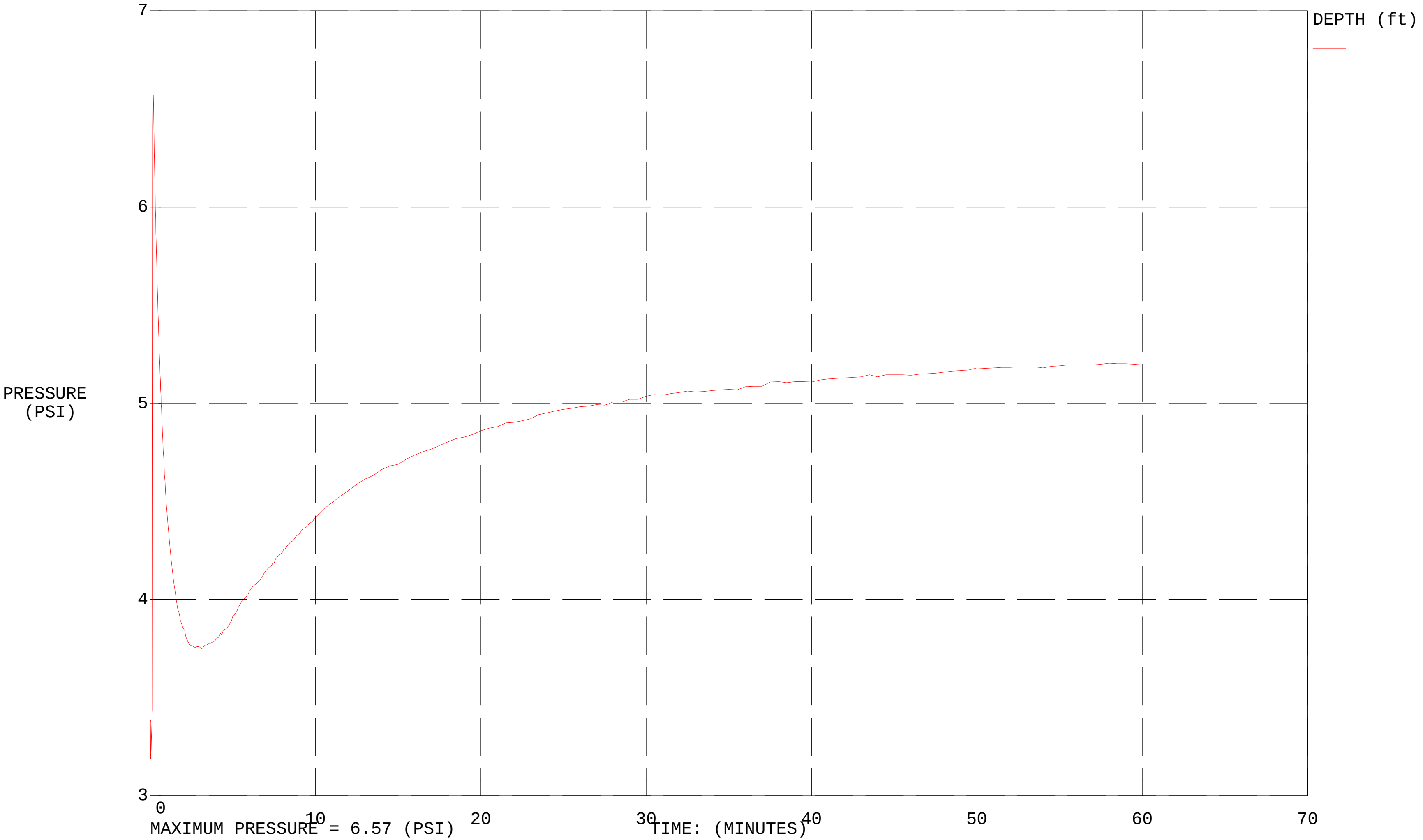


MAXIMUM PRESSURE = 2.774 (PSI)

HYDROSTATIC PRESSURE = 2.803 (PSI), WATER TABLE:

COMMENT: NV5 / CPT-1 / 535 NW Inlet Ave Lincoln City

TEST DATE: 7/22/2022 10:31:56 AM



MAXIMUM PRESSURE = 6.57 (PSI)
HYDROSTATIC PRESSURE = 6.855 (PSI), WATER TABLE:

NV5

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Improving Lives