

PLS

ENGINEERING

PRELIMINARY TECHNICAL INFORMATION REPORT

Paradise Park Site Plan

La Center, Washington

Prepared by:

**PLS Engineering
Consulting Engineers and Planners
604 W. Evergreen Blvd
Vancouver, WA 98660
PH: (360) 944-6519
pm@plsengineering.com**

Prepared for:

**Luke Sasse
9321 NE 72nd Ave Bldg C#7
Vancouver, WA 98665**

Submitted: May 2024

TABLE OF CONTENTS

CERTIFICATE OF ENGINEER	2
VICINITY MAPS.....	3
<i>(a). Site Location Map.....</i>	<i>3</i>
<i>(b). Soils Map</i>	<i>4</i>
SECTION A - PROJECT OVERVIEW	5
SECTION B - QUANTITY CONTROL ANALYSIS AND DESIGN	5
SECTION C - CONVEYANCE SYSTEMS ANALYSIS AND DESIGN.....	7
SECTION D - RUNOFF WATER QUALITY TREATMENT	7
SECTION E - SOILS EVALUATION	8
SECTION F - SPECIAL REPORTS AND STUDIES.....	8
SECTION G - OTHER PERMITS	8
SECTION H - MAINTENANCE AND OPERATIONS MANUAL	8

Technical Appendix:

Appendix A: Design Criteria

- Curve Numbers
- Manning's "n" Values
- Isopluvial Maps (2-, 10-, 25-, and 100-year)

Appendix B: Stormwater Models

- Preliminary Hydro CAD model
- Water Quality

Appendix C: Basin Maps

- Pre-Developed
- Post-Developed

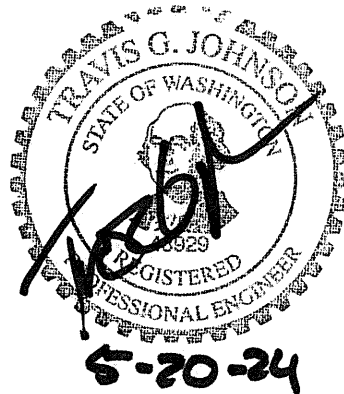
Appendix D: Geotechnical Report

Appendix E: Operations & Maintenance Manual


CERTIFICATE OF ENGINEER

***Paradise Park Site Plan
Preliminary Drainage Report***

The technical information and data contained in this report were prepared by the undersigned, whose seal, as a professional engineer licensed to practice as such, is affixed below.



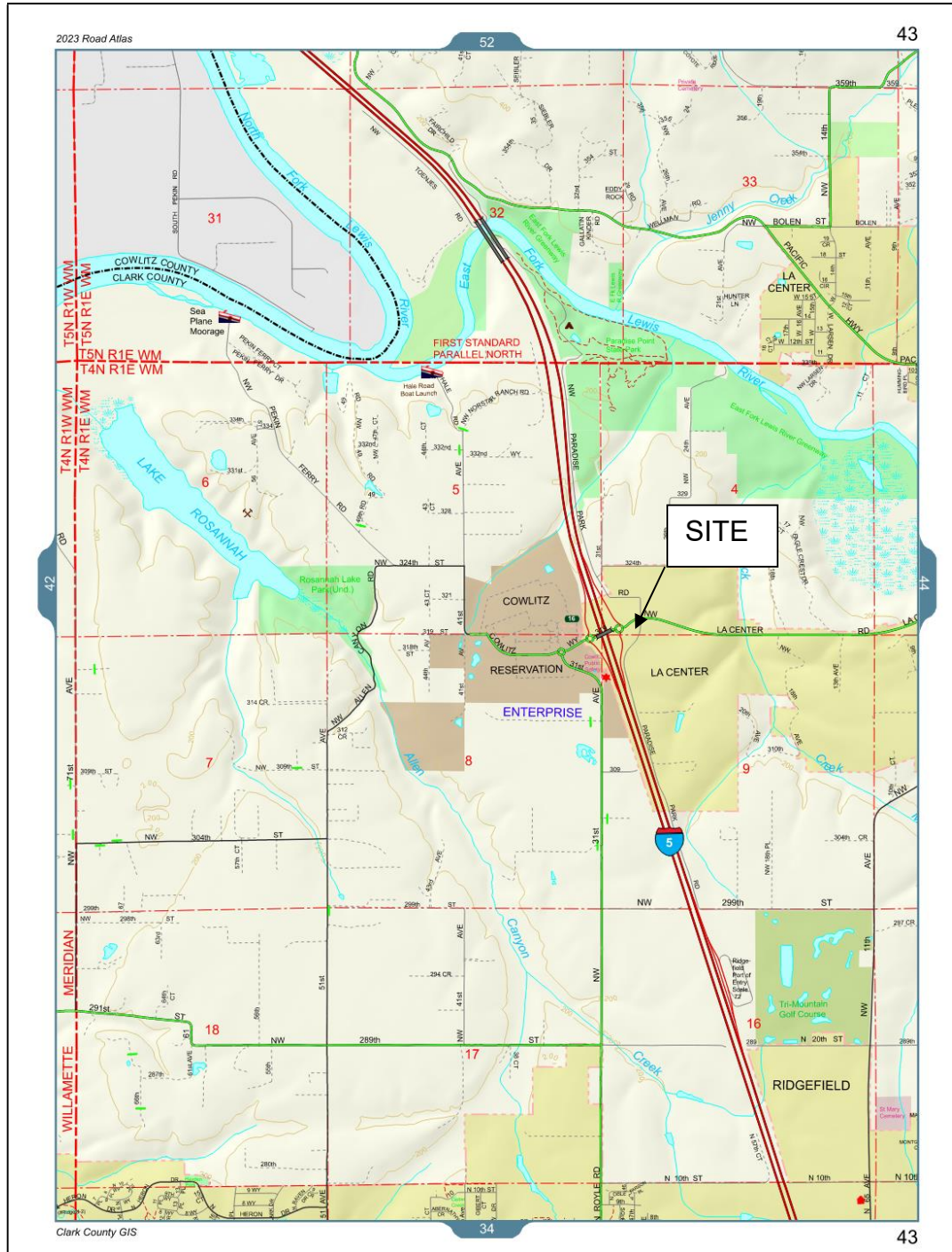
This document was:
Prepared by:


Travis G. Johnson, PE

VICINITY MAPS

(a) Site Location Map

Clark County Atlas
SW 1/4, S04, T4N, R1E La Center, WA



(b). Soils Map

USDA SCS Map 1" = 2130'

**Outlined Area of Interest (AOI) is an estimate of property boundary

Map Unit Legend:

GeD (Gee silt loam, 8-20% slopes):	62.2% of site
GeB (Gee silt loam, 0-8% slopes):	33.4% of site
OdB (Odne silt loam, 0-5% slopes):	4.4% of site



SECTION A – PROJECT OVERVIEW

Paradise Park Binding Site Plan is 146,501 square feet, 3.36-acre site located in La Center, WA. The site currently has no identifiable address per Clark County GIS but is identified by the Clark County Assessor's office as parcel 209708000 and further identified within the SW ¼ of section 4, T4N, R1E of the Willamette Meridian in Clark County, Washington. NW La Center Road borders the site to the North and Paradise Park Road borders the site to the west. To the south is land owned by the Cowlitz Indian Tribe that contains a storm facility and to the east is a Type Ns unnamed stream.

The site's existing topography is moderately sloped with slopes ranging from 3-11% sloping from the northwest to the southeast corner. Along the east boundary line, the site begins to increase in grade as it gets closer to the offsite stream.

Paradise Park Site Plan proposes a hotel on a lot and a drive thru end cap with a commercial space on another lot.

Due to negligible infiltration rates onsite, the project will utilize an underground detention system with a flow control structure to store and release stormwater runoff to a flow spreader located at the east side of the site, being the natural discharge location. Stormwater runoff will be piped to a treatment vault for treatment before being routed to the detention pond.

The combined impervious area generated by the project includes approximately 141,284 ft². NW La Center Road sheds to the south directing all runoff from the entire roadway to the frontage of the site. PLS proposes to provide treatment for the entire roadway, however PLS is not proposing flow control for the existing asphalt. Flow control is proposed for the entire site and for the widening that is proposed for the roadway. There is a small portion of roadway that can not be detained or treated because of the low elevations at the east side of the site. PLS will over-detain and over-treat for the runoff that cannot be captured to meet the City of La Center minimum requirements.

SECTION B – QUANTITY CONTROL ANALYSIS AND DESIGN

Per Chapter 18.320 of the La Center Municipal Code (LCMC), the site will be required to mitigate for stormwater runoff impacts generated as a result of the proposed improvements. The hydrologic analysis of this site was performed in accordance with the guidelines contained in LCMC and Chapters III-1 and III-2 of the Puget Sound Manual. The storm events were assumed to have a 24-hour duration and follow a Type II storm distribution. Rainfall depth for the 2, 10, 25, and 100-year 24-hour storm events are 2.2, 3.2, 3.7, and 4.1 inches respectively, as obtained from the Isopluvial maps for Clark County included in Appendix A. The detention facility has been designed to release runoff at rates equal to or less than the predevelopment peak runoff rates for the 2, 10, 25 and 100-year, 24-hour storm events as stated in LCMC Code Section 18.320.220 (3)(d)(i). In addition, the facility has been designed utilizing Figure III-1.1 Volume Correction Factor from the Puget Sound Manual. This resulted in a correction factor of 1.45 for the detention facilities or a storage multiplier of 0.69.

The live storage area of the stormwater facilities was assumed to be empty at the beginning of the design storm event. The hydrological analysis was completed using HydroCAD v 10.0, which allows the SCS TR-20 method of hydrograph routing to be utilized and the TR-55 method to determine the times of concentration. The soil characteristics were obtained from USDA NRCS website. As can be seen on the soils map located in the appendix of this report, there are multiple soil types covering this site. These soil types consist of hydrologic soil groups (HSG) C. The Runoff Curve Numbers (RCNs) that were used in the design of the project were taken from Table

III-1.3 of the Puget Sound Manual. An RCN value of 76 was used for the HSG C soil with the assumption that the site is forested for predeveloped conditions. An RCN value of 69 was used for post-development landscaping and an RCN value of 98 was used for impervious areas. The offsite basin for post-development flows was modeled with an RCN value of 98 for the existing roadways area that is being routed through the detention system.

Tables 1 & 2 below shows a tabulation of the project site areas for pre- and post-developed conditions.

Table 1- Summary of Pre-Developed Areas

	Basin	Area (sq-ft)	Total (acres)
Pre-Developed Area			
	Onsite	164014	3.765
	Offsite	30,329	0.696

Table 2- Summary of Post-Developed Areas

Existing hard surface to remain	0 ft ²
New hard surface	141,284 ft ²
Replaced hard surface	0 ft ²
Native vegetation converted to lawn or landscaping	22,730 ft ²
Native vegetation converted to pasture	0 ft ²
Total land-disturbing activity	150,000 ft ²
Pollution-generating hard surface	98,500 ft ²
Pollution-generating pervious surface	0 ft ²
Total pollution-generating surfaces	98,500 ft ²
Total non-pollution-generating surfaces	22,730 ft ²

For the post-development prelim analysis a single basin was modeled with all runoff draining to the designed detention facility located at the southeast corner of the property.

The HydroCAD model basin includes offsite basin and an onsite basin for pre-development and one basin for post-development which is shown on the basin maps found in Appendix C of this report. RCN values of 98 were used for impervious areas, 76 for forested conditions and 69 was used for landscaping areas that will be amended post construction. Please refer to the HydroCAD stormwater model located in Appendix B, for tabulated acreage, imperviousness, curve numbers, length and grade of overland flow, and other hydrological parameters used in completing the analysis. Basin Maps are included in Appendix C.

Water quantity control for the development will be accomplished utilizing a detention facility. The underground detention facility will be constructed in the southeast corner of the site and will be accessed by a. A flow control structure with 3 orifices has been designed to release detained stormwater at rates equal to or less than the predeveloped discharge rates for the 2, 10, 25 and 100 year design storm events. See Appendix B for the HydroCAD report.

SECTION C – CONVEYANCE SYSTEMS ANALYSIS AND DESIGN

The pipes for the conveyance system will be designed for the 100-year storm event per LCMC 18.320.220 and will be sized to carry flows from the contributing drainage areas upon full buildout while operating in an open flow regime. The conveyance calculations for the stormwater pipes will be included with the Final TIR.

SECTION D – RUNOFF WATER QUALITY TREATMENT

Runoff from pollution generating surfaces will be treated using a Peak Diversion Stormfilter vault. Stormfilter Media cartridge systems supplied by Contech Engineered Solutions will be used to treat stormwater runoff from the site's impervious surfaces. Runoff will be treated through the vault before being piped to the detention facility. The number of required Stormfilter cartridges in the system will be based on the water quality treatment flow rate calculated for pollution generating and non-pollution generating surfaces and the treatment capacity of the filters supplied by Contech Engineered Solutions. Stormfilters have gained Washington Department of Ecology approval and have been allowed to be sized as offline systems because peak storms bypass the Stormfilter treatment chamber via an inlet/bypass assembly. See Appendix B for water quality flow rates.

SECTION E – SOILS EVALUATION

There are two identified soil types located on this site. A soils map, obtained from USDA NRCS website is located before the narrative of this report. The soil types onsite consist of Gee silt loam and Odne silt loam. These soils are in hydrologic soil group (HSG) C, except Odne silt loam is in hydrologic soil group D, and is mapped as a hydric soil. Since there is only a small portion Odne soils all hydrological calculations used a HSG C soil for the entire site.

SECTION F – SPECIAL REPORTS AND STUDIES

A geotechnical report, traffic analysis report, archeological report and critical areas report were all completed for this site. All reports have been included as part of the subdivision application. The Geotechnical Report is provided in Appendix D.

SECTION G – OTHER PERMITS

No other permits to be submitted for this site at this time.

SECTION H – MAINTENANCE AND OPERATIONS MANUAL

All of the stormwater facilities associated with this development are to be owned & maintained by the property owner. A maintenance and operations manual will be provided during final engineering.

APPENDIX A

Design Criteria

Curve Numbers

Manning's "n" Values

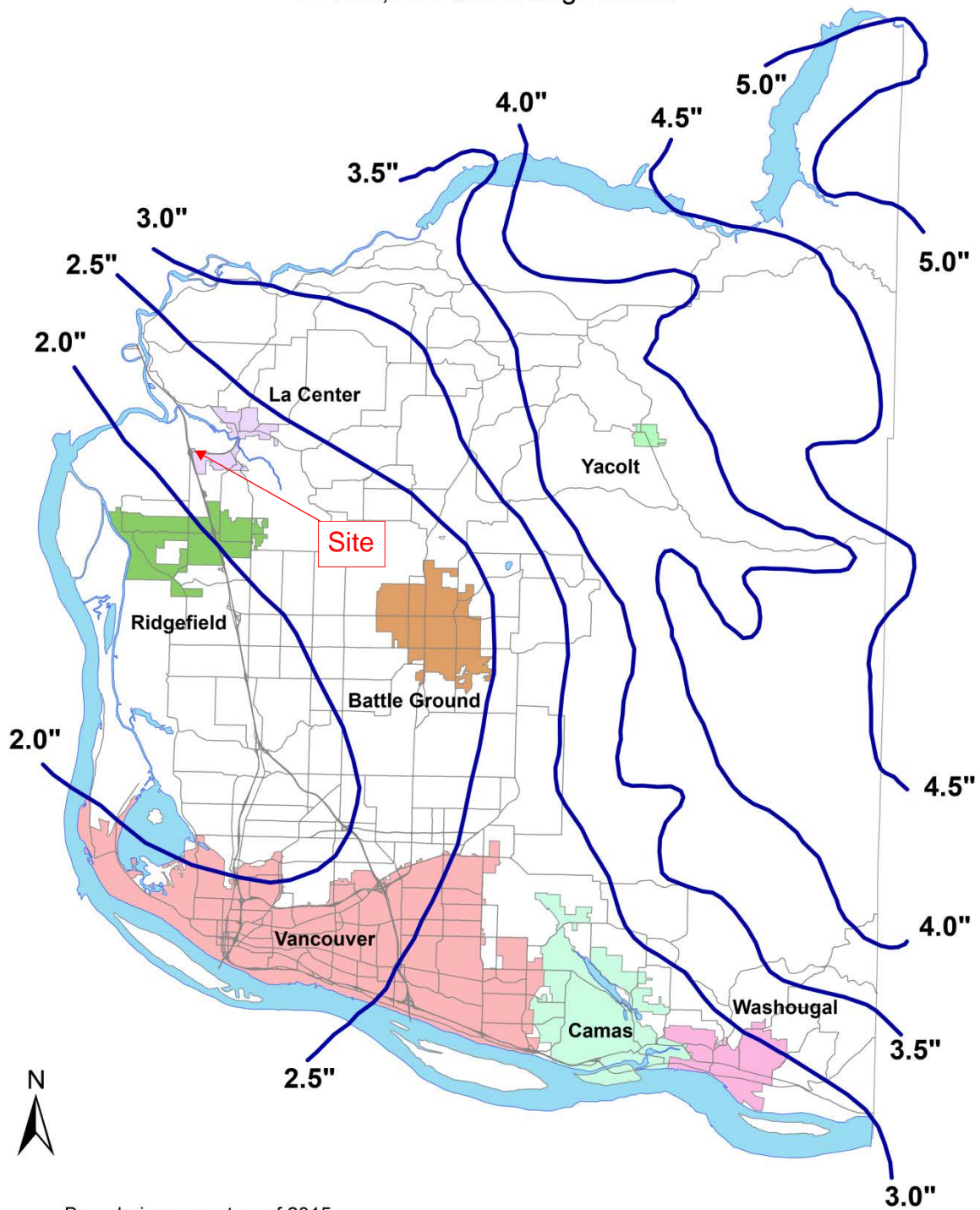
Isopluvial Maps (2-, 10-, 25- and 100-Year)

Table III-1.3 SCS Western Washington Runoff Curve Numbers
(Published by SCS in 1982) Runoff curve numbers for selected agricultural, suburban and urban land use for Type 1A rainfall distribution, 24-hour storm duration.

LAND USE DESCRIPTION		CURVE NUMBERS BY HYDROLOGIC SOIL GROUP			
		A	B	C	D
Cultivated land(1):	winter condition	86	91	94	95
Mountain open areas:	low growing brush & grasslands	74	82	89	92
Meadow or pasture:		65	78	85	89
Wood or forest land:	undisturbed	42	64	76	81
Wood or forest land:	young second growth or brush	55	72	81	86
Orchard:	with cover crop	81	88	92	94
Open spaces, lawns, parks, golf courses, cemeteries, landscaping.					
Good condition:	grass cover on $\geq 75\%$ of the area	68	80	86	90
Fair condition:	grass cover on 50-75% of the area	77	85	90	92
Gravel roads & parking lots:		76	85	89	91
Dirt roads & parking lots:		72	82	87	89
Impervious surfaces, pavement, roofs etc.		98	98	98	98
Open water bodies: lakes, wetlands, ponds etc.		100	100	100	100
Single family residential(2):		Separate curve number shall be selected for pervious & impervious portions of the site or basin			
Dwelling Unit/Gross Acre	%Impervious(3)				
1.0 DU/GA	15				
1.5 DU/GA	20				
2.0 DU/GA	25				
2.5 DU/GA	30				
3.0 DU/GA	34				
3.5 DU/GA	38				
4.0 DU/GA	42				
4.5 DU/GA	46				
5.0 DU/GA	48				
5.5 DU/GA	50				
6.0 DU/GA	52				
6.5 DU/GA	54				
7.0 DU/GA	56				
PUD's, condos, apartments, commercial businesses & industrial areas	%impervious must be computed				

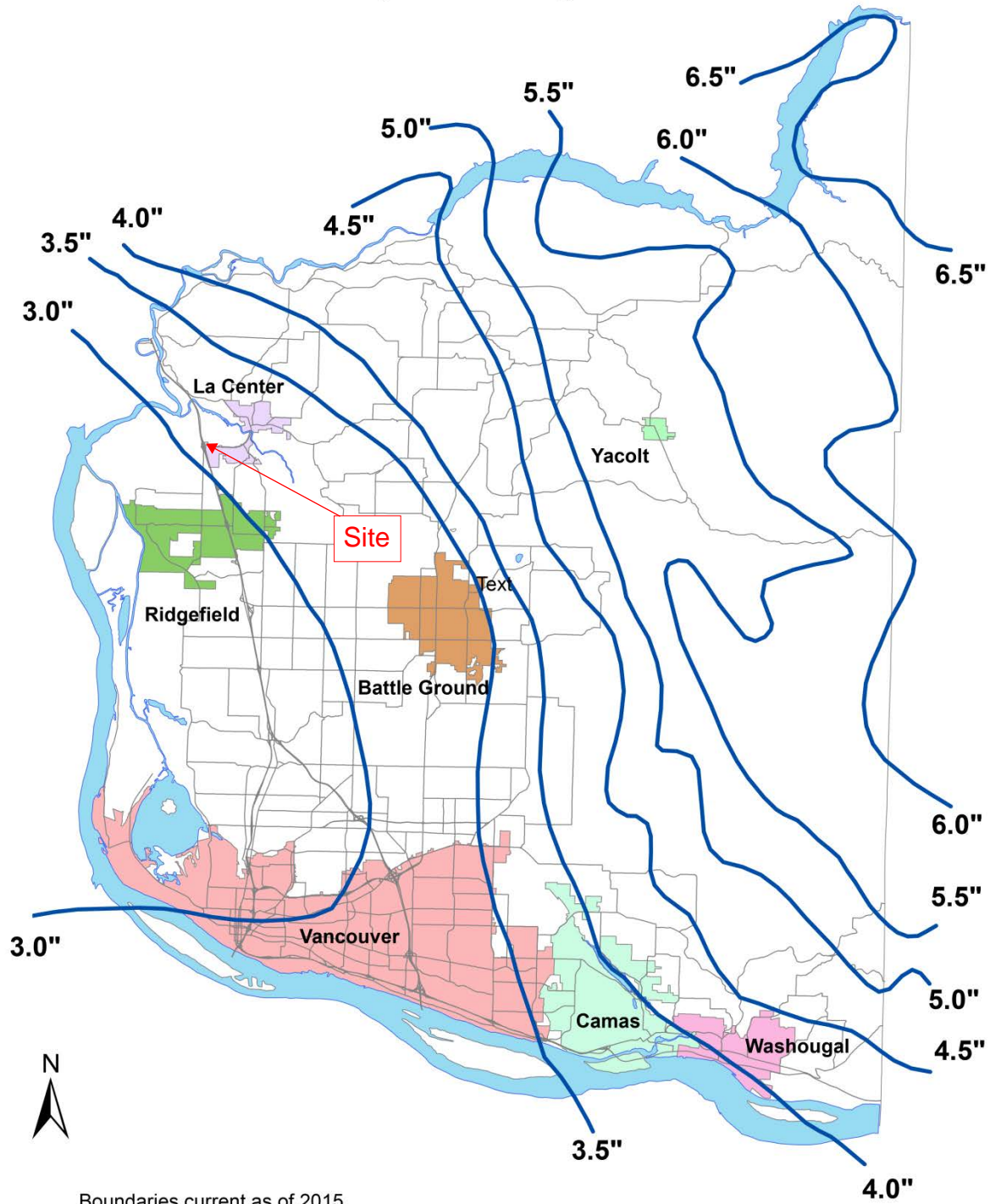
- (1) For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Sec. 4, Hydrology, Chapter 9, August 1972.
- (2) Assumes roof and driveway runoff is directed into street/storm system.
- (3) The remaining pervious areas (lawn) are considered to be in good condition for these curve numbers.

Isopluvial Map for Clark County 2-Year, 24-Hour Design Storm

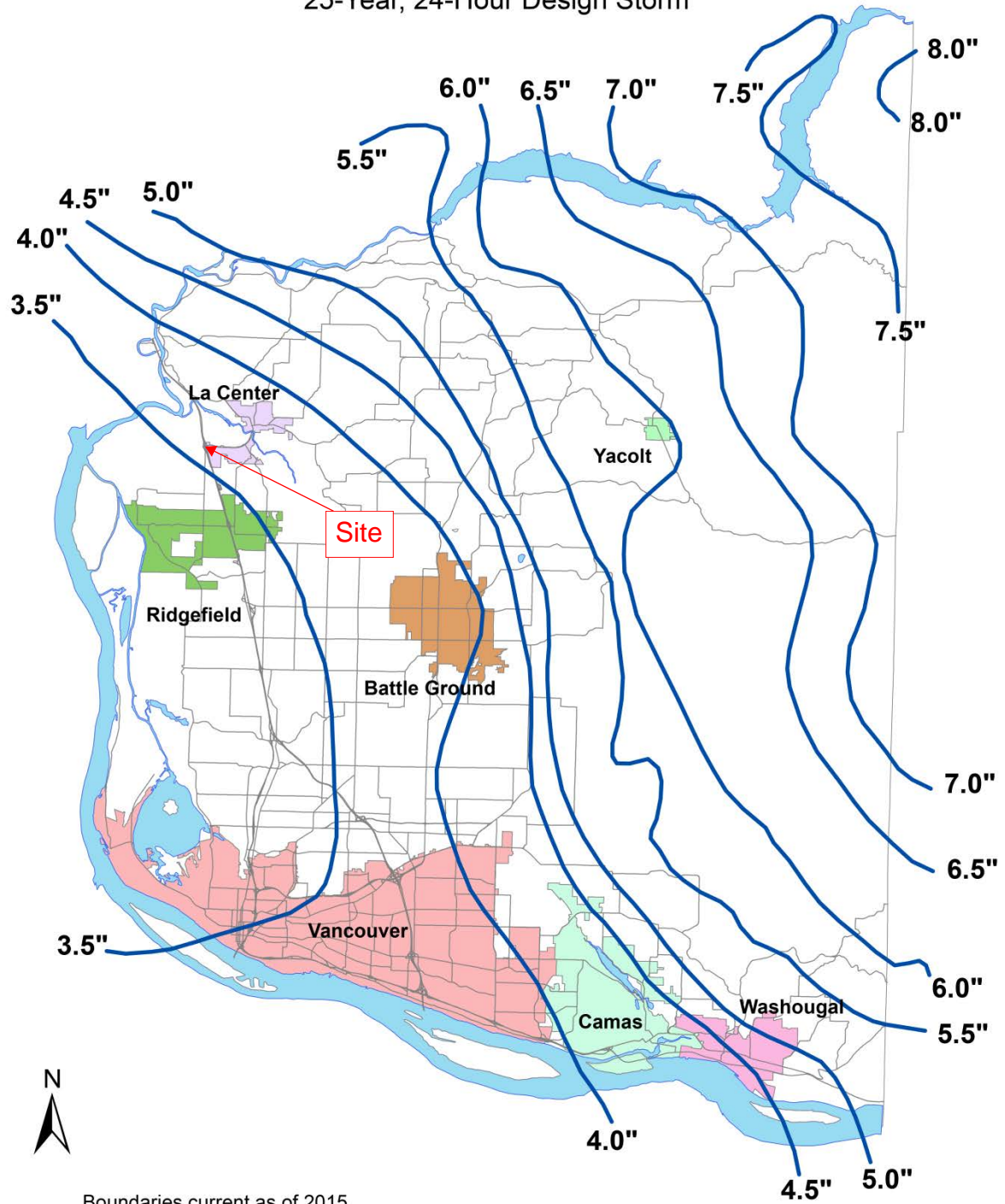


Boundaries current as of 2015

Isopluvial Map for Clark County
10-Year, 24-Hour Design Storm

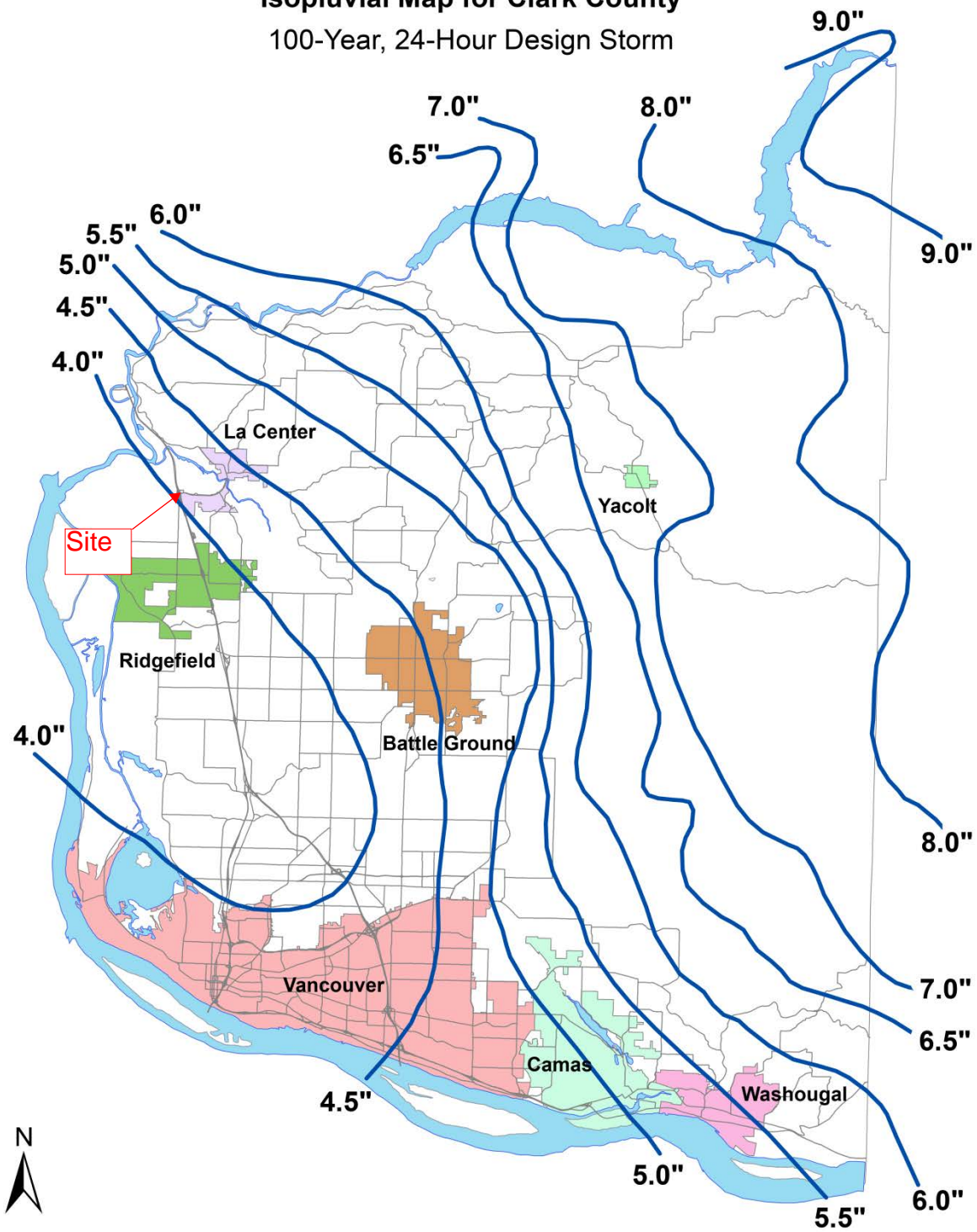


Isopluvial Map for Clark County 25-Year, 24-Hour Design Storm



Boundaries current as of 2015.

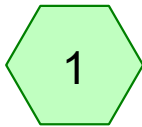
Isopluvial Map for Clark County **100-Year, 24-Hour Design Storm**



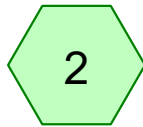
Boundaries current as of 2015.

APPENDIX B

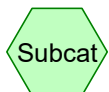
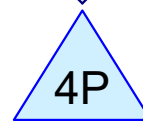
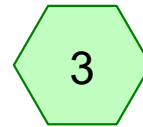
Stormwater Models



Onsite



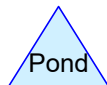
Offsite



Subcat



Reach



Pond



Link

Routing Diagram for Prelim Pond

Prepared by HP Inc., Printed 5/30/2024

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Printed 5/30/2024

Page 2

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
0.522	69	Landscape (3)
3.765	76	Onsite Pre-developed SG C / Forested (1)
3.243	98	Impervious (3)
0.696	98	OFFSITE Roadway (2)
8.227	86	TOTAL AREA

Prelim Pond

Prepared by HP Inc.

Printed 5/30/2024

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Page 3

Soil Listing (all nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
0.000	HSG B	
0.000	HSG C	
0.000	HSG D	
8.227	Other	1, 2, 3
8.227		TOTAL AREA

Prelim Pond

Prepared by HP Inc.

Printed 5/30/2024

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Page 4

Ground Covers (all nodes)

HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcat Number
0.000	0.000	0.000	0.000	3.243	3.243	Impervious	
0.000	0.000	0.000	0.000	0.522	0.522	Landscape	
0.000	0.000	0.000	0.000	0.696	0.696	OFFSITE Roadway	
0.000	0.000	0.000	0.000	3.765	3.765	Onsite Pre-developed SG C / Forested	
0.000	0.000	0.000	0.000	8.227	8.227	TOTAL AREA	

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 2 Year Rainfall=2.20"

Printed 5/30/2024

Page 5

Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points

Runoff by SCS TR-20 method, UH=SCS

Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1: Onsite

Runoff Area=164,014 sf 0.00% Impervious Runoff Depth>0.46"
Flow Length=311' Tc=21.2 min CN=76 Runoff=1.82 cfs 0.144 af

Subcatchment 2: Offsite

Runoff Area=30,329 sf 100.00% Impervious Runoff Depth>1.84"
Tc=6.0 min CN=98 Runoff=2.08 cfs 0.107 af

Subcatchment 3:

Runoff Area=164,014 sf 86.14% Impervious Runoff Depth>1.48"
Tc=6.0 min CN=94 Runoff=9.83 cfs 0.465 af

Pond 4P:

Peak Elev=2.74' Storage=5,990 cf Inflow=9.83 cfs 0.465 af
Outflow=3.63 cfs 0.458 af

Total Runoff Area = 8.227 ac Runoff Volume = 0.716 af Average Runoff Depth = 1.04"
52.11% Pervious = 4.287 ac 47.89% Impervious = 3.940 ac

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 2 Year Rainfall=2.20"

Printed 5/30/2024

Page 6

Summary for Subcatchment 1: Onsite

Runoff = 1.82 cfs @ 12.17 hrs, Volume= 0.144 af, Depth> 0.46"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

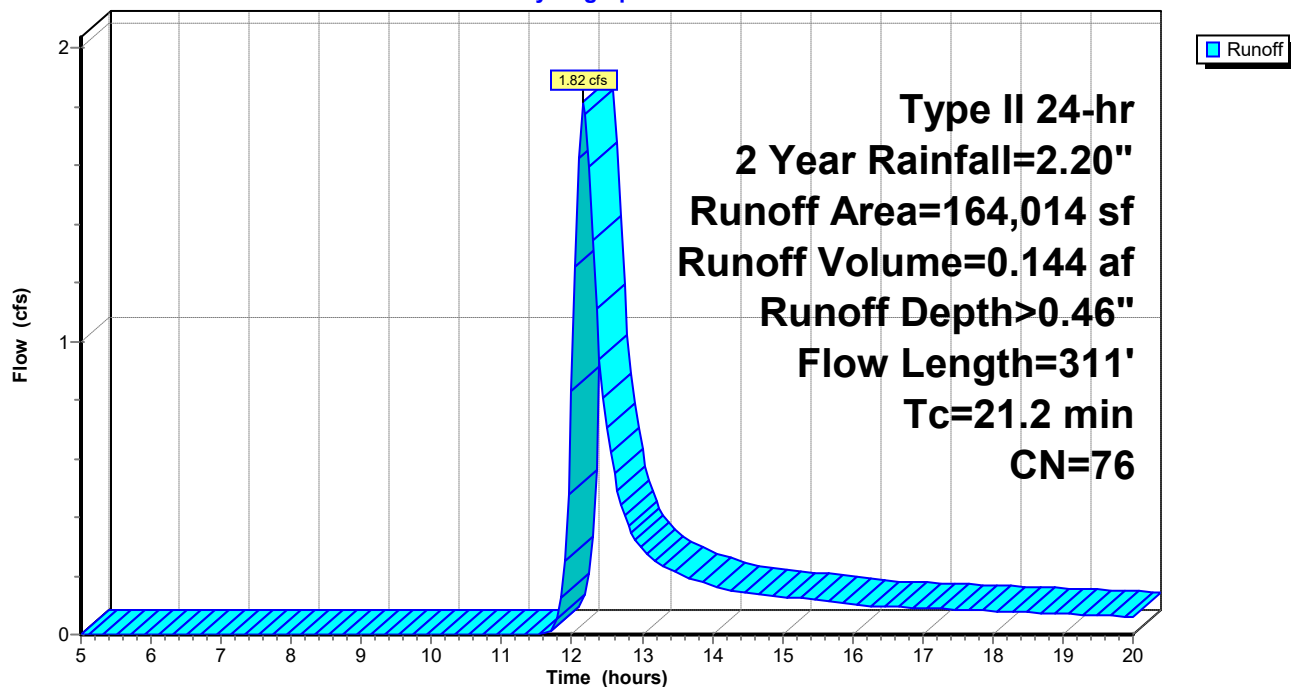
Type II 24-hr 2 Year Rainfall=2.20"

Area (sf)	CN	Description
* 164,014	76	Onsite Pre-developed SG C / Forested
164,014		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
21.1	300	0.0400	0.24		Sheet Flow, Grass: Short n= 0.150 P2= 2.30"
0.1	11	0.0660	1.80		Shallow Concentrated Flow, Short Grass Pasture Kv= 7.0 fps
21.2	311	Total			

Subcatchment 1: Onsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 2 Year Rainfall=2.20"

Printed 5/30/2024

Page 7

Summary for Subcatchment 2: Offsite

Runoff = 2.08 cfs @ 11.96 hrs, Volume= 0.107 af, Depth> 1.84"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

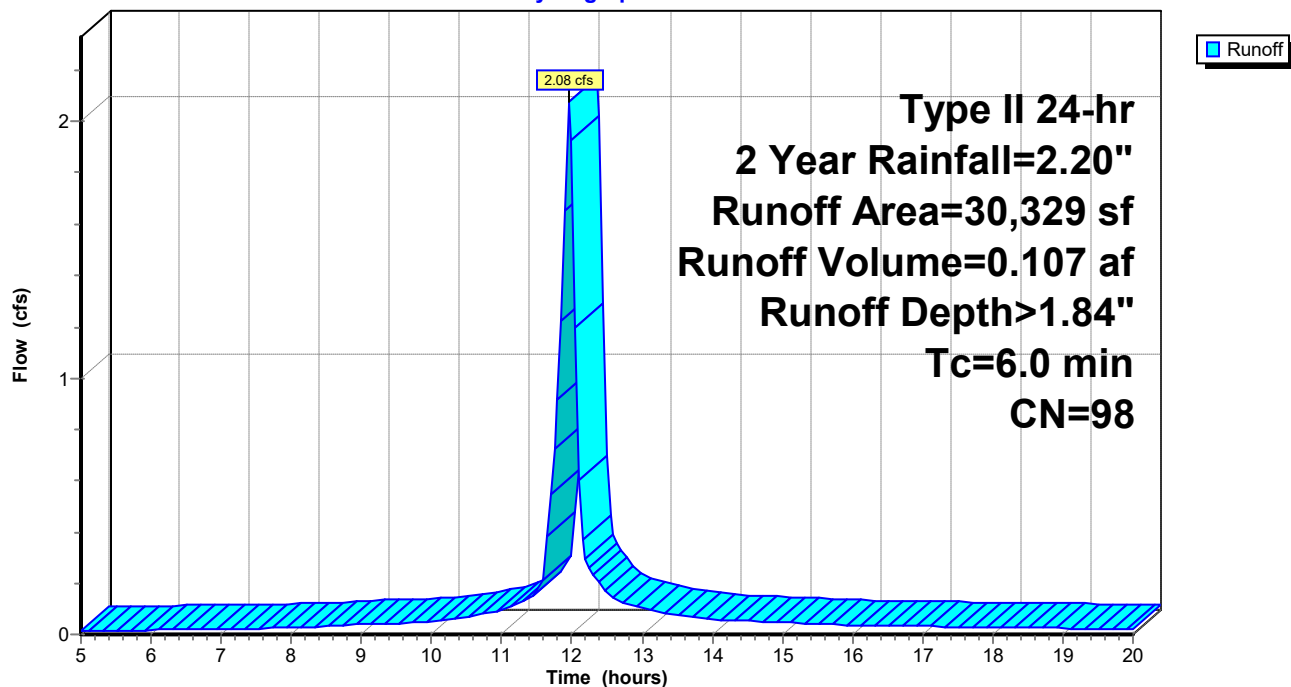
Type II 24-hr 2 Year Rainfall=2.20"

	Area (sf)	CN	Description
*	30,329	98	OFFSITE Roadway
	30,329		100.00% Impervious Area

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

Subcatchment 2: Offsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 2 Year Rainfall=2.20"

Printed 5/30/2024

Page 8

Summary for Subcatchment 3:

Runoff = 9.83 cfs @ 11.97 hrs, Volume= 0.465 af, Depth> 1.48"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

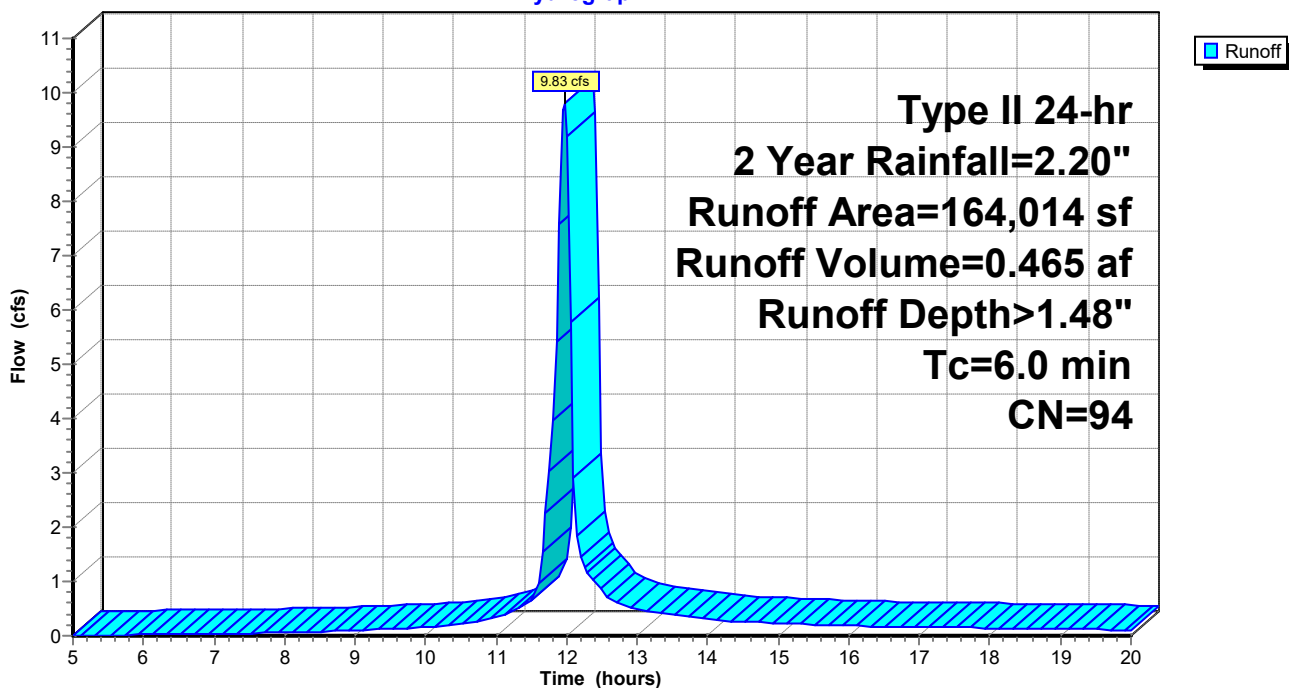
Type II 24-hr 2 Year Rainfall=2.20"

	Area (sf)	CN	Description
*	141,284	98	Impervious
*	22,730	69	Landscape
	164,014	94	Weighted Average
	22,730		13.86% Pervious Area
	141,284		86.14% Impervious Area

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

Subcatchment 3:

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 2 Year Rainfall=2.20"

Printed 5/30/2024

Page 9

Summary for Pond 4P:

Inflow Area = 3.765 ac, 86.14% Impervious, Inflow Depth > 1.48" for 2 Year event
Inflow = 9.83 cfs @ 11.97 hrs, Volume= 0.465 af
Outflow = 3.63 cfs @ 12.09 hrs, Volume= 0.458 af, Atten= 63%, Lag= 7.7 min
Primary = 3.63 cfs @ 12.09 hrs, Volume= 0.458 af

Routing by Stor-Ind method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Peak Elev= 2.74' @ 12.09 hrs Surf.Area= 0 sf Storage= 5,990 cf

Plug-Flow detention time= 26.8 min calculated for 0.456 af (98% of inflow)
Center-of-Mass det. time= 20.2 min (781.5 - 761.3)

Volume	Invert	Avail.Storage	Storage Description
#1	0.00'	26,908 cf	Custom Stage Data Listed below x 0.69

Elevation (feet)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
0.00	0	0
2.35	6,499	6,499
4.69	12,999	19,498
7.00	19,499	38,997

Device	Routing	Invert	Outlet Devices
#1	Primary	0.00'	9.5" Vert. Orifice/Grate C= 0.600
#2	Primary	2.80'	10.0" Vert. Orifice/Grate C= 0.600

Primary OutFlow Max=3.63 cfs @ 12.09 hrs HW=2.74' (Free Discharge)

↑ **1=Orifice/Grate** (Orifice Controls 3.63 cfs @ 7.37 fps)

└ **2=Orifice/Grate** (Controls 0.00 cfs)

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

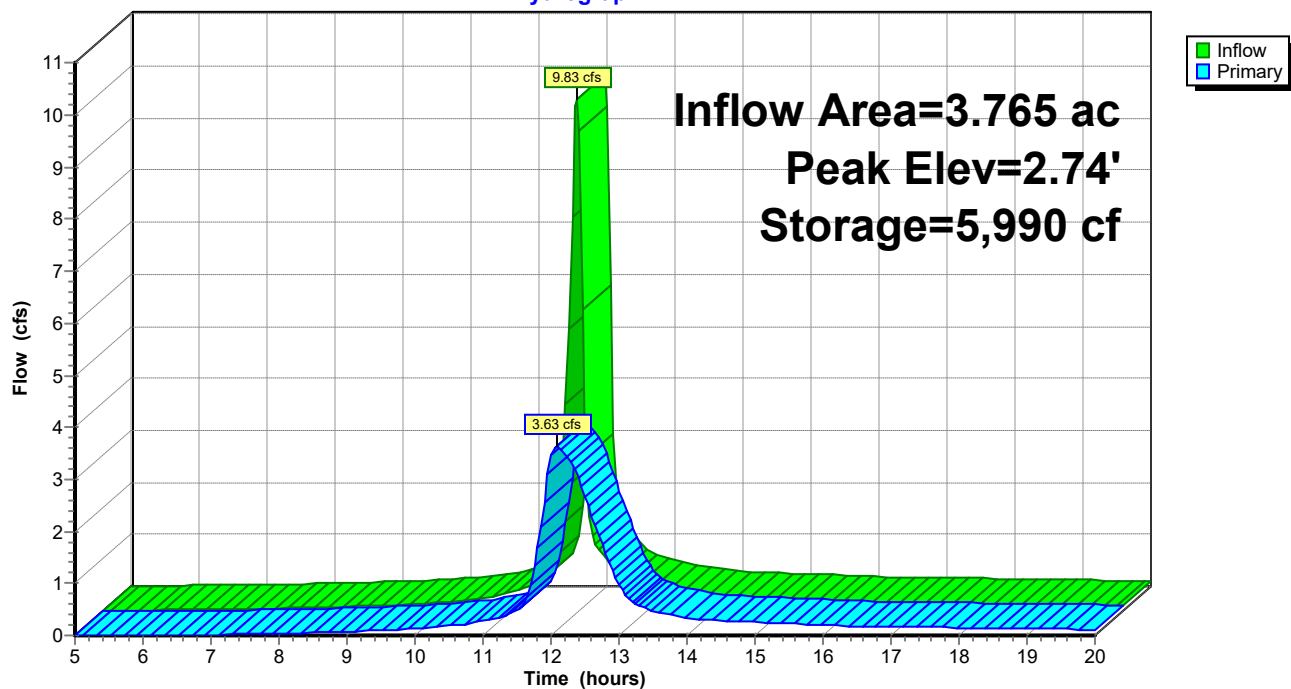
Type II 24-hr 2 Year Rainfall=2.20"

Printed 5/30/2024

Page 10

Pond 4P:

Hydrograph



Prelim Pond*Type II 24-hr 10 Year Rainfall=3.20"*

Prepared by HP Inc.

Printed 5/30/2024

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Page 11

Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points

Runoff by SCS TR-20 method, UH=SCS

Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1: OnsiteRunoff Area=164,014 sf 0.00% Impervious Runoff Depth>1.04"
Flow Length=311' Tc=21.2 min CN=76 Runoff=4.48 cfs 0.326 af**Subcatchment 2: Offsite**Runoff Area=30,329 sf 100.00% Impervious Runoff Depth>2.74"
Tc=6.0 min CN=98 Runoff=3.06 cfs 0.159 af**Subcatchment 3:**Runoff Area=164,014 sf 86.14% Impervious Runoff Depth>2.39"
Tc=6.0 min CN=94 Runoff=15.34 cfs 0.749 af**Pond 4P:**Peak Elev=3.72' Storage=9,736 cf Inflow=15.34 cfs 0.749 af
Outflow=6.19 cfs 0.739 af**Total Runoff Area = 8.227 ac Runoff Volume = 1.233 af Average Runoff Depth = 1.80"**
52.11% Pervious = 4.287 ac 47.89% Impervious = 3.940 ac

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 10 Year Rainfall=3.20"

Printed 5/30/2024

Page 12

Summary for Subcatchment 1: Onsite

Runoff = 4.48 cfs @ 12.15 hrs, Volume= 0.326 af, Depth> 1.04"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

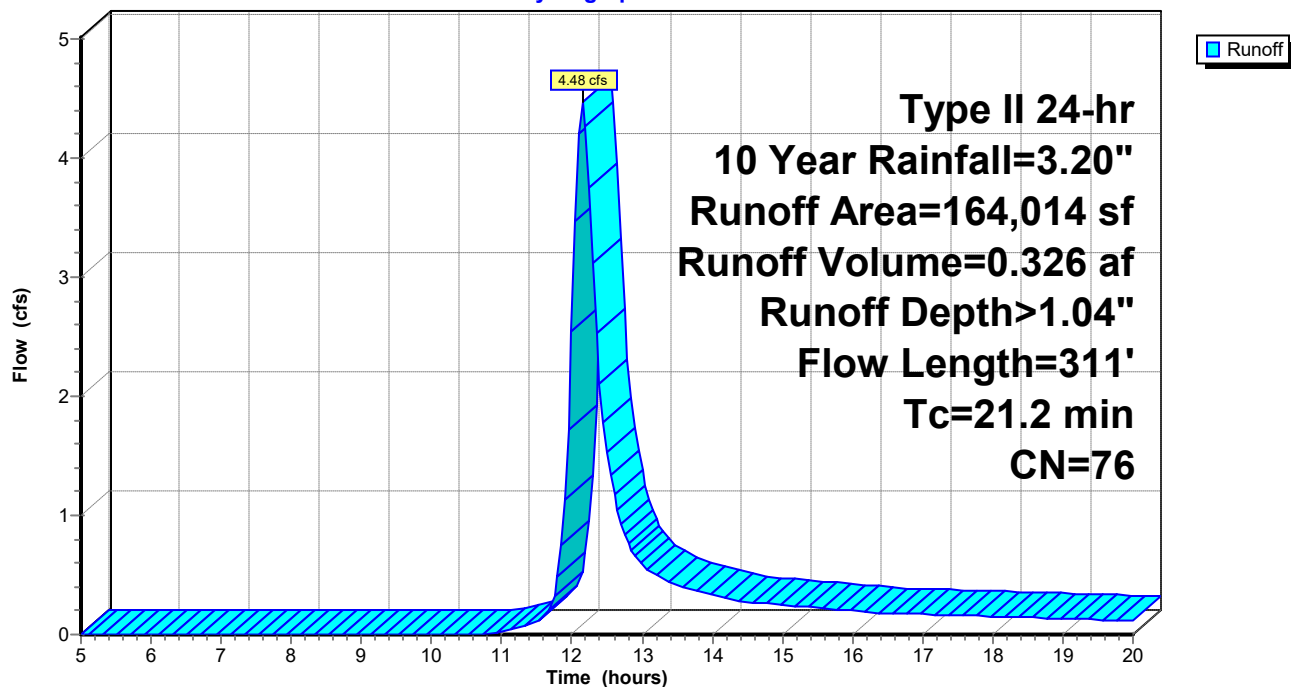
Type II 24-hr 10 Year Rainfall=3.20"

Area (sf)	CN	Description
* 164,014	76	Onsite Pre-developed SG C / Forested
164,014		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
21.1	300	0.0400	0.24		Sheet Flow, Grass: Short n= 0.150 P2= 2.30"
0.1	11	0.0660	1.80		Shallow Concentrated Flow, Short Grass Pasture Kv= 7.0 fps
21.2	311	Total			

Subcatchment 1: Onsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 10 Year Rainfall=3.20"

Printed 5/30/2024

Page 13

Summary for Subcatchment 2: Offsite

Runoff = 3.06 cfs @ 11.96 hrs, Volume= 0.159 af, Depth> 2.74"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

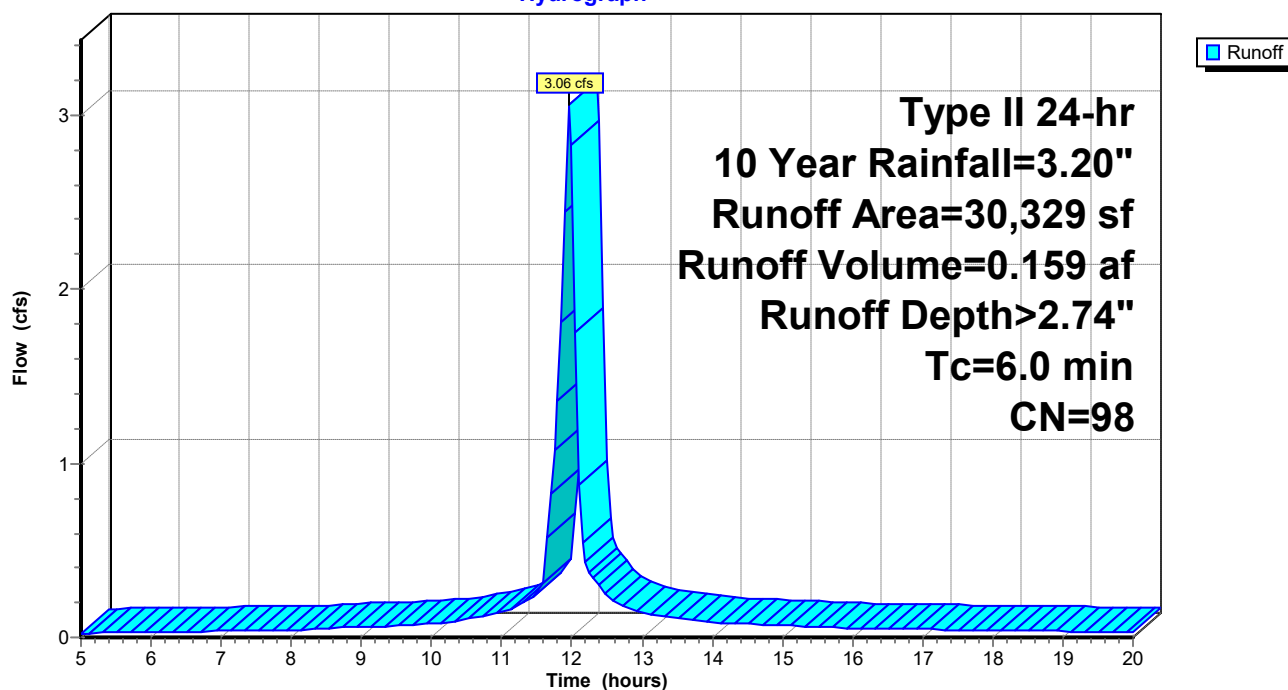
Type II 24-hr 10 Year Rainfall=3.20"

	Area (sf)	CN	Description
*	30,329	98	OFFSITE Roadway
	30,329		100.00% Impervious Area

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

Subcatchment 2: Offsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 10 Year Rainfall=3.20"

Printed 5/30/2024

Page 14

Summary for Subcatchment 3:

Runoff = 15.34 cfs @ 11.96 hrs, Volume= 0.749 af, Depth> 2.39"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

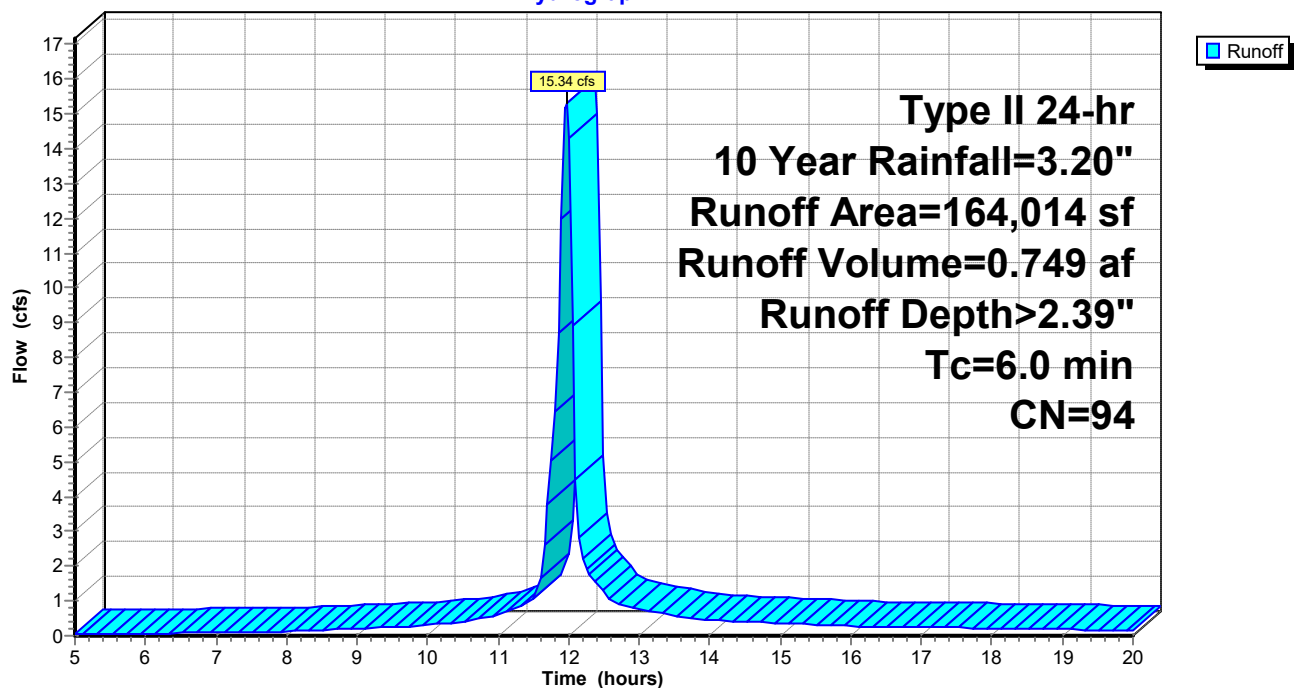
Type II 24-hr 10 Year Rainfall=3.20"

	Area (sf)	CN	Description
*	141,284	98	Impervious
*	22,730	69	Landscape
	164,014	94	Weighted Average
	22,730		13.86% Pervious Area
	141,284		86.14% Impervious Area

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

Subcatchment 3:

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 10 Year Rainfall=3.20"

Printed 5/30/2024

Page 15

Summary for Pond 4P:

[82] Warning: Early inflow requires earlier time span

Inflow Area = 3.765 ac, 86.14% Impervious, Inflow Depth > 2.39" for 10 Year event
Inflow = 15.34 cfs @ 11.96 hrs, Volume= 0.749 af
Outflow = 6.19 cfs @ 12.08 hrs, Volume= 0.739 af, Atten= 60%, Lag= 7.2 min
Primary = 6.19 cfs @ 12.08 hrs, Volume= 0.739 af

Routing by Stor-Ind method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

Peak Elev= 3.72' @ 12.08 hrs Surf.Area= 0 sf Storage= 9,736 cf

Plug-Flow detention time= 26.3 min calculated for 0.739 af (99% of inflow)

Center-of-Mass det. time= 20.9 min (772.1 - 751.2)

Volume	Invert	Avail.Storage	Storage Description
#1	0.00'	26,908 cf	Custom Stage Data Listed below x 0.69

Elevation (feet)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
0.00	0	0
2.35	6,499	6,499
4.69	12,999	19,498
7.00	19,499	38,997

Device	Routing	Invert	Outlet Devices
#1	Primary	0.00'	9.5" Vert. Orifice/Grate C= 0.600
#2	Primary	2.80'	10.0" Vert. Orifice/Grate C= 0.600

Primary OutFlow Max=6.15 cfs @ 12.08 hrs HW=3.71' (Free Discharge)

↑ **1=Orifice/Grate** (Orifice Controls 4.31 cfs @ 8.76 fps)

└ **2=Orifice/Grate** (Orifice Controls 1.84 cfs @ 3.37 fps)

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

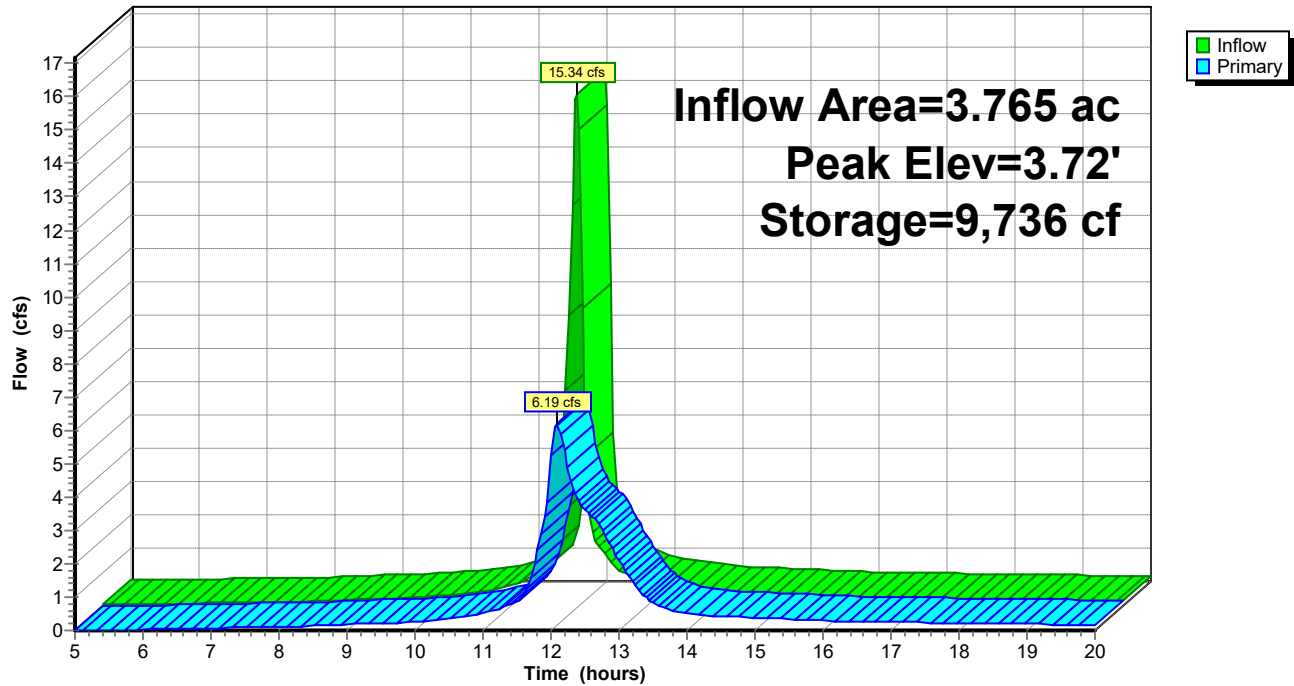
Type II 24-hr 10 Year Rainfall=3.20"

Printed 5/30/2024

Page 16

Pond 4P:

Hydrograph



Prelim Pond*Type II 24-hr 25 Year Rainfall=3.70"*

Prepared by HP Inc.

Printed 5/30/2024

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Page 17

Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points

Runoff by SCS TR-20 method, UH=SCS

Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1: OnsiteRunoff Area=164,014 sf 0.00% Impervious Runoff Depth>1.37"
Flow Length=311' Tc=21.2 min CN=76 Runoff=5.98 cfs 0.430 af**Subcatchment 2: Offsite**Runoff Area=30,329 sf 100.00% Impervious Runoff Depth>3.20"
Tc=6.0 min CN=98 Runoff=3.55 cfs 0.186 af**Subcatchment 3:**Runoff Area=164,014 sf 86.14% Impervious Runoff Depth>2.84"
Tc=6.0 min CN=94 Runoff=18.07 cfs 0.891 af**Pond 4P:**Peak Elev=4.20' Storage=11,563 cf Inflow=18.07 cfs 0.891 af
Outflow=7.22 cfs 0.882 af**Total Runoff Area = 8.227 ac Runoff Volume = 1.507 af Average Runoff Depth = 2.20"**
52.11% Pervious = 4.287 ac 47.89% Impervious = 3.940 ac

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 25 Year Rainfall=3.70"

Printed 5/30/2024

Page 18

Summary for Subcatchment 1: Onsite

Runoff = 5.98 cfs @ 12.15 hrs, Volume= 0.430 af, Depth> 1.37"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

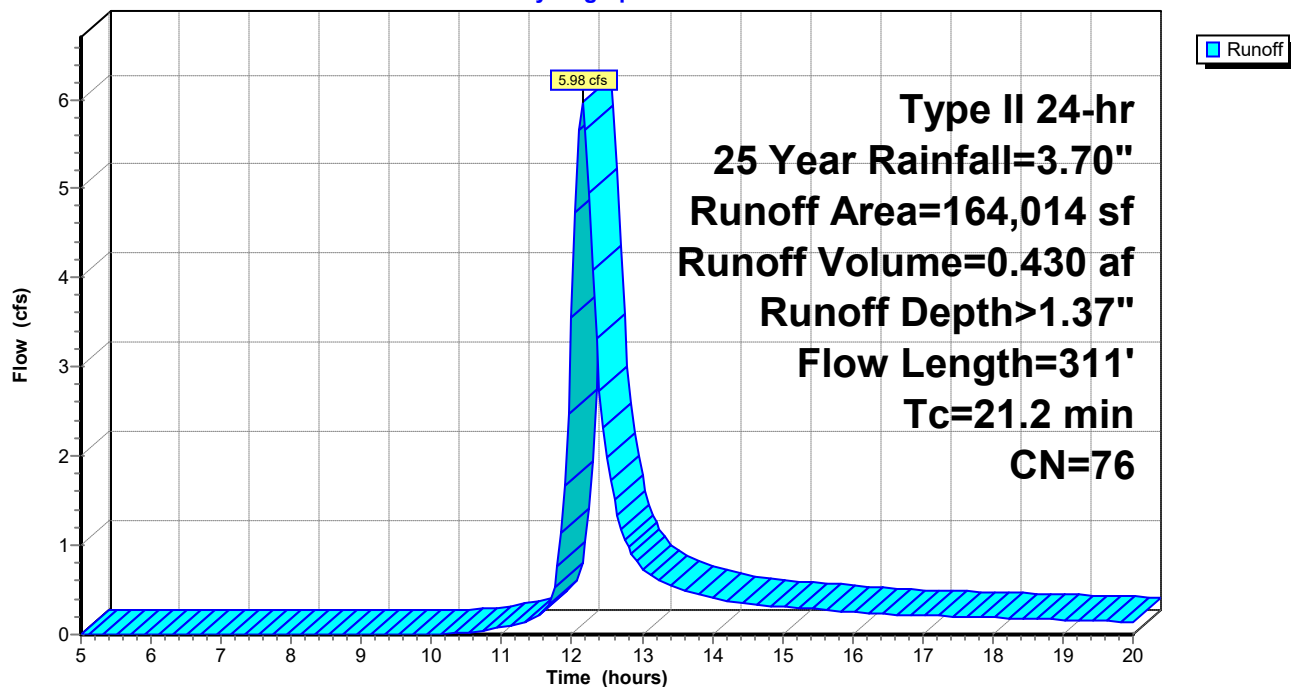
Type II 24-hr 25 Year Rainfall=3.70"

Area (sf)	CN	Description
* 164,014	76	Onsite Pre-developed SG C / Forested
164,014		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
21.1	300	0.0400	0.24		Sheet Flow, Grass: Short n= 0.150 P2= 2.30"
0.1	11	0.0660	1.80		Shallow Concentrated Flow, Short Grass Pasture Kv= 7.0 fps
21.2	311	Total			

Subcatchment 1: Onsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 25 Year Rainfall=3.70"

Printed 5/30/2024

Page 19

Summary for Subcatchment 2: Offsite

Runoff = 3.55 cfs @ 11.96 hrs, Volume= 0.186 af, Depth> 3.20"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

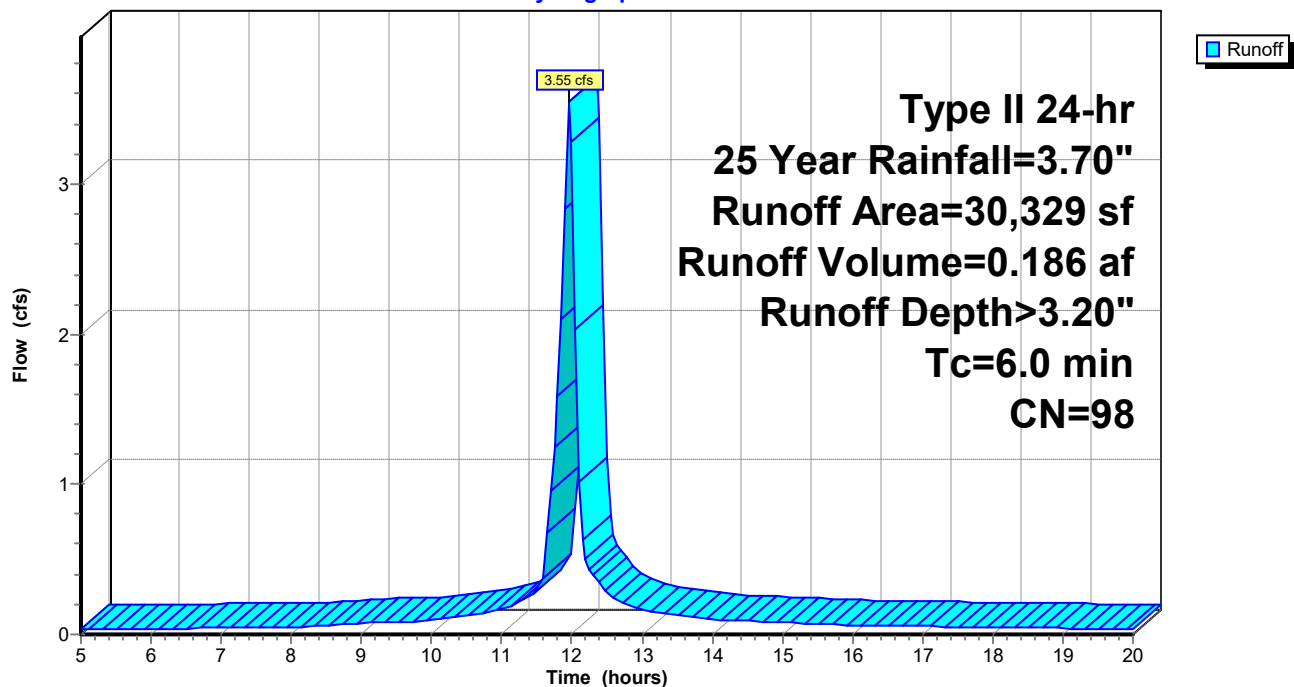
Type II 24-hr 25 Year Rainfall=3.70"

	Area (sf)	CN	Description
*	30,329	98	OFFSITE Roadway
	30,329		100.00% Impervious Area

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

Subcatchment 2: Offsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 25 Year Rainfall=3.70"

Printed 5/30/2024

Page 20

Summary for Subcatchment 3:

Runoff = 18.07 cfs @ 11.96 hrs, Volume= 0.891 af, Depth> 2.84"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

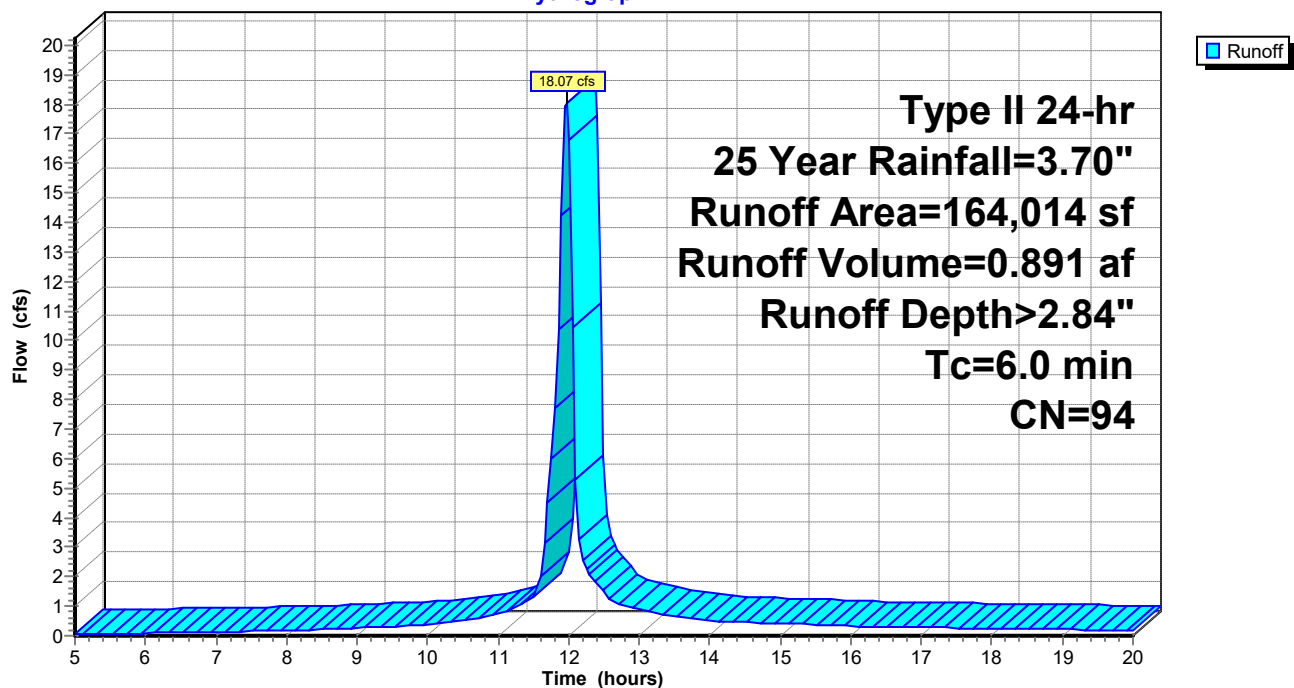
Type II 24-hr 25 Year Rainfall=3.70"

	Area (sf)	CN	Description
*	141,284	98	Impervious
*	22,730	69	Landscape
	164,014	94	Weighted Average
	22,730		13.86% Pervious Area
	141,284		86.14% Impervious Area

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

Subcatchment 3:

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 25 Year Rainfall=3.70"

Printed 5/30/2024

Page 21

Summary for Pond 4P:

[82] Warning: Early inflow requires earlier time span

Inflow Area = 3.765 ac, 86.14% Impervious, Inflow Depth > 2.84" for 25 Year event
Inflow = 18.07 cfs @ 11.96 hrs, Volume= 0.891 af
Outflow = 7.22 cfs @ 12.08 hrs, Volume= 0.882 af, Atten= 60%, Lag= 7.3 min
Primary = 7.22 cfs @ 12.08 hrs, Volume= 0.882 af

Routing by Stor-Ind method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

Peak Elev= 4.20' @ 12.08 hrs Surf.Area= 0 sf Storage= 11,563 cf

Plug-Flow detention time= 25.8 min calculated for 0.881 af (99% of inflow)

Center-of-Mass det. time= 20.9 min (768.8 - 747.9)

Volume	Invert	Avail.Storage	Storage Description
#1	0.00'	26,908 cf	Custom Stage Data Listed below x 0.69

Elevation (feet)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
0.00	0	0
2.35	6,499	6,499
4.69	12,999	19,498
7.00	19,499	38,997

Device	Routing	Invert	Outlet Devices
#1	Primary	0.00'	9.5" Vert. Orifice/Grate C= 0.600
#2	Primary	2.80'	10.0" Vert. Orifice/Grate C= 0.600

Primary OutFlow Max=7.19 cfs @ 12.08 hrs HW=4.18' (Free Discharge)

↑ **1=Orifice/Grate** (Orifice Controls 4.61 cfs @ 9.37 fps)

└ **2=Orifice/Grate** (Orifice Controls 2.58 cfs @ 4.72 fps)

Prelim Pond

Prepared by HP Inc.

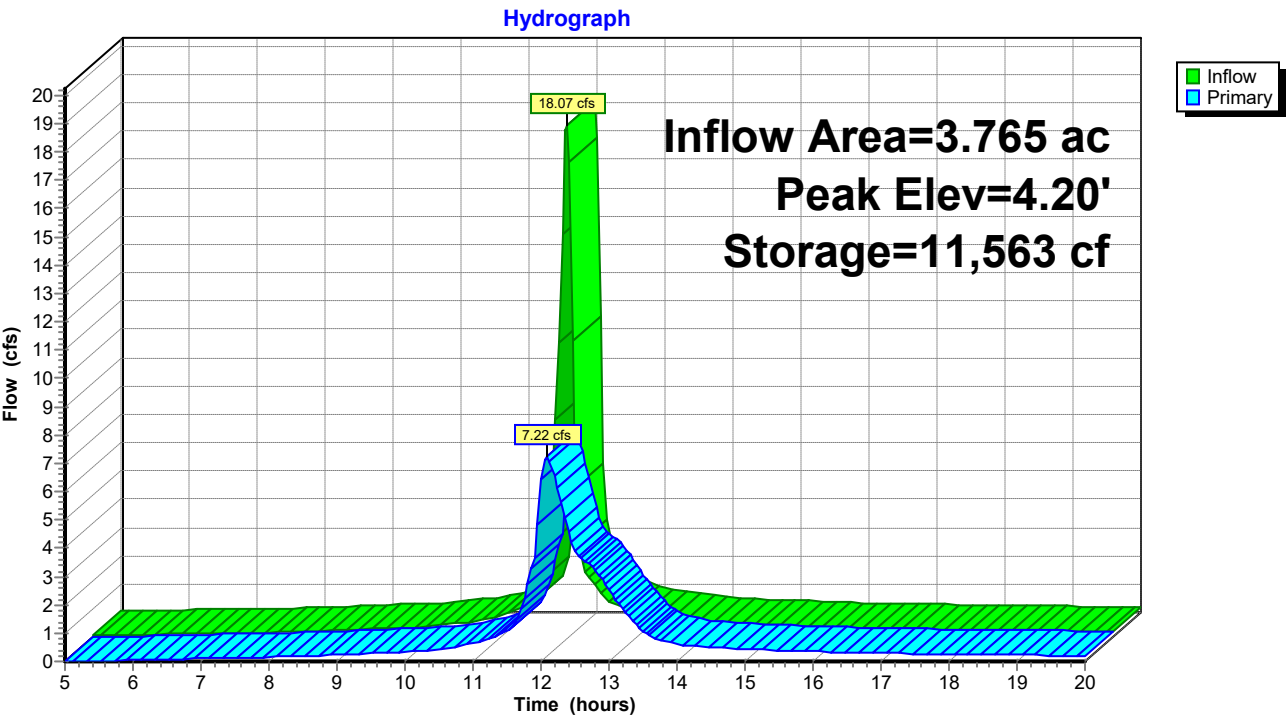
HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 25 Year Rainfall=3.70"

Printed 5/30/2024

Page 22

Pond 4P:



Prelim Pond*Type II 24-hr 100 Year Rainfall=4.10"*

Prepared by HP Inc.

Printed 5/30/2024

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Page 23

Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points

Runoff by SCS TR-20 method, UH=SCS

Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1: OnsiteRunoff Area=164,014 sf 0.00% Impervious Runoff Depth>1.65"
Flow Length=311' Tc=21.2 min CN=76 Runoff=7.24 cfs 0.518 af**Subcatchment 2: Offsite**Runoff Area=30,329 sf 100.00% Impervious Runoff Depth>3.56"
Tc=6.0 min CN=98 Runoff=3.94 cfs 0.206 af**Subcatchment 3:**Runoff Area=164,014 sf 86.14% Impervious Runoff Depth>3.21"
Tc=6.0 min CN=94 Runoff=20.24 cfs 1.006 af**Pond 4P:**Peak Elev=4.59' Storage=13,073 cf Inflow=20.24 cfs 1.006 af
Outflow=7.93 cfs 0.995 af**Total Runoff Area = 8.227 ac Runoff Volume = 1.731 af Average Runoff Depth = 2.52"**
52.11% Pervious = 4.287 ac 47.89% Impervious = 3.940 ac

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 100 Year Rainfall=4.10"

Printed 5/30/2024

Page 24

Summary for Subcatchment 1: Onsite

Runoff = 7.24 cfs @ 12.15 hrs, Volume= 0.518 af, Depth> 1.65"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

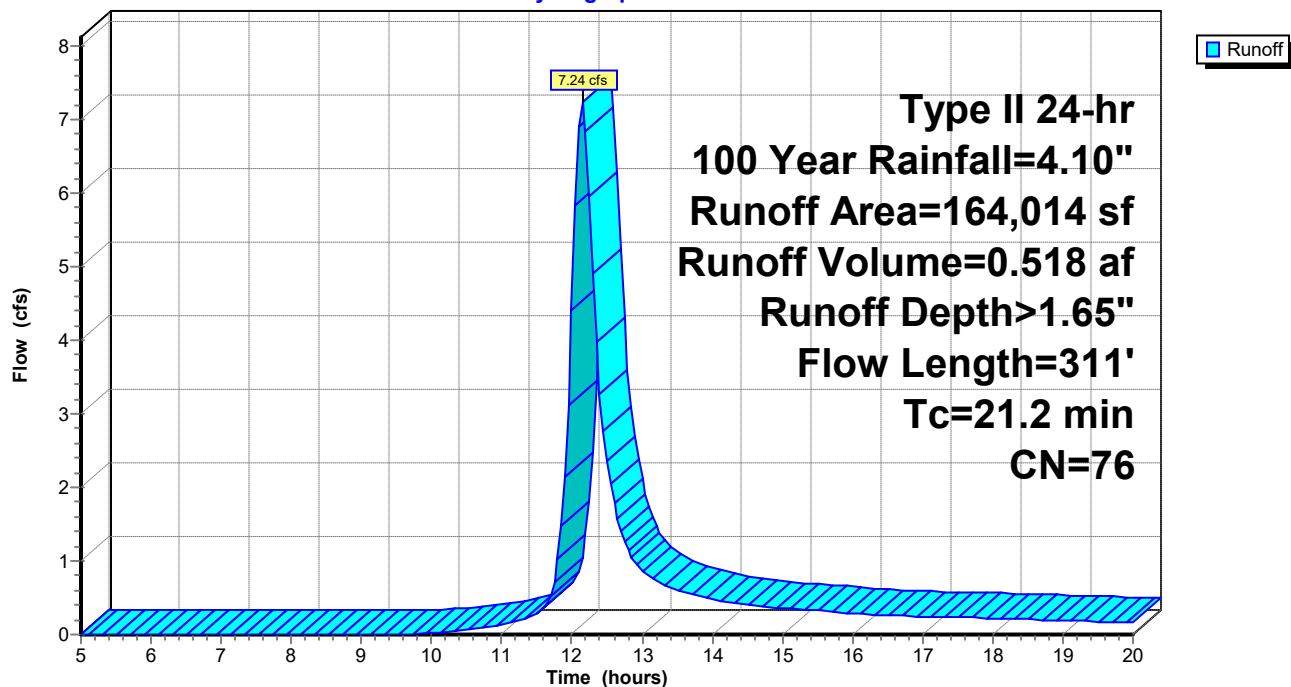
Type II 24-hr 100 Year Rainfall=4.10"

Area (sf)	CN	Description
* 164,014	76	Onsite Pre-developed SG C / Forested
164,014		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
21.1	300	0.0400	0.24		Sheet Flow, Grass: Short n= 0.150 P2= 2.30"
0.1	11	0.0660	1.80		Shallow Concentrated Flow, Short Grass Pasture Kv= 7.0 fps
21.2	311	Total			

Subcatchment 1: Onsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 100 Year Rainfall=4.10"

Printed 5/30/2024

Page 25

Summary for Subcatchment 2: Offsite

Runoff = 3.94 cfs @ 11.96 hrs, Volume= 0.206 af, Depth> 3.56"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

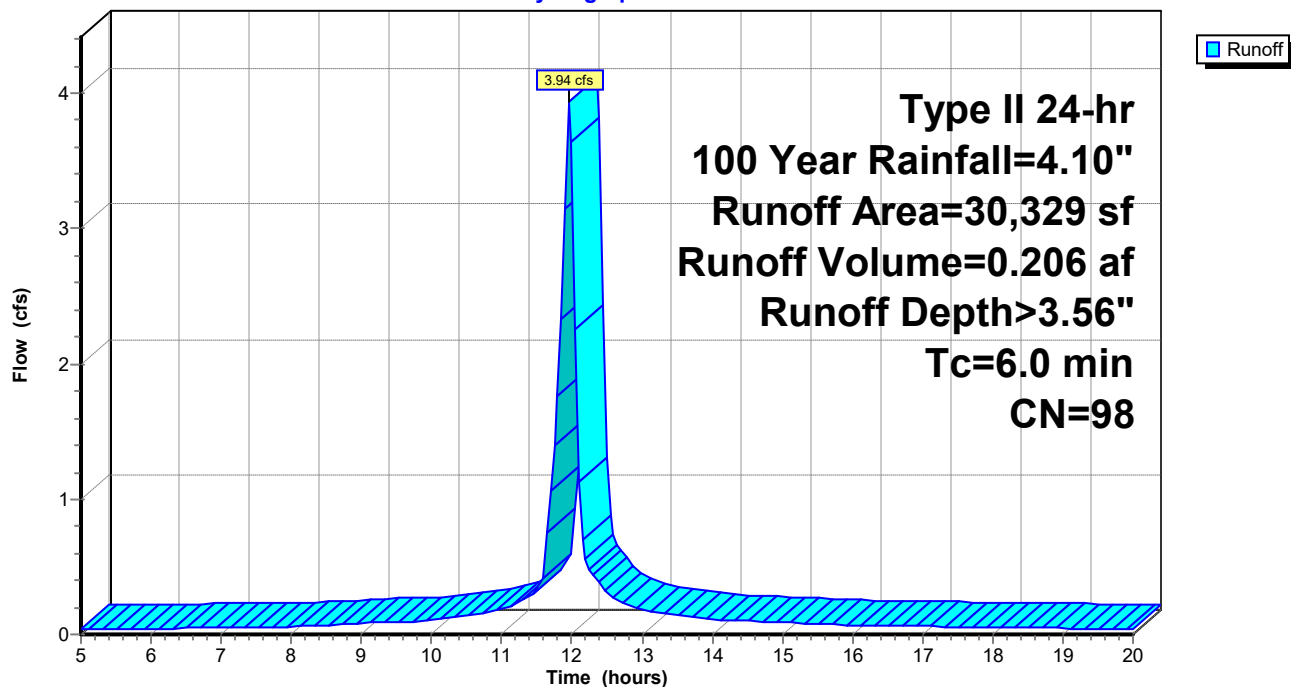
Type II 24-hr 100 Year Rainfall=4.10"

	Area (sf)	CN	Description
*	30,329	98	OFFSITE Roadway
	30,329		100.00% Impervious Area

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

Subcatchment 2: Offsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 100 Year Rainfall=4.10"

Printed 5/30/2024

Page 26

Summary for Subcatchment 3:

Runoff = 20.24 cfs @ 11.96 hrs, Volume= 1.006 af, Depth> 3.21"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

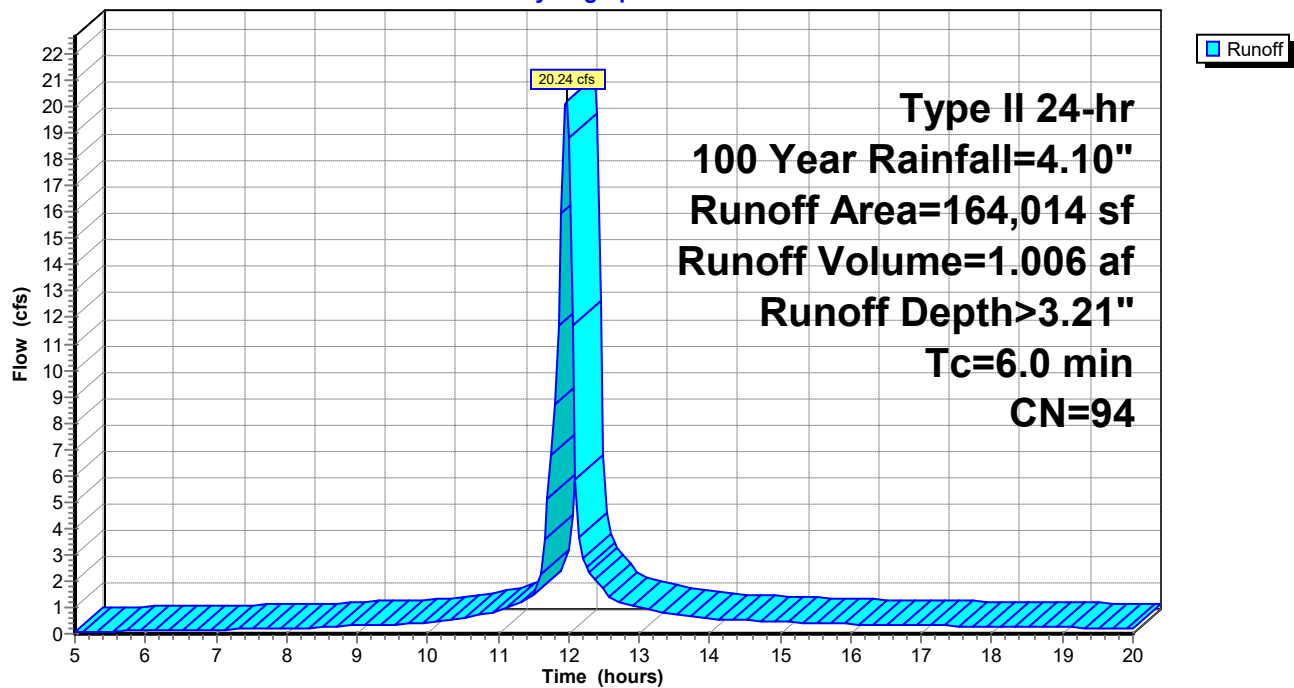
Type II 24-hr 100 Year Rainfall=4.10"

	Area (sf)	CN	Description
*	141,284	98	Impervious
*	22,730	69	Landscape
	164,014	94	Weighted Average
	22,730		13.86% Pervious Area
	141,284		86.14% Impervious Area

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

Subcatchment 3:

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr 100 Year Rainfall=4.10"

Printed 5/30/2024

Page 27

Summary for Pond 4P:

[82] Warning: Early inflow requires earlier time span

Inflow Area = 3.765 ac, 86.14% Impervious, Inflow Depth > 3.21" for 100 Year event
Inflow = 20.24 cfs @ 11.96 hrs, Volume= 1.006 af
Outflow = 7.93 cfs @ 12.09 hrs, Volume= 0.995 af, Atten= 61%, Lag= 7.4 min
Primary = 7.93 cfs @ 12.09 hrs, Volume= 0.995 af

Routing by Stor-Ind method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

Peak Elev= 4.59' @ 12.09 hrs Surf.Area= 0 sf Storage= 13,073 cf

Plug-Flow detention time= 25.6 min calculated for 0.995 af (99% of inflow)

Center-of-Mass det. time= 20.9 min (766.7 - 745.8)

Volume	Invert	Avail.Storage	Storage Description
#1	0.00'	26,908 cf	Custom Stage Data Listed below x 0.69

Elevation (feet)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
0.00	0	0
2.35	6,499	6,499
4.69	12,999	19,498
7.00	19,499	38,997

Device	Routing	Invert	Outlet Devices
#1	Primary	0.00'	9.5" Vert. Orifice/Grate C= 0.600
#2	Primary	2.80'	10.0" Vert. Orifice/Grate C= 0.600

Primary OutFlow Max=7.90 cfs @ 12.09 hrs HW=4.57' (Free Discharge)

↑ **1=Orifice/Grate** (Orifice Controls 4.84 cfs @ 9.84 fps)

— **2=Orifice/Grate** (Orifice Controls 3.06 cfs @ 5.61 fps)

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

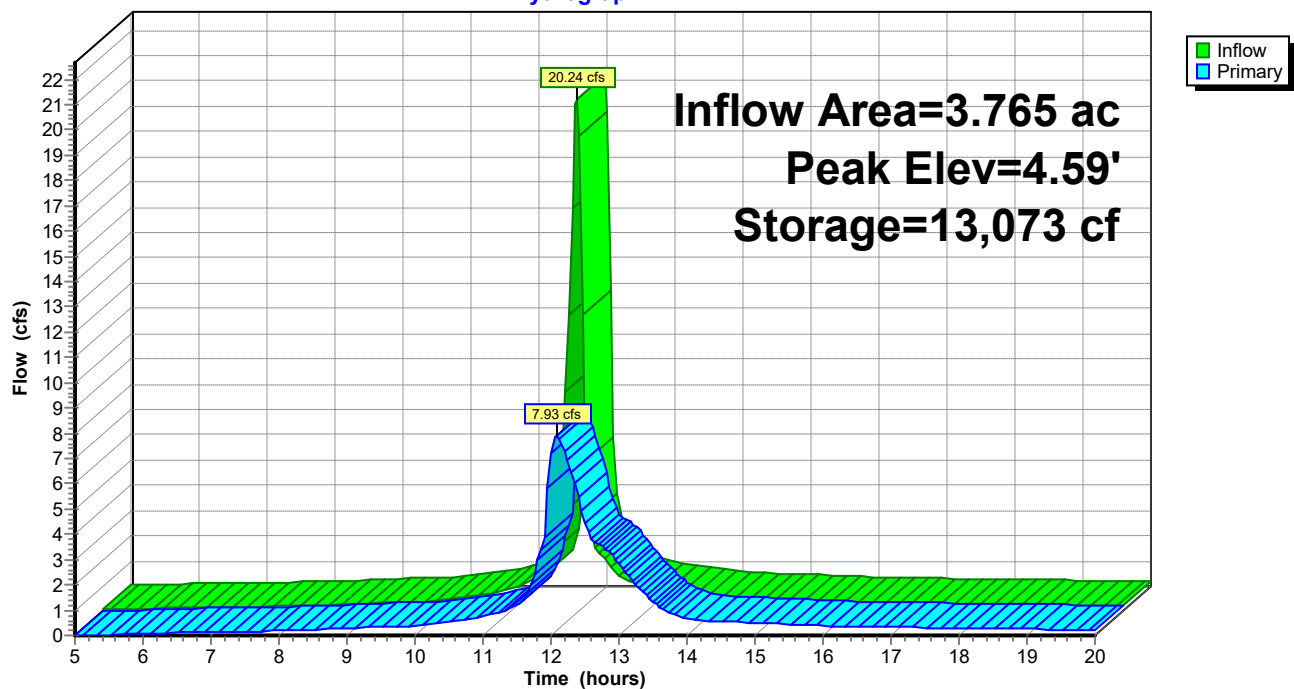
Type II 24-hr 100 Year Rainfall=4.10"

Printed 5/30/2024

Page 28

Pond 4P:

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr WQ Rainfall=1.54"

Printed 5/30/2024

Page 29

Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points

Runoff by SCS TR-20 method, UH=SCS

Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1: Onsite

Runoff Area=164,014 sf 0.00% Impervious Runoff Depth>0.17"
Flow Length=311' Tc=21.2 min CN=76 Runoff=0.52 cfs 0.054 af

Subcatchment 2: Offsite

Runoff Area=30,329 sf 100.00% Impervious Runoff Depth>1.23"
Tc=6.0 min CN=98 Runoff=1.43 cfs 0.072 af

Subcatchment 3:

Runoff Area=164,014 sf 86.14% Impervious Runoff Depth>0.91"
Tc=6.0 min CN=94 Runoff=6.18 cfs 0.284 af

Pond 4P:

Peak Elev=1.83' Storage=3,492 cf Inflow=6.18 cfs 0.284 af
Outflow=2.84 cfs 0.278 af

Total Runoff Area = 8.227 ac Runoff Volume = 0.410 af Average Runoff Depth = 0.60"
52.11% Pervious = 4.287 ac 47.89% Impervious = 3.940 ac

Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr WQ Rainfall=1.54"

Printed 5/30/2024

Page 30

Summary for Subcatchment 1: Onsite

Runoff = 0.52 cfs @ 12.20 hrs, Volume= 0.054 af, Depth> 0.17"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

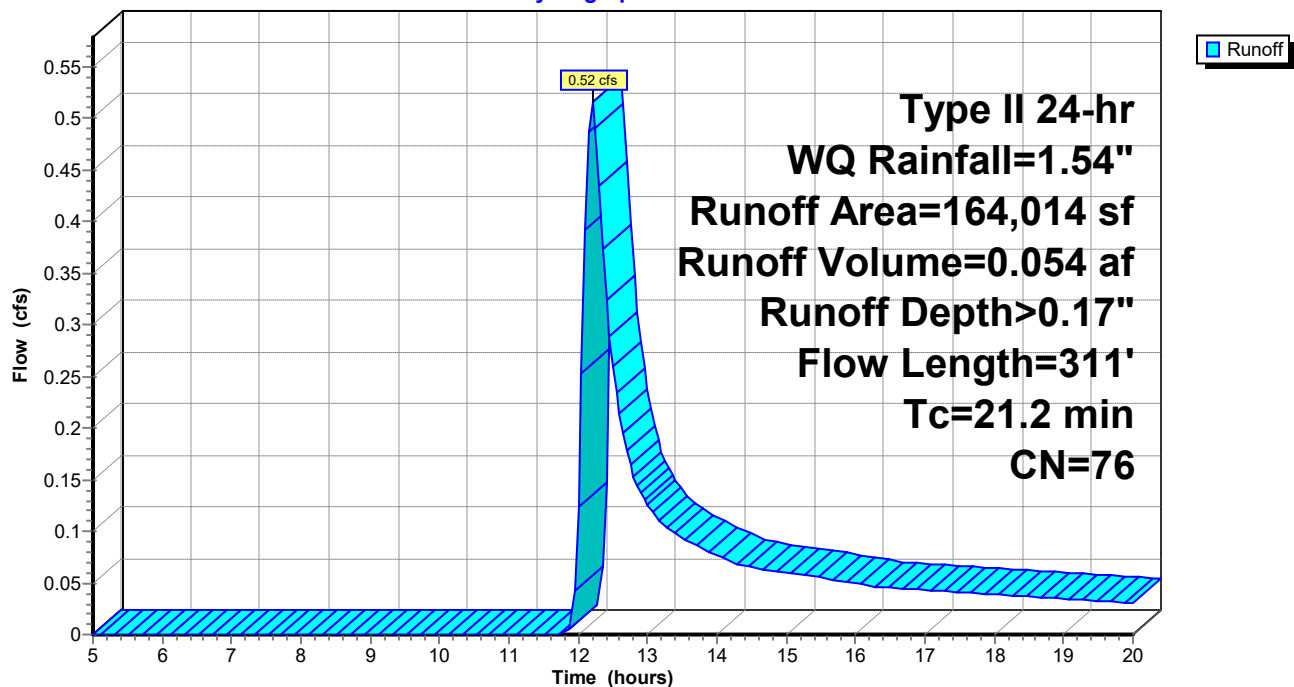
Type II 24-hr WQ Rainfall=1.54"

Area (sf)	CN	Description
* 164,014	76	Onsite Pre-developed SG C / Forested
164,014		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
21.1	300	0.0400	0.24		Sheet Flow, Grass: Short n= 0.150 P2= 2.30"
0.1	11	0.0660	1.80		Shallow Concentrated Flow, Short Grass Pasture Kv= 7.0 fps
21.2	311	Total			

Subcatchment 1: Onsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr WQ Rainfall=1.54"

Printed 5/30/2024

Page 31

Summary for Subcatchment 2: Offsite

Runoff = 1.43 cfs @ 11.96 hrs, Volume= 0.072 af, Depth> 1.23"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

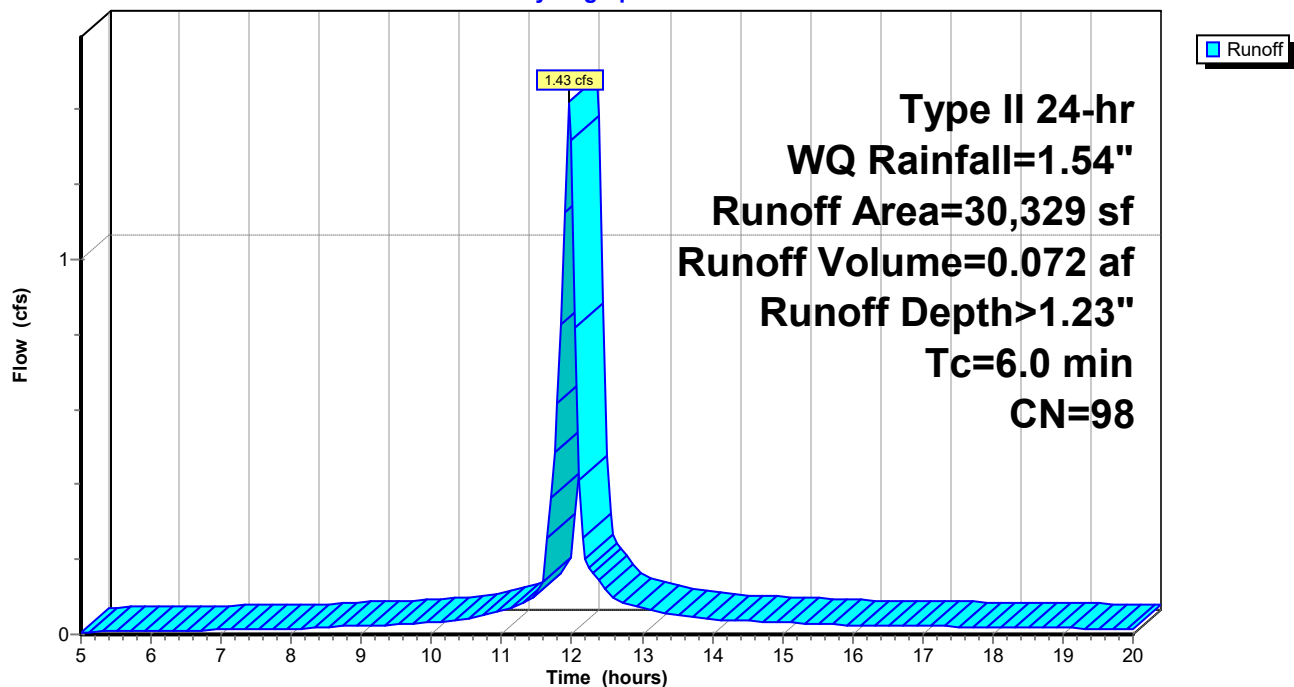
Type II 24-hr WQ Rainfall=1.54"

	Area (sf)	CN	Description
*	30,329	98	OFFSITE Roadway
	30,329		100.00% Impervious Area

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

Subcatchment 2: Offsite

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr WQ Rainfall=1.54"

Printed 5/30/2024

Page 32

Summary for Subcatchment 3:

Runoff = 6.18 cfs @ 11.97 hrs, Volume= 0.284 af, Depth> 0.91"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs

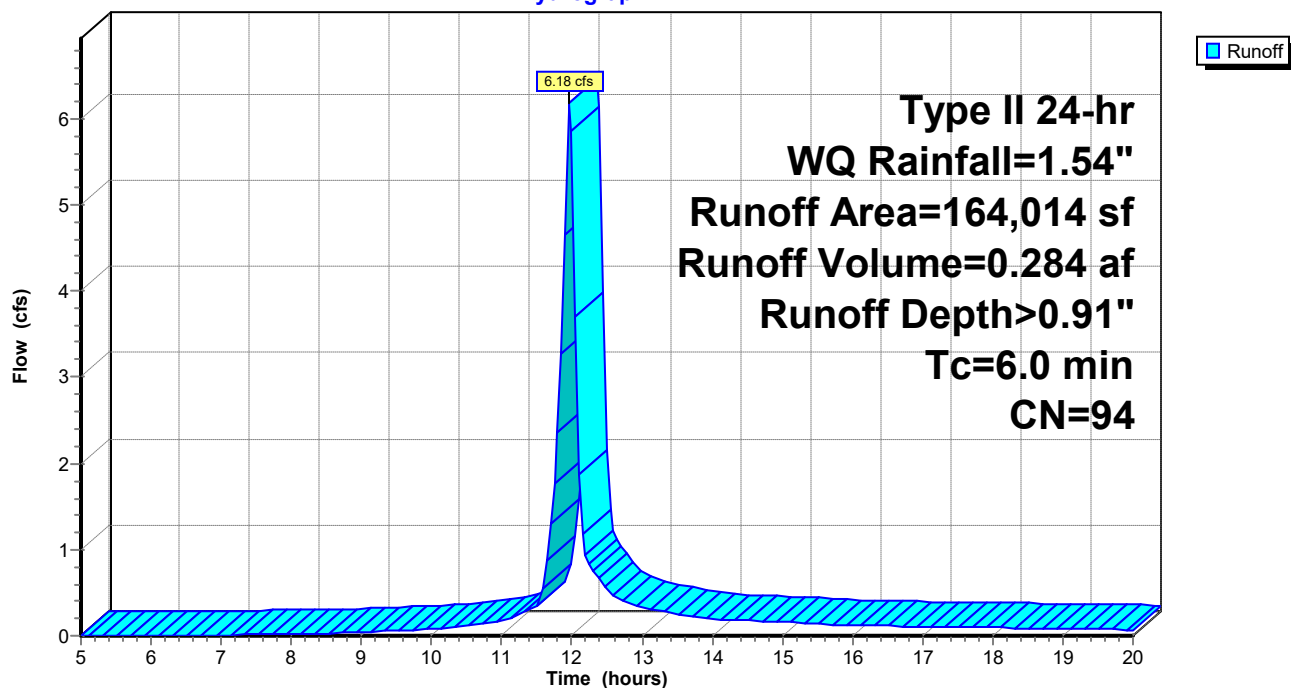
Type II 24-hr WQ Rainfall=1.54"

	Area (sf)	CN	Description
*	141,284	98	Impervious
*	22,730	69	Landscape
	164,014	94	Weighted Average
	22,730		13.86% Pervious Area
	141,284		86.14% Impervious Area

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

Subcatchment 3:

Hydrograph



Prelim Pond

Prepared by HP Inc.

HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr WQ Rainfall=1.54"

Printed 5/30/2024

Page 33

Summary for Pond 4P:

Inflow Area = 3.765 ac, 86.14% Impervious, Inflow Depth > 0.91" for WQ event
Inflow = 6.18 cfs @ 11.97 hrs, Volume= 0.284 af
Outflow = 2.84 cfs @ 12.07 hrs, Volume= 0.278 af, Atten= 54%, Lag= 6.4 min
Primary = 2.84 cfs @ 12.07 hrs, Volume= 0.278 af

Routing by Stor-Ind method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Peak Elev= 1.83' @ 12.07 hrs Surf.Area= 0 sf Storage= 3,492 cf

Plug-Flow detention time= 27.7 min calculated for 0.277 af (98% of inflow)
Center-of-Mass det. time= 19.1 min (791.9 - 772.7)

Volume	Invert	Avail.Storage	Storage Description
#1	0.00'	26,908 cf	Custom Stage Data Listed below x 0.69

Elevation (feet)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
0.00	0	0
2.35	6,499	6,499
4.69	12,999	19,498
7.00	19,499	38,997

Device	Routing	Invert	Outlet Devices
#1	Primary	0.00'	9.5" Vert. Orifice/Grate C= 0.600
#2	Primary	2.80'	10.0" Vert. Orifice/Grate C= 0.600

Primary OutFlow Max=2.81 cfs @ 12.07 hrs HW=1.81' (Free Discharge)

↑ **1=Orifice/Grate** (Orifice Controls 2.81 cfs @ 5.72 fps)

└ **2=Orifice/Grate** (Controls 0.00 cfs)

Prelim Pond

Prepared by HP Inc.

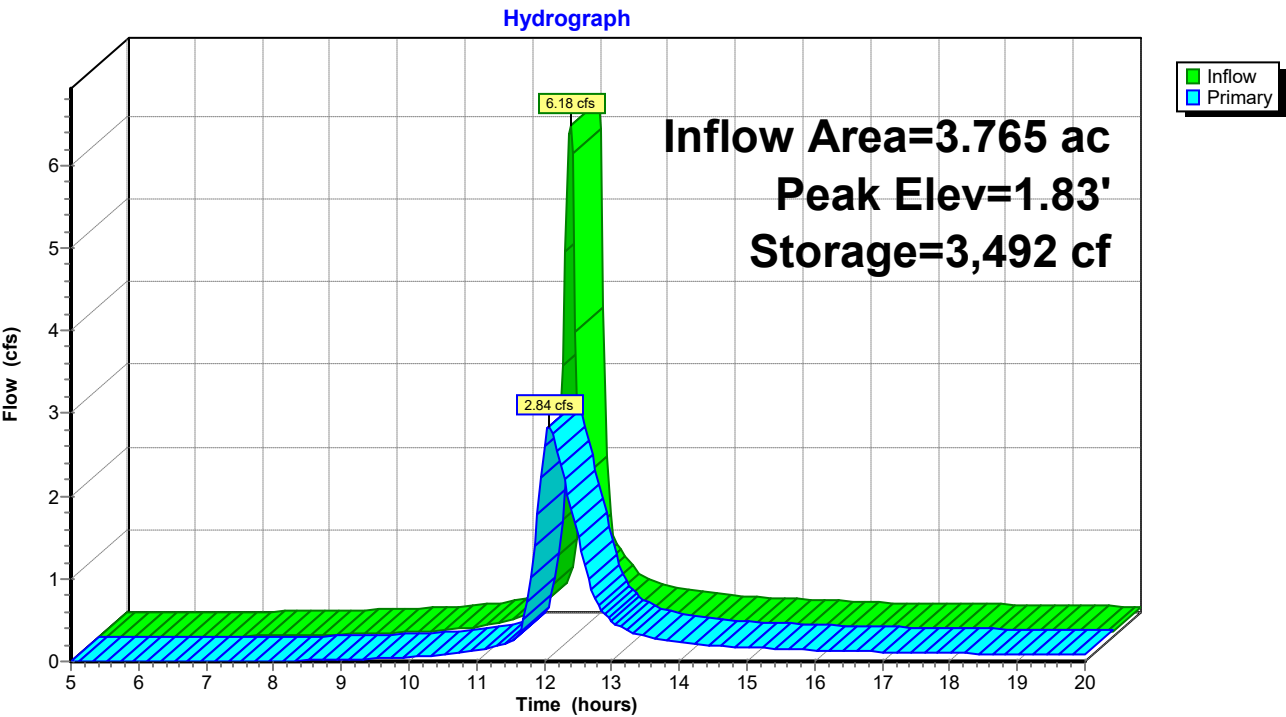
HydroCAD® 10.00 s/n 04953 © 2011 HydroCAD Software Solutions LLC

Type II 24-hr WQ Rainfall=1.54"

Printed 5/30/2024

Page 34

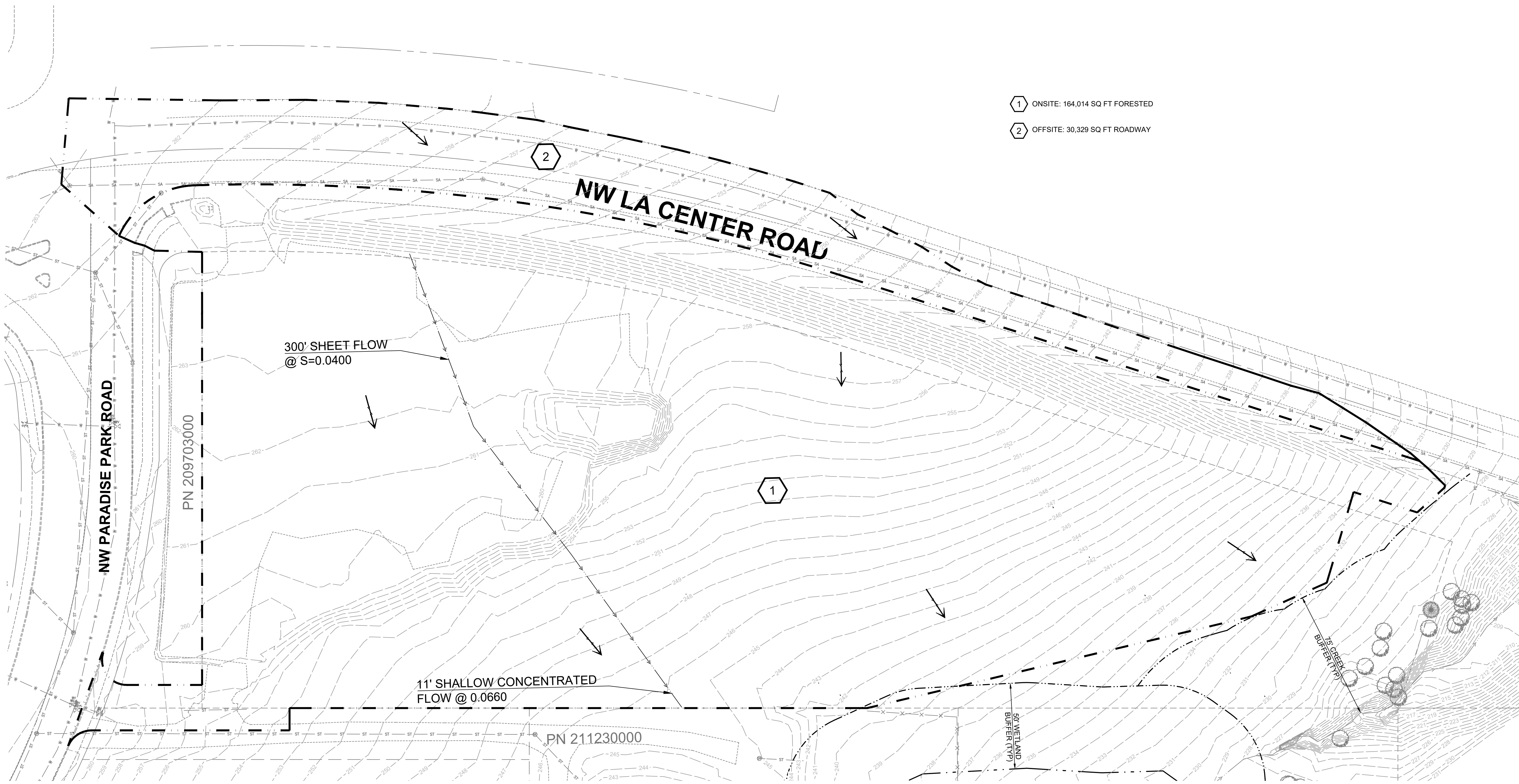
Pond 4P:



APPENDIX C

Basin Maps

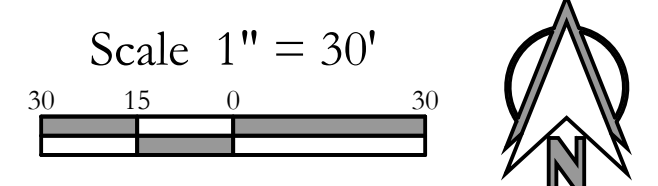
Pre-Developed
Post-Developed



- 1 ONSITE: 164,014 SQ FT FORESTED
- 2 OFFSITE: 30,329 SQ FT ROADWAY

Drainage Basin Legend
Subbasin Divide
Drainage Way

Hydrologic Modeling Reference Symbols
Flow Path
Subcatchment Area Id
Reach Id
Ponding Area Id



Pre-Development Basin Map For:

Paradise Park Binding Site Plan

A Site Located In The City Of La Center, Washington

Engineering - Surveying - Planning | 604 W. Evergreen Blvd., Vancouver, WA 98660 | PH (360) 944-6519 | Fax (360) 944-6539

PLS ENGINEERING

Revisions					
1	2	3	4	5	6

Project No. 3564

SCALE: H: 1" = 30'
V: N/A

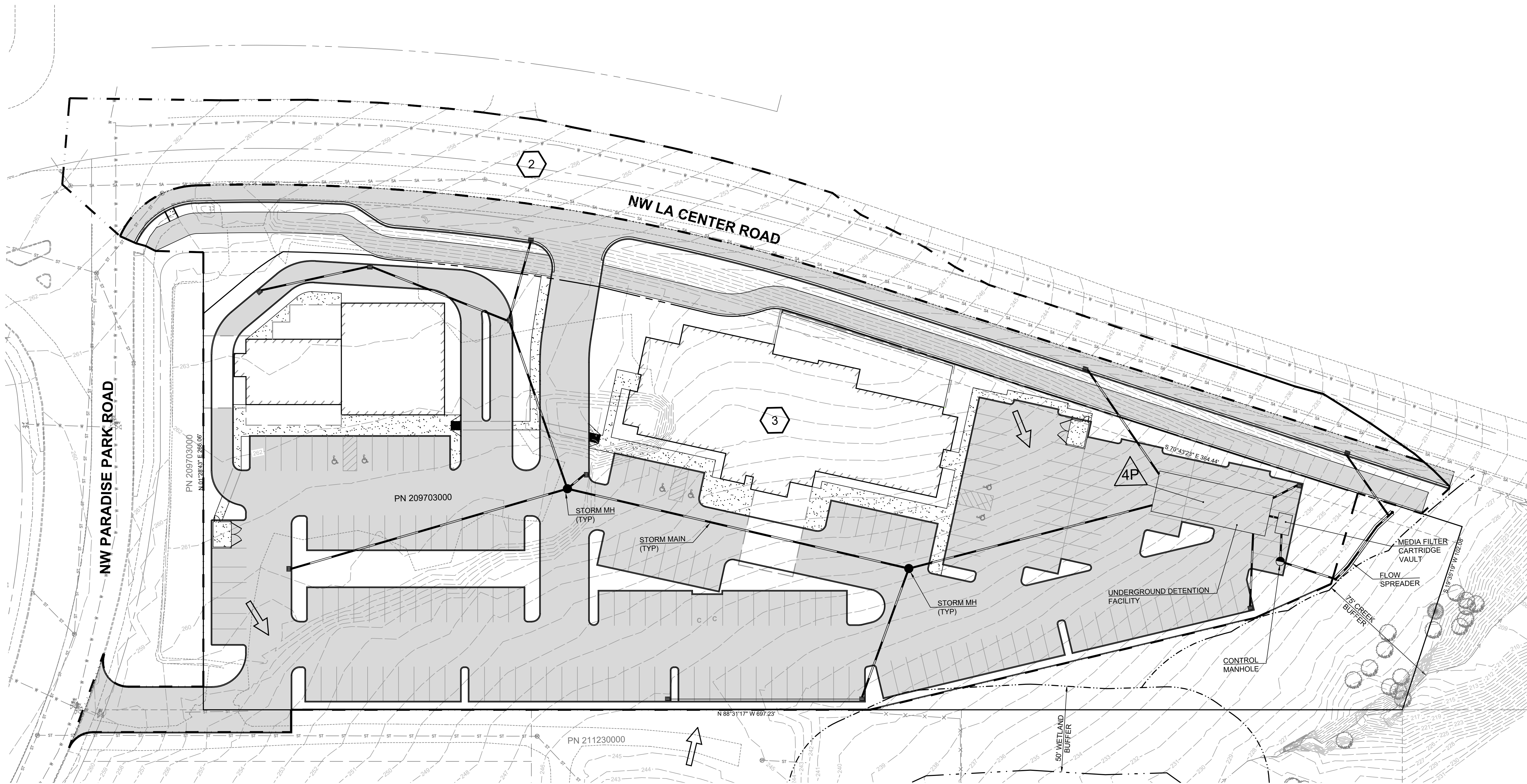
DESIGNED BY: THJ

DRAFTED BY: MM

REVIEWED BY: TGJ

01

02



Drainage Basin Legend

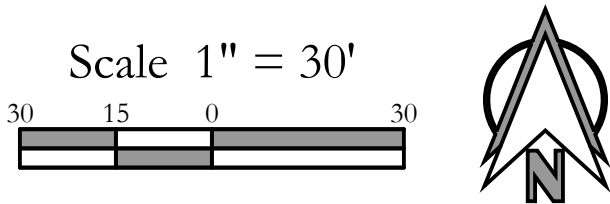
Subbasin Divide

Drainage Way

Hydrologic Modeling Reference Symbols

Subcatchment Area Id

Ponding Area Id



Revisions					
1	2	3	4	5	6

Project No. 3564	
SCALE:	H: 1" = 30' V: N/A
DESIGNED BY:	THJ
DRAFTED BY:	MM
REVIEWED BY:	TGJ

APPENDIX D

Geotechnical Report



REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation and Consultation Services

Proposed La Center Hotel and Commercial Development Project

Parcel #209708000

NW La Center Road

La Center (Clark County), Washington

for

Timberland, Inc.

**Project No. 1171.009.G
April 16, 2024**



REDMOND GEOTECHNICAL SERVICES

April 16, 2024

Mr. Luke Sasse
Timberland, Inc.
9321 NE 72nd Avenue, Building C#7
Vancouver, Washington 98665

Dear Mr. Sasse:

**Re: Geotechnical Investigation and Consultation Services,
Proposed La Center Hotel and Commercial Development Site, Parcel #209708000,
NW La Center Road, La Center (Clark County), Washington**

Submitted herewith is our report entitled "Geotechnical Investigation and Consultation Services, Proposed La Center Hotel and Commercial Development Site, Parcel #209708000, NW La Center Road, La Center (Clark County), Washington". The scope of our services was outlined in our formal proposal to Mr. Luke Sasse of Timberland, Inc dated January 22, 2024. Written authorization of our services was provided by Mr. Luke Sasse of Timberland, Inc on January 22, 2024.

During the course of our investigation, we have kept you and/or others advised of our schedule and preliminary findings. We appreciate the opportunity to assist you with this phase of the project. Should you have any questions regarding this report, please do not hesitate to call.

Sincerely,

Daniel M. Redmond, P.E., G.E.
President/Principal Engineer

Cc: Mr. Travis Johnson
PLS Engineering



EXPIRES 3-22-26

TABLE OF CONTENTS

	Page No.
INTRODUCTION	1
PROJECT DESCRIPTION	1
SCOPE OF WORK	2
SITE CONDITIONS	3
Site Geology	3
Surface Conditions	3
Subsurface Soil Conditions	4
Groundwater	5
Slope Hazards	5
INFILTRATION TESTING	6
LABORATORY TESTING	6
SEISMICITY AND EARTHQUAKE SOURCES	6
Liquefaction	7
Landslides	8
Surface Rupture	8
Tsunami and Seiche	8
Flooding and Erosion	8
CONCLUSIONS AND RECOMMENDATIONS	9
General	9
Site Preparation	10
Foundation Support	11
Shallow Foundations	11

Table of Contents (continued)

Floor Slab Support	12
Retaining/Below Grade Walls	12
Pavements	13
Automobile Parking and Access Drives	14
Pavement Subgrade, Base Course & Asphalt Materials	14
Wet Weather Grading and Soft Spot Mitigation	14
Shrink-Swell and Frost Heave	15
Excavations/Slopes	15
Surface Drainage/Groundwater	16
Design Infiltration Rates	16
Seismic Design Considerations	17
CONSTRUCTION MONITORING AND TESTING	17
CLOSURE AND LIMITATIONS	18
LEVEL OF CARE	18
REFERENCES	19
ATTACHMENTS	
Figure No. 1 - Site Vicinity and Geologic Map	
Figure No. 2 - Proposed Site Development Plan	
Figure No. 3 - Site Exploration Plan	
Figure No. 4 - Typical Key and Bench Fill Slope Detail	
Figure No. 5 - Typical Perimeter Footing/Retaining Wall Drain Detail	
Figure No. 6 – Clark County GIS Contour Map	
Figure No. 7 – Clark County GIS Percent Slope Map	
Figure No. 8 – Clark County GIS Severe Erosion Hazard Map	
Figure No. 9 – Clark County GIS Steep Slope and Landslide Hazard Map	
Figure No. 10 – Clark County GIS NEHRP Site Classes Map	
Figure No. 11 – Clark County GIS Liquefaction Susceptibility Map	

Table of Contents (continued)

APPENDIX A

Test Boring/Test Pit Logs and Laboratory Data

**GEOTECHNICAL INVESTIGATION AND CONSULTATION SERVICES
PROPOSED LA CENTER HOTEL AND COMMERCIAL DEVELOPMENT SITE
PARCEL #209708000
NW LA CENTER ROAD
LA CENTER (CLARK COUNTY) WASHINGTON**

INTRODUCTION

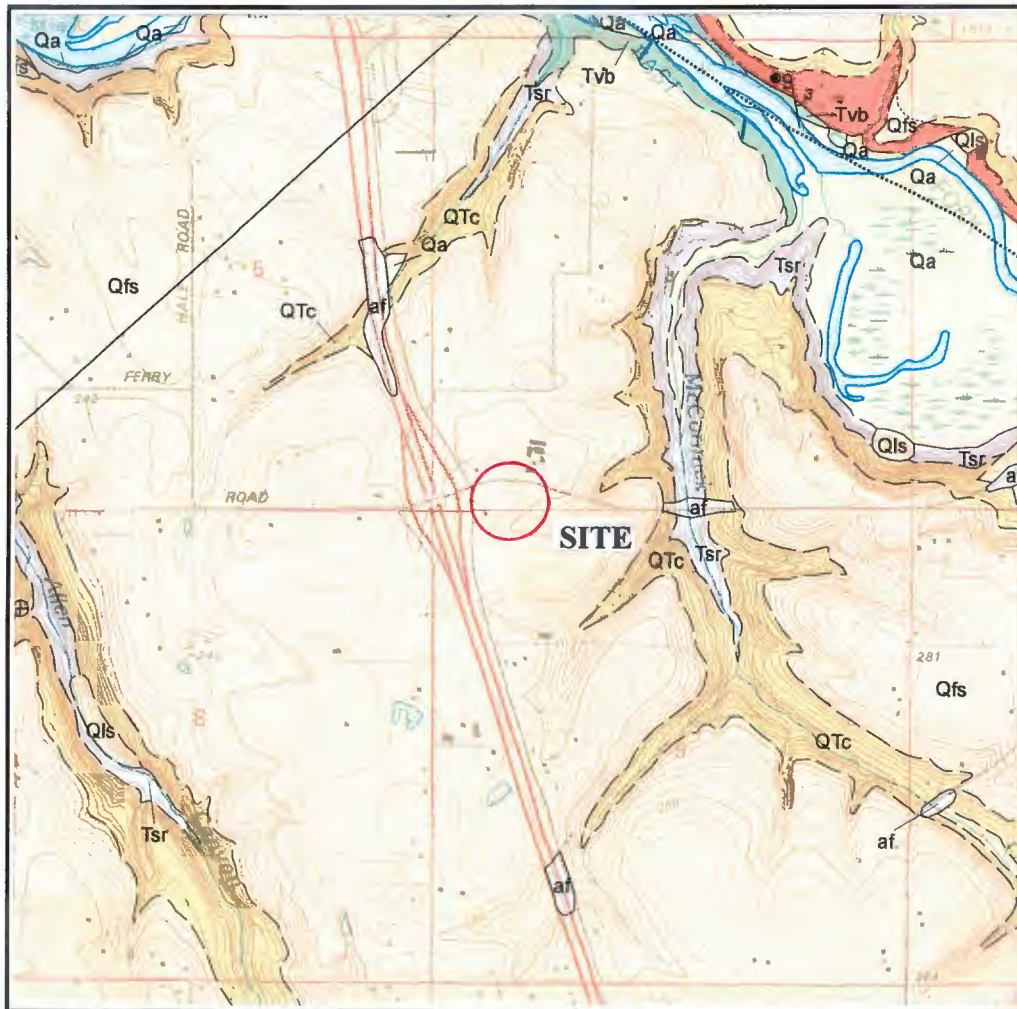
Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation and Consultation Services at the site of the proposed new La Center hotel and commercial development located to the south of NW La Center Road and to the east of NW Paradise Park Road in La Center (Clark County), Washington. The general location of the subject site is shown on the Site Vicinity and Geologic Map, Figure No. 1. The purpose of our geotechnical investigation and consultation services at this time was to explore the existing subsurface soils and/or groundwater conditions across the subject site and to evaluate any potential concerns with regard to development at the site as well as to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new La Center hotel and commercial development project.

PROJECT DESCRIPTION

Based on a review of the proposed site development plan, we understand that present plans for the project will consist of the construction of a new hotel as well as a new L-shaped commercial building which will include a new restaurant and drive through coffee business (see Proposed Site Development Plan, Figure No. 2). Reportedly, the new hotel will be a five-story structure with a base and/or ground floor footprint of approximately 10,000 square feet while the new restaurant and drive through coffee commercial building will be a single-story structure with a footprint of approximately 3,900 and 2,200 square feet, respectively. Additionally, we understand that the new hotel and commercial buildings will be wood-frame structures with concrete slab-on-grade floors.

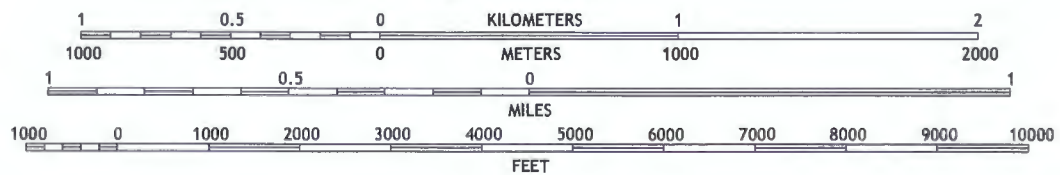
Structural loading information, although currently unavailable, is expected to result in maximum dead plus live continuous (strip) and individual (spread) column-type footings loads on the order of about 2.0 to 5.0 kips per lineal foot (klf) and 10 to 100 kips, respectively.

Although a specific site grading plan is not available at this time, we understand that both cuts and fills are presently planned for the project. In general, both cuts and/or fills of between five (5) to ten (10) feet are generally anticipated across the site. In this regard, due to the gently sloping nature of the existing and/or finish grade sloping site conditions, we generally do not anticipate that development of the site will require construction of a partial and/or below grade floor(s). However, the construction and/or use of retaining wall(s) is anticipated.



**RIDGEFIELD QUADRANGLE
WASHINGTON
7.5-MINUTE SERIES**

SCALE 1:24 000



CONTOUR INTERVAL 10 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

SITE VICINITY AND GEOLOGIC MAP

LA CENTER HOTEL SITE

#209708000, NW LA CENTER ROAD

Project No. 1171.009.G

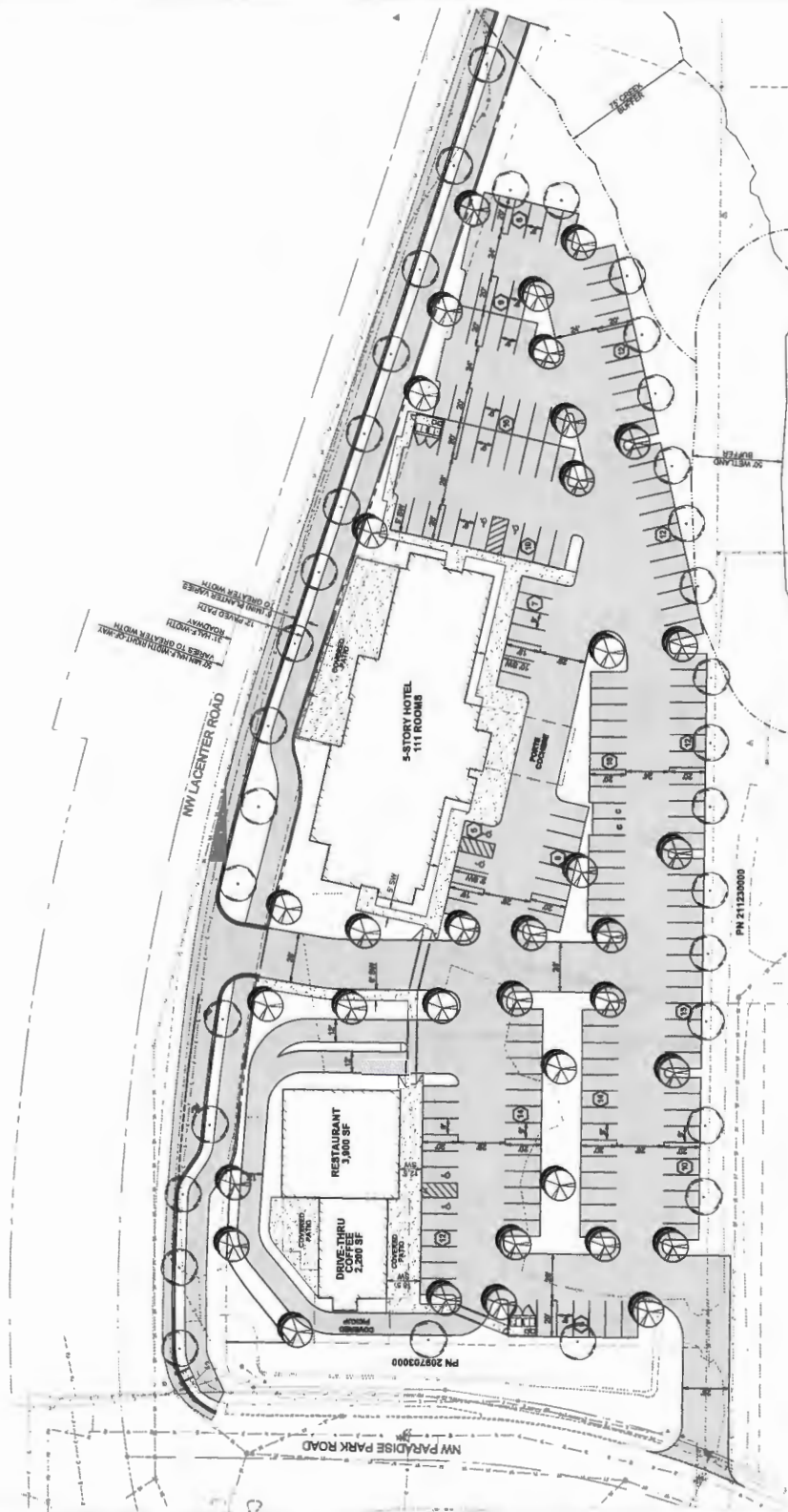
Figure No. 1

Other associated site improvements for the project will include construction of new paved access drives and parking areas. Additionally, the project will include the construction of new underground utility services as well as new concrete curbs and sidewalks. Further, we anticipate that development of the site may also include the collection of storm water for on-site treatment and possible on-site disposal.

SCOPE OF WORK

The subject site has reportedly been flagged by the City of La Center as a potential landslide hazard area. However, the "Relative Earthquake Hazard Map (NEHRP) of Ridgefield and/or Clark County, Washington" indicates that the subject property is located in Site Class C which is a low Relative Hazard. Additionally, the subject site is shown as having very low to low liquefaction susceptibility. In this regard, the purpose of our geotechnical studies was to evaluate the overall subsurface soil and/or groundwater conditions underlying the subject site with regard to the proposed new hotel and commercial development and construction at the site and any associated impacts or concerns with respect to development at the site as well as provide appropriate geotechnical design and construction recommendations for the project. Specifically, our geotechnical investigation included the following scope of work items:

1. Review of available and relevant geologic and/or geotechnical investigation reports for the subject site and/or area.
2. A detailed field reconnaissance and subsurface exploration program of the soil and ground water conditions underlying the site by means of seven (7) exploratory test pit excavations and three (3) drilled test borings. The exploratory test pits were excavated to depths ranging from about six (6) to seven (7) feet beneath existing site and/or surface grades while the test borings were drilled to depths of between twenty-one and one-half (21.5) to twenty-six and one-half (26.5) feet beneath the existing site and/or surface grades at the approximate locations as shown on the Site Exploration Plan, Figure No. 3. Additionally, field infiltration testing was also performed within various test pits excavated across the subject site.
3. Laboratory testing to evaluate and identify pertinent physical and engineering properties of the subsurface soils encountered relative to the planned site development and construction at the site. The laboratory testing program included tests to help evaluate the natural (field) moisture content and dry density, maximum dry density and optimum moisture content, Atterberg Limits and gradational characteristics, consolidation tests and (remolded) direct shear strength tests as well as "R"-value tests.
4. A literature review and engineering evaluation and assessment of the regional seismicity to evaluate the potential ground motion hazard(s) at the subject site. The evaluation and assessment included a review of the regional earthquake history and sources such as potential seismic sources, maximum credible earthquakes, and reoccurrence intervals as well as a discussion of the possible ground response to the selected design earthquake(s), fault rupture, landsliding, liquefaction, and tsunami and seiche flooding.



PROPOSED SITE DEVELOPMENT PLAN

LA CENTER HOTEL SITE

#209708000, NW LA CENTER ROAD

Project No. 1171.009.G

Figure No. 2

5. Engineering analyses utilizing the field and laboratory data as a basis for furnishing recommendations for foundation support of the proposed new apartment structures. Recommendations include maximum design allowable contact bearing pressure(s), depth of footing embedment, estimates of foundation settlement, lateral soil resistance, and foundation subgrade preparation. Additionally, construction and/or permanent subsurface water drainage considerations have also been prepared. Further, our report includes recommendations regarding site preparation, placement and compaction of structural fill materials, suitability of the on-site soils for use as structural fill, criteria for import fill materials, and preparation of foundation, pavement and/or floor slab subgrades.
6. Flexible pavement design and construction recommendations for the proposed new private access drives and parking area improvements.

SITE CONDITIONS

Site Geology

The subject site and/or area is underlain by cataclysmic-flood deposits and/or fine-grained facies (Qfs) of Pleistocene age (Geologic Map of the Ridgefield Quadrangle, Clark and Cowlitz Counties, Washington dated 2004). Characteristics include unconsolidated clay, silt and fine to medium sand; grain size decreases gradually to the north. Throughout the quadrangle mantel low-relief surfaces below about Elevation 300 feet but above Holocene floodplains. Rare fresh exposures reveal rhythmite beds several centimeters thick, each bed probably the product of a separate flood event (Waitt, 1980, 1985, 1994; Clague and others, 2003), but in most exposures deposit is oxidized light brown, and bedding is inconspicuous. Composed largely of quartz, feldspar, and conspicuous muscovite, which indicate deposition by the Columbia River rather than by local streams. Interpreted as slack-water deposits of large floods initiated by the failure of the ice dam at Glacial Lake Missoula in western Montana during the late Pleistocene (Bretz, 1925, 1959; Bretz and others, 1956, Waitt, 1980, 1985, 1994, 1996; Barker and Bunker, 1985; Atwater 1986; O'Connor and Baker, 1992; Benito and O'Connor, 2003).

Surface Conditions

The subject proposed new La Center hotel and commercial development property consists of one (1) generally irregular shaped tax lot (Parcel #209708000) which encompasses a total plan area of approximately 3.36 acres. The proposed hotel and commercial development property is roughly located to the south of NW La Center Road and to the east of NW Paradise Park Road.

The subject property is presently unimproved. However, the westerly portion of the site is currently being used to stockpile, crush and/or recycle asphalt and concrete rubble. Surface vegetation across the easterly portion of the site generally consists of a light to moderate growth of grass and weeds.

Topographically, the westerly/northwesterly portion of the site is characterized as relatively flat-lying terrain (i.e., less than 5 percent) descending downward towards the east/southeast while the central and easterly portions of the site are characterized by gently sloping terrain (i.e., 5 to 15 percent) descending downward towards the southeast with overall topographic relief estimated at about fifty-two (52) feet and ranges from a low about Elevation 211 feet near the southeasterly portion of the subject site to a high of about Elevation 263 near the northwesterly corner of the site. Additionally, the easterly southeasterly portion of the site is bounded by a seasonal drainage basis and/or stormwater facility associated with McCormick Creek. However, our site reconnaissance performed on February 6, 2024, did not reveal any evidence of existing and/or past landsliding in and/or near the lower easterly portion of the site.

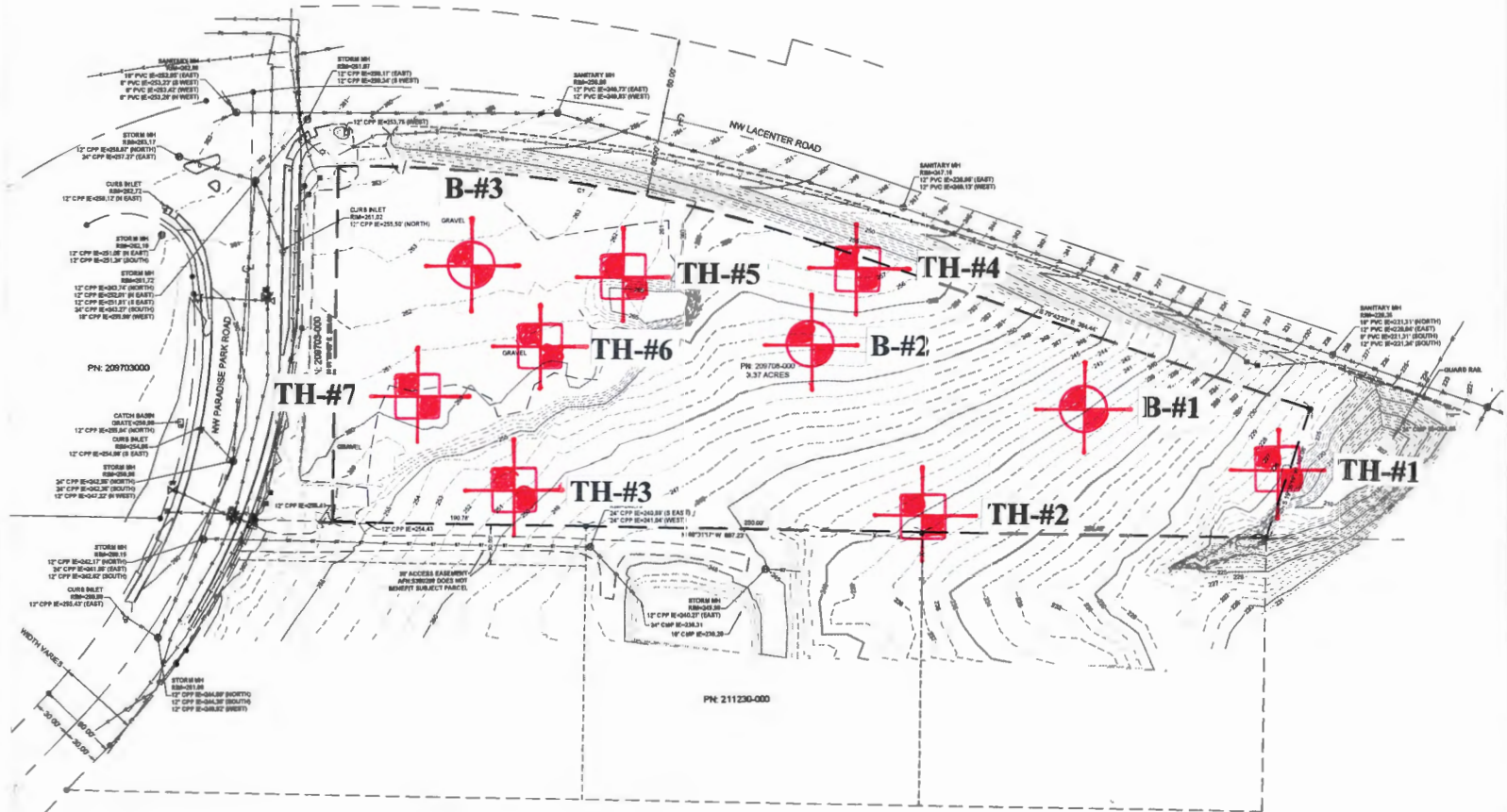
Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of seven (7) exploratory test pits excavated to depths ranging from about six (6) to seven (7) feet beneath existing site grades on February 6, 2024 with a track-mounted excavator and three (3) exploratory test borings drilled to depths ranging from twenty-one and one-half (21.5) to twenty-six and one-half (26.5) feet beneath the existing site and or surface grades on February 17, 2024 with tracked Geoprobe drilling equipment. The location of the exploratory test pits and test borings were located in the field by marking off distances from existing and/or known site features and are shown in relation to the existing site features and/or site improvements on the Site Exploration Plan, Figure No. 3. Detailed logs of the test borings and test pit explorations, presenting conditions encountered at each location explored, are presented in the Appendix, Figure No's. A-5 through A-11.

The exploratory test borings and test pit excavations were observed by staff from Redmond Geotechnical Services, LLC who logged each of the test boring and test pit explorations and obtained representative samples of the subsurface soils encountered across the site. Additionally, the elevation of the exploratory test borings and test pit excavations were referenced from an existing conditions site survey performed by PLS Engineering and should be considered as approximate. All subsurface soils encountered at the site and/or within the exploratory test borings and test pit excavations were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-4.

The test boring and test pit explorations revealed that the subject site is generally underlain by native soil deposits comprised of fine-grained facies and/or cataclysmic flood deposits (Qfs) of Pleistocene age. However, existing fill materials were also encountered across the upper westerly portion of the site. Specifically, the native soil deposits are composed of a surficial layer of dark brown, wet, soft to very soft, organic to highly organic, sandy, clayey silt topsoil materials to depths of about 12 to 14 inches. These surficial topsoil materials were in turn underlain by medium to gray-brown with gray mottling, very moist, medium stiff, clayey, sandy silt to the maximum depth explored of about twenty-six and one-half (26.5) feet beneath the existing site and/or surface grades. These clayey, sandy silt subgrade soils become stiff at a depth of about ten (10) feet and wet to saturated below a depth of about fifteen (15) to twenty (20) feet and are best characterized by relatively moderate strength and low to moderate compressibility.

SITE EXPLORATION PLAN



LEGEND



B-#1 Indicates approximate location of exploratory test boring

TH-#7 Indicates approximate location of exploratory test hole



Approximate Scale: 1" = 135'

In addition, fill soil materials were also encountered across the westerly portion of the site and consist of about one (1) foot of asphalt grindings over about two (2) to three (3) feet of a mixture of clayey, sandy silt to silty sand with organics and gravel as well as construction debris (i.e., brick fragments and concrete rubble). In general, the existing fill materials were found to be poorly to moderately compacted and are best characterized by relatively moderate to high compressibility and moderate to low strength.

Groundwater

Groundwater was encountered within two (2) of the exploratory test borings (B-#1 and B-#2) at the time of drilling at depths of about fifteen (15) to twenty (20) feet beneath existing surface grades except. Additionally, the easterly portion of the subject property is bounded by an existing seasonal drainage basin and/or stormwater facility associated with drainage to McCormick Creek..

In this regard, groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions and/or associated with runoff across the site as well as changes in site utilization. As such, we are generally of the opinion that the static water levels and/or surface water ponding observed and/or not observed during our recent field exploration work generally reflect a seasonal groundwater level at and/or beneath the site of at least ten (10) feet.

Slope Hazards

We evaluated potential slope hazards by examining historic aerial photographs of the subject site, reviewing available topographic and geologic maps, LiDAR imagery, and conducting a geologic reconnaissance of the subject property. During the reconnaissance, we observed existing roadways and/or structures for indications of slope movement. We also made a visual examination of the slopes and/or road cuts in the immediate area of the subject property.

There are no steep slopes (i.e., greater than 25 percent) at and/or on the subject property. The steepest existing slope gradient at the subject property is approximately 10 to 15 percent which is located within the central and/or easterly portion(s) of the subject site.

Our site reconnaissance of the subject property found it to be as shown on available maps, LiDAR and aerial photos - moderately easterly/southeasterly sloping terrain and showing no evidence of anomalous topography, nor any evidence of present and/or past slope instability. Given that the subject property as well as adjacent properties to the north, south, east and west have been developed for several years, one would expect to find fairly obvious evidence of existing slope movement and/or slope failures if the slopes at and/or near to the subject property are marginally stable. However, our review of LiDAR imagery and aerial photos found no evidence of slope instability at and/or on other undeveloped and/or developed properties located immediately adjacent to the subject property. Additionally, the existing seasonal drainage basin and/or stormwater facility located to the east of the subject property shows no evidence of landsliding and/or slope movement.

INFILTRATION TESTING

We performed one (1) field infiltration test at the site on February 6, 2024. The infiltration test was performed in test hole TH-#1 at depths of about four (4) feet beneath the existing site and/or surface grades. The subgrade soil encountered in the infiltration test hole consisted of clayey, sandy silt. The infiltration testing was performed in general conformance with current EPA and/or the Clark County Encased Falling Head test method which consisted of advancing a 6-inch diameter PVC pipe approximately 6 inches into the exposed soil horizon at the test location. Using a steady water flow, water was discharged into the pipe and allowed to penetrate and saturate the subgrade soils. The water level was adjusted over a two (2) hour period and allowed to achieve a saturated subgrade soil condition consistent with the bottom elevation of the surrounding test pit excavation. Following the required saturating period, water was again added into the PVC pipe and the time and/or rate at which the water level dropped was monitored and recorded. Each measurable drop in the water level was recorded until a consistent infiltration rate was observed and/or repeated.

Based on the results of the field infiltration testing at the site (see Field Infiltration Test Results, Figure No. A-17), we have found that the native clayey, sandy silt subgrade soil deposits possess an ultimate infiltration rate of about 1.4 inches per hour (in/hr).

LABORATORY TESTING

Representative samples of the on-site subsurface soils were collected at selected depths and intervals from various test pit excavations and returned to our laboratory for further examination and testing and/or to aid in the classification of the subsurface soils as well as to help evaluate and identify their engineering strength and compressibility characteristics. The laboratory testing consisted of visual and textural sample inspection, moisture content and dry density determinations, maximum dry density and optimum moisture content, Atterberg Limits and gradation analyses as well as consolidation, direct shear strength and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-12 through A-17.

SEISMICITY AND EARTHQUAKE SOURCES

The seismicity of the southwest Washington and northwest Oregon area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The CSZ is located offshore and extends from northern California to British Columbia. Within this zone, the oceanic Juan de Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers (km). The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Washington and Oregon coastlines.

Sequences of interlayered peat and sands have been interpreted to be the result of large Subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago. A study by Geomatrix (1995) and/or USGS (2008) suggests that the maximum earthquake associated with the CSZ is moment magnitude (Mw) 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes that have occurred within Subduction zones in other parts of the world. An Mw 9 earthquake would involve a rupture of the entire CSZ. As discussed by Geomatrix (1995) this has not occurred in other subduction zones that have exhibited much higher levels of historical seismicity than the CSZ. However, the 2008 USGS report has assigned a probability of 0.67 for a Mw 9 earthquake and a probability of 0.33 for a Mw 8.3 earthquake. For the purpose of this study an earthquake of Mw 9.0 was assumed to occur within the CSZ.

The intraplate zone encompasses the portion of the subducting Juan de Fuca Plate located at a depth of approximately 30 to 50 km below western Washington and western Oregon. Very low levels of seismicity have been observed within the intraplate zone in western Oregon and western Washington. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence were suggested in the Geomatrix (1995) study and include changes in the direction of Subduction between Oregon, Washington, and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia magnitude 7.1 and the 1965 Puget Sound magnitude 6.5 earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of magnitude 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The third source of seismicity that can result in ground shaking within the Vancouver and southwest Washington area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in this area is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0), Oregon earthquakes were crustal earthquakes.

Liquefaction

Seismic induced soil liquefaction is a phenomenon in which loose, granular soils and some silty soils, located below the water table, develop high pore water pressures and lose strength due to ground vibrations induced by earthquakes. Soil liquefaction can result in lateral flow of material into river channels, ground settlements and increased lateral and uplift pressures on underground structures. Buildings supported on soils that have liquefied often settle and tilt and may displace laterally. Soils located above the ground water table cannot liquefy, but granular soils located above the water table may settle during the earthquake shaking.

Our review of the subsurface soil test boring logs from our exploratory field explorations (B-#1 through B-#3) and laboratory test results indicate that the site is generally underlain by medium stiff to stiff, clayey, sandy silt soils to depths of at least twenty-six and one-half (26.5) feet beneath existing site grades. Additionally, groundwater was generally not encountered above a depth of about fifteen (15) feet at the time of our field work.

As such, due to the medium stiff to stiff and/or cohesive nature of the clayey, sandy silt subgrade soil deposits beneath the site, it is our opinion that the native clayey, sandy silt subgrade soil deposits located beneath the subject site have a very low potential for liquefaction during the design earthquake motions previously described.

Landslides

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, the subject property does not contain any steep slopes. As such, development of the subject site into the planned hotel and commercial development does not appear to present a potential geologic and/or landslide hazard provided that the site grading and development activities conform with the recommendations presented within this report.

Surface Rupture

Although the site is generally located within a region of the country known for seismic activity, no known faults exist on and/or immediately adjacent to the subject site. As such, the risk of surface rupture due to faulting is considered negligible.

Tsunami and Seiche

A tsunami, or seismic sea wave, is produced when a major fault under the ocean floor moves vertically and shifts the water column above it. A seiche is a periodic oscillation of a body of water resulting in changing water levels, sometimes caused by an earthquake. Tsunami and seiche are not considered a potential hazard at this site because the site is not near to the coast and/or there are no adjacent significant bodies of water.

Flooding and Erosion

Stream flooding is a potential hazard that should be considered in lowland areas of Clark County and La Center. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new hotel and commercial structures and the associated site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Clark County requirements for the 100-year flood levels of any nearby creeks, streams and/or drainage basins.

CONCLUSIONS AND RECOMMENDATIONS

General

Our review of available geologic maps, examination of historic aerial photographs and LiDAR imagery as well as our site reconnaissance found no evidence of landslides at and/or immediately adjacent to the subject property. As such, it appears that the potential for instability and/or landsliding at the subject property by the City of La Center is likely based on the presence of moderately steep slopes (i.e., 15 to 25 percent) located within the existing seasonal drainage basin and/or stormwater facility located adjacent to the easterly portion of the site.

In this regard, based on the results of our field explorations, laboratory testing, and engineering analyses, it is our opinion that the site is presently stable and suitable for the proposed new La Center hotel and commercial development and its associated site improvements provided that the recommendations contained within this report are properly incorporated into the design and construction of the project.

The primary features of concern at the site are 1) the presence of moisture sensitive clayey and silty subgrade soils across the site, 2) the presence of gently sloping site conditions across the central and easterly portions of the site, 3) the presence of the existing fill materials, and 4) the relatively low infiltration rates anticipated within the near surface clayey and silty subgrade soils.

With regard to the moisture sensitive clayey and silty subgrade soils, we are generally of the opinion that all site grading and earthwork activities be scheduled for the drier summer months which is typically June through September. In regard to the gently sloping site conditions across the central and easterly portions of the site, we are of the opinion that the existing slopes across the central and easterly portions of the site are stable. However, site grading and/or structural fill placement should be minimized where possible and should generally limit cuts and/or fills to about five (5) to ten (10) feet unless approved by the Geotechnical Engineer. Additionally, where existing site slopes and/or surface grades exceed about 20 percent (1V:5H) and/or in order to construct the proposed new site improvements, benching and keying of all fills into the natural site slopes may be required. With regard to the presence of the existing fill materials, we are of the opinion that the existing fill materials are unsuitable for support of the proposed new site improvements. As such, we recommend that all of the existing fill materials be removed in their entirety down to an approved native subgrade soil. In this regard, close monitoring by the Geotechnical Engineer during the site grading and earthwork operations will be required. In regard to the relatively low infiltration rates anticipated within the clayey and silty subgrade soils beneath the site, we generally do not recommend any storm water infiltration within structural fills and/or near the top of any cut and/or fill slope. However, some limited storm water infiltration may be feasible across the easterly portion of the site and/or within low lying areas of the site where the existing and/or finish slope gradients are no steeper than about 15 percent (1V:5H). In this regard, we recommend that all proposed storm water detention and/or infiltration systems for the project be reviewed and approved by Redmond Geotechnical Services, LLC.

The following sections of this report provide specific recommendations regarding subgrade preparation and grading as well as foundation and floor slab design and construction for the new La Center hotel and commercial development project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new hotel and commercial buildings as well as their associated structural and/or site improvement area(s) be stripped and cleared of all existing improvements, any existing fill materials, surface debris, existing vegetation, topsoil materials, and/or any other deleterious materials present at the time of construction. In general, we envision that the site stripping to remove existing vegetation and topsoil materials will generally be about 12 to 14 inches. However, localized areas requiring deeper removals, such as the existing undocumented and/or unsuitable fill materials as well as any old foundation remnants, will likely be encountered and should be evaluated at the time of construction by the Geotechnical Engineer. The stripped and cleared materials should be properly disposed of as they are generally considered unsuitable for use/reuse as fill materials.

Following the completion of the site stripping and clearing work and prior to the placement of any required structural fill materials and/or structural improvements, the exposed subgrade soils within the planned structural improvement area(s) should be inspected and approved by the Geotechnical Engineer and possibly proof-rolled with a half and/or fully loaded dump truck. Areas found to be soft or otherwise unsuitable should be over-excavated and removed or scarified and recompacted as structural fill. During wet and/or inclement weather conditions, proof rolling and/or scarification and recompaction as noted above may not be appropriate.

The on-site native clayey, sandy silt subgrade soil materials are generally considered suitable for use/reuse as structural fill materials provided that they are free of organic materials, debris, and rock fragments in excess of about 6 inches in dimension. However, if site grading is performed during wet or inclement weather conditions, the use of some of the on-site native soil materials which contain significant silt and clay sized particles will be difficult at best. In this regard, during wet or inclement weather conditions, we recommend that an import structural fill material be utilized which should consist of a free-draining (clean) granular fill (sand & gravel) containing no more than about 5 percent fines. Representative samples of the materials which are to be used as structural fill materials should be submitted to the Geotechnical Engineer and/or laboratory for approval and determination of the maximum dry density and optimum moisture content for compaction.

In general, all site earthwork and grading activities should be scheduled for the drier summer months (June through September) if possible. However, if wet weather site preparation and grading is required, it is generally recommended that the stripping of topsoil materials be accomplished with a tracked excavator utilizing a large smooth-toothed bucket working from areas yet to be excavated. Additionally, the loading of strippings into trucks and/or protection of moisture sensitive subgrade soils will also be required during wet weather grading and construction.

In this regard, we recommend that areas in which construction equipment will be traveling be protected by covering the exposed subgrade soils with a geotextile fabric such as Mirafi FW404 followed by at least 12 inches or more of crushed aggregate base rock. Further, the geotextile fabric should have a minimum Mullen burst strength of at least 250 pounds per square inch for puncture resistance and an apparent opening size (AOS) between the U.S. Standard No. 70 and No. 100 sieves.

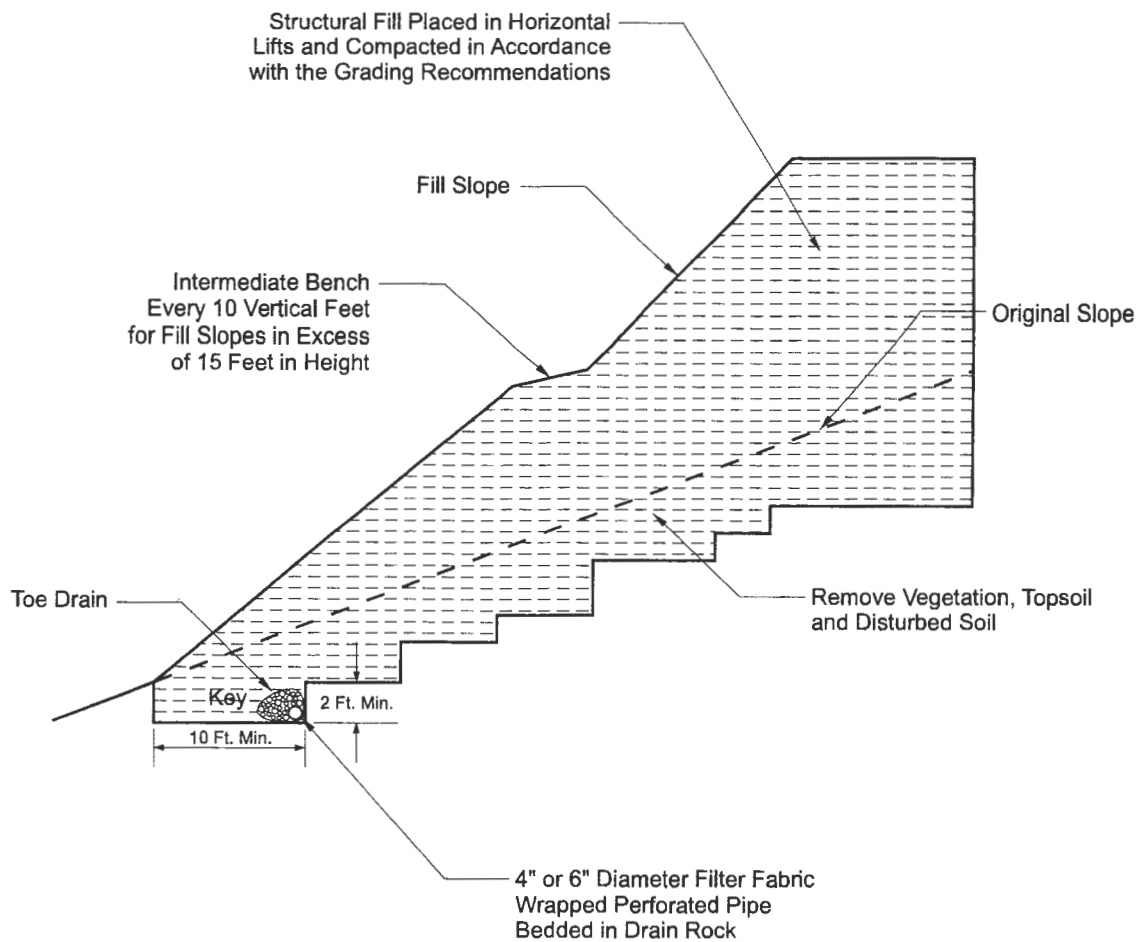
All structural fill materials placed within the new building and/or pavement areas should be moistened or dried as necessary to near (within 3 percent) optimum moisture conditions and compacted by mechanical means to a minimum of 92 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Structural fill materials should be placed in lifts (layers) such that when compacted do not exceed about 8 inches. Additionally, all fill materials placed within five (5) lineal feet of the perimeter (limits) of the proposed hotel and commercial structures and/or pavements should be considered structural fill. Additionally, due to the sloping site conditions, we recommend that all structural fill materials planned in areas where existing surface and/or slope gradients exceed about 20 percent (1V:5H) be properly benched and/or keyed into the native (natural) slope subgrade soils (see Typical Key and Bench Fill Slope Detail, Figure No. 4). In general, a bench width of at least eight (8) feet and a keyway depth of at least one (1) foot is recommended. However, the actual bench width and keyway depth should be determined at the time of construction by the Geotechnical Engineer. Further, all fill slopes should be constructed with a finish slope surface gradient no steeper than about 2H:1V. All aspects of the site grading, including a review of the proposed site grading plan(s), should be approved and/or monitored by a representative of Redmond Geotechnical Services, LLC.

Foundation Support

Based on the results of our investigation, it is our opinion that the site of the proposed new hotel and commercial development is suitable for support of the planned single- and/or five-story wood-frame structures provided that the following foundation design recommendations are followed. The following sections of this report present specific foundation design and construction recommendations for the planned new hotel and commercial structures.

Shallow Foundations

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved native (untreated) subgrade soil materials and/or clayey, sandy silt structural fill soils based on an allowable contact bearing pressure of about 2,500 pounds per square foot (psf). However, an allowable contact bearing pressure of up to 3,000 pounds per square foot (psf) may be used for design where foundations are supported by at least 8 inches or more of import crushed rock structural fill placed above an approved native subgrade soil. These recommended allowable contact bearing pressures are intended for dead loads and sustained live loads and may be increased by one-third for the total of all loads including short-term wind or seismic loads.



TYPICAL KEY AND BENCH FILL SLOPE DETAIL

Project No. 1171.009.G

LA CENTER HOTEL SITE
#209708000, NW LA CENTER ROAD

Figure No. 4

In general, continuous strip footings should have a minimum width of at least 16 inches and be embedded at least 18 inches below the lowest adjacent finish grade (includes frost protection). Individual column footings (where required) should be embedded at least 18 inches below grade and have a minimum width of at least 24 inches. Additionally, if foundation excavation and construction work is planned to be performed during wet and/or inclement weather conditions, we recommend that a 2- to 4-inch layer of compacted crushed rock be used to help protect the exposed foundation bearing surfaces until the placement of concrete.

Total and differential settlements of foundations constructed as recommended above and supported by approved native subgrade soils or by properly compacted structural fill materials are expected to be well within the tolerable limits for this type of single- and/or five-story wood-frame structure and should generally be less than about 1-inch and 1/2-inch, respectively.

Allowable lateral frictional resistance between the base of the footing element and the supporting subgrade bearing soil can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.30 and 0.45 for native silty subgrade soils and/or import gravel fill materials, respectively. In addition, lateral loads may be resisted by passive earth pressures on footings poured "neat" against in-situ (native) subgrade soils or properly backfilled with structural fill materials based on an equivalent fluid density of 300 pounds per cubic foot (pcf). This recommended value includes a factor of safety of approximately 1.5 which is appropriate due to the amount of movement required to develop full passive resistance.

Floor Slab Support

In order to provide uniform subgrade reaction beneath concrete slab-on-grade floors, we recommend that the floor slab area be underlain by a minimum of 6 inches of free-draining (less than 5 percent passing the No. 200 sieve), well-graded, crushed rock. The crushed rock should help provide a capillary break to prevent migration of moisture through the slab. However, additional moisture protection can be provided by using a 10-mil polyolefin geo-membrane sheet such as StegoWrap.

The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Where floor slab subgrade materials are undisturbed, firm and stable and where the underslab aggregate base rock section has been prepared and compacted as recommended above, we recommend that a modulus of subgrade reaction of 150 pci be used for design.

Retaining/Below Grade Walls

Retaining and/or below grade walls should be designed to resist lateral earth pressures imposed by native soils or granular backfill materials as well as any adjacent surcharge loads. For walls which are unrestrained at the top and free to rotate about their base, we recommend that active earth pressures be computed on the basis of the following equivalent fluid densities:

Non-Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	35	30
3H:1V	60	50
2H:1V	90	80

For walls which are fully restrained at the top and prevented from rotation about their base, we recommend that at-rest earth pressures be computed on the basis of the following equivalent fluid densities:

Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	55	50
3H:1V	75	70
2H:1V	95	90

The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher. For seismic loading, we recommend an additional uniform pressure of 7H where H is the height of the wall in feet.

Backfill materials behind walls should be compacted to 90 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Special care should be taken to avoid over-compaction near the walls which could result in higher lateral earth pressures than those indicated herein. In areas within three (3) to five (5) feet behind walls, we recommend the use of hand-operated compaction equipment.

Pavements

Flexible pavement design for the project was determined on the basis of projected (anticipated) traffic volume and loading conditions relative to laboratory subgrade soil strength ("R"-value) characteristics. Based on an average laboratory subgrade "R"-value of 28 (Resilient Modulus = 5,000 to 10,000) and utilizing the Asphalt Institute Flexible Pavement Design Procedures and/or the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Design of Pavement Structures" manual, we recommend that the asphaltic concrete pavement section(s) for the new apartment development areas at the site consist of the following:

Automobile Parking and Access Drives

The following documents and/or design input parameters were used to help determine the flexible pavement section design for new private automobile parking and access drive areas:

	<u>Asphaltic Concrete Thickness (inches)</u>	<u>Crushed Base Rock Thickness (inches)</u>
Automobile Parking Areas	3.0	8.0
Automobile Drive Areas	3.0	10.0

Note: Where heavy vehicle traffic is anticipated such as those required for fire and/or garbage trucks, we recommend that the automobile drive area pavement section be increased by adding 0.5 inches of asphaltic concrete and 2.0 inches of aggregate base rock. Additionally, the above recommended flexible pavement section(s) assumes a design life of 20 years.

Pavement Subgrade, Base Course & Asphalt Materials

The above recommended pavement section(s) were based on the design assumptions listed herein and on the assumption that construction of the pavement section(s) will be completed during an extended period of reasonably dry weather. All thicknesses given are intended to be the minimum acceptable. Increased base rock sections and the use of a woven geotextile fabric may be required during wet and/or inclement weather conditions and/or in order to adequately support construction traffic and protect the subgrade during construction. Additionally, the above recommended pavement section(s) assume that the subgrade will be prepared as recommended herein, that the exposed subgrade soils will be properly protected from rain and construction traffic, and that the subgrade is firm and unyielding at the time of paving. Further, it assumes that the subgrade is graded to prevent any ponding of water which may tend to accumulate in the base course.

Pavement base course materials should consist of well-graded 1-1/4 inch and/or 5/8-inch minus crushed base rock having less than 5 percent fine materials passing the No. 200 sieve. The base course and asphaltic concrete materials should conform to the requirements set forth in the latest edition of the Washington Department of Transportation, Standard Specifications for Highway Construction. The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. The asphaltic concrete paving materials should be compacted to at least 92 percent of the theoretical maximum density as determined by the ASTM D-2041 (Rice Gravity) test method.

Wet Weather Grading and Soft Spot Mitigation

Construction of the proposed new paved site improvements is generally recommended during dry weather. However, during wet weather grading and construction, excavation to subgrade can proceed during periods of light to moderate rainfall provided that the subgrade remains covered with aggregate.

A total aggregate thickness of 8- to 12-inches may be necessary to protect the subgrade soils from heavy construction traffic. Construction traffic should not be allowed directly on the exposed subgrade but only atop a sufficient compacted base rock thickness to help mitigate subgrade pumping. If the subgrade becomes wet and pumps, no construction traffic shall be allowed on the road alignment. Positive site drainage shall be maintained if site paving will not occur before the onset of the wet season.

Depending on the timing for the project, any soft subgrade found during proof-rolling or by visual observations can either be removed and replaced with properly dried and compacted fill soils or removed and replaced with compacted crushed aggregate. However, and where approved by the Geotechnical Engineer, the soft area may be covered with a bi-axial geogrid and covered with compacted crushed aggregate.

Soil Shrink-Swell and Frost Heave

The results of the laboratory "R"-value tests indicate that the native subgrade soils possess a low to moderate expansion potential. As such, the exposed subgrade soils should not be allowed to completely dry and should be moistened to near optimum moisture content (plus or minus 3 percent) at the time of the placement of the crushed aggregate base rock materials. Additionally, exposure of the subgrade soils to freezing weather may result in frost heave and softening of the subgrade. As such, all subgrade soils exposed to freezing weather should be evaluated and approved by the Geotechnical Engineer prior to the placement of the crushed aggregate base rock materials.

Excavation/Slopes

Temporary excavations of up to about four (4) feet in depth may be constructed with near vertical inclinations. Temporary excavations greater than about four (4) feet but less than eight (8) feet should be excavated with inclinations of at least 1 to 1 (horizontal to vertical) or properly braced/shored. Where excavations are planned to exceed about eight (8) feet, this office should be consulted. All shoring systems and/or temporary excavation bracing for the project should be the responsibility of the excavation contractor. Permanent slopes should be constructed no steeper than about 2H to 1V unless approved by the Geotechnical Engineer.

Depending on the time of year in which trench excavations occur, trench dewatering may be required in order to maintain dry working conditions if the invert elevations of the proposed utilities are located at and/or below the groundwater level. If groundwater is encountered during utility excavation work, we recommend placing trench stabilization materials along the base of the excavation. Trench stabilization materials should consist of 1-foot of well-graded gravel, crushed gravel, or crushed rock with a maximum particle size of 4 inches and less than 5 percent fines passing the No. 200 sieve. The material should be free of organic matter and other deleterious material and placed in a single lift and compacted until well keyed.

Surface Drainage/Groundwater

We recommend that positive measures be taken to properly finish grade the site so that drainage waters from the hotel and commercial structures and landscaping areas as well as adjacent properties or buildings are directed away from the new hotel and commercial structures foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the hotel and commercial structures to a suitable outfall. Roof downspouts should not be connected to foundation drains. A minimum ground slope of about 2 percent is generally recommended in unpaved areas around the proposed new hotel and commercial structures.

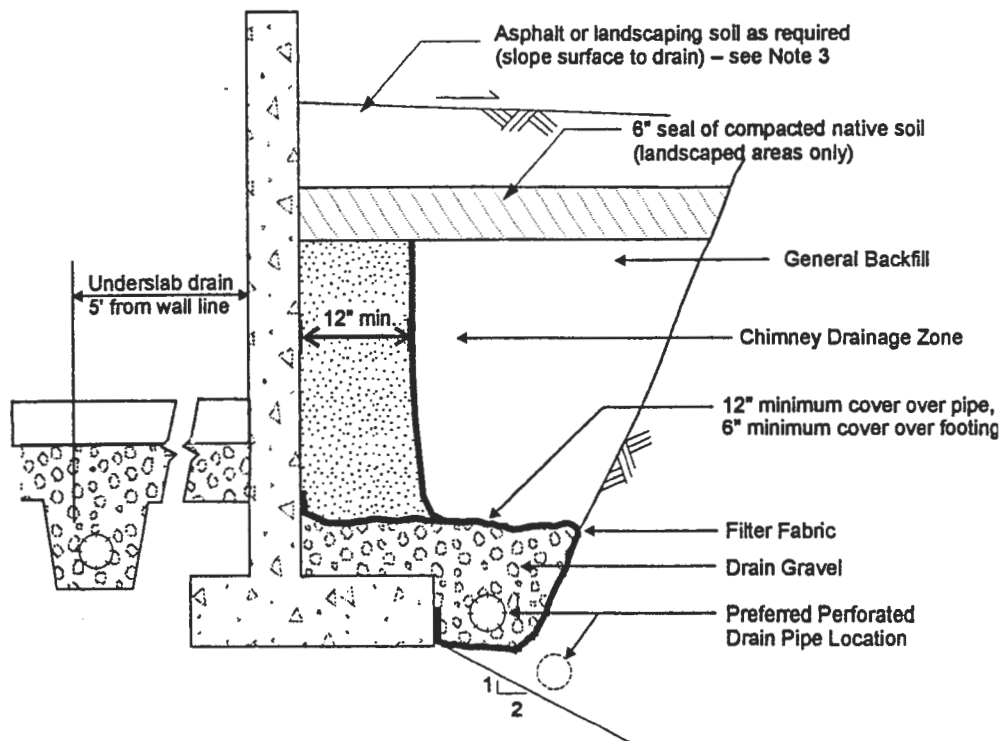
Groundwater was encountered at the site within two (2) of the exploratory test borings (B-#1 and B-#2) drilled at the site at the time of our field work at depths of about fifteen (15) and twenty (20) feet beneath existing site grades. However, the easterly portion of the site contains an existing seasonal drainage basin. Additionally, the site is underlain by medium stiff, clayey, sandy silt soil. Further, groundwater elevations in the area and/or across the subject property may fluctuate seasonally and may temporarily pond/perch near the ground surface during periods of prolonged rainfall. As such, based on our current understanding of the possible site grading required to bring the subject site to finish design grade(s), we are of the opinion that an underslab drainage system is generally not required for the proposed hotel and/or commercial structures. However, a perimeter foundation drain is recommended for any perimeter footings and/or below grade retaining walls. A typical recommended perimeter footing/retaining wall drain detail is shown on Figure No. 5. Further, due to our understanding that various surface infiltration basins and/or facilities may be utilized for the project as well as the relatively low infiltration rates of the near surface clayey, sandy silt subgrade soil deposits anticipated within and/or near to the foundation bearing level of the proposed hotel and/or commercial structures, we are generally of the opinion that storm water detention and/or disposal systems should not be utilized around and/or up-gradient of the proposed hotel and/or commercial structures unless approved by the Geotechnical Engineer.

Design Infiltration Rates

Based on the results of our field infiltration testing, we recommend using the following infiltration rate to design any on-site near surface storm water infiltration and/or disposal systems for the project:

Subgrade Soil Type	Recommended Infiltration Rate
clayey, sandy SILT (ML)	0.7 inches per hour (in/hr)

Note: A safety factor of two (2) was used to calculate the above recommended design infiltration rate. Additionally, given the gradational variability of the on-site clayey, sandy silt subgrade soils beneath the site as well as the anticipation of some site grading for the project, it is generally recommended that field testing be performed during and/or following construction of any on-site storm water infiltration system(s) in order to confirm that the above recommended design infiltration rates are appropriate.



SCHEMATIC - NOT TO SCALE

NOTES:

1. Filter Fabric to be non-woven geotextile (Amoco 4545, Mirafi 140N, or equivalent)
2. Lay perforated drain pipe on minimum 0.5% gradient, widening excavation as required. Maintain pipe above 2:1 slope, as shown.
3. All-granular backfill is recommended for support of slabs, pavements, etc. (see text for structural fill).
4. Drain gravel to be clean, washed $\frac{3}{4}$ " to $1\frac{1}{2}$ " gravel.
5. General backfill to be on-site gravels, or $\frac{3}{4}$ "-0 or $1\frac{1}{2}$ "-0 crushed rock compacted to 92% Modified Proctor (AASHTO T-180).
6. Chimney drainage zone to be 12" wide (minimum) zone of clean washed, medium to coarse sand or drain gravel if protected with filter fabric. Alternatively, prefabricated drainage structures (Miradrain 6000 or similar) may be used.

TYPICAL PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL

Project No. 1171.009.G

**LA CENTER HOTEL SITE
#209708000, NW LA CENTER ROAD**

Figure No. 5

Seismic Design Considerations

Structures at the site should be designed to resist earthquake loading in accordance with the methodology described in the latest edition of the State of Washington Structural Specialty Code (WSSC), ASCE 7-16 and/or Amendments to the 2018 International Building Code (IBC). The maximum considered earthquake ground motion for short period and 1.0 period spectral response may be determined from the Washington Structural Specialty Code, ASCE 7-16 and/or from the 2015 National Earthquake Hazard Reduction Program (NEHRP) "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" published by the Building Seismic Safety Council. We recommend Site Class "D" be used for design. Using this information, the structural engineer can select the appropriate site coefficient values (F_a and F_v) from ASCE 7-16 and/or the 2018 IBC to determine the maximum considered earthquake spectral response acceleration for the project. However, we have assumed the following response spectrum for the project:

Table 1. ASCE 7-16 Recommended Seismic Design Parameters

Site Class	S_s	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.807	0.381	1.177	1.919	0.950	0.732	0.633	0.488

Notes: 1. S_s and S_1 were established based on the USGS 2015 mapped maximum considered earthquake spectral acceleration maps for 2% probability of exceedence in 50 years.

2. F_a and F_v were established based on ASCE 7-16 using the selected S_s and S_1 values.

CONSTRUCTION MONITORING AND TESTING

We recommend that **Redmond Geotechnical Services, LLC** be retained to provide construction monitoring and testing services during all earthwork operations for the proposed new La Center hotel and commercial development. The purpose of our monitoring services would be to confirm that the site conditions reported herein are as anticipated, provide field recommendations as required based on the actual conditions encountered, document the activities of the grading contractor and assess his/her compliance with the project specifications and recommendations. It is important that our representative meet with the contractor prior to any site grading to help establish a plan that will minimize costly over-excavation and site preparation work. Of primary importance will be observations made during site preparation and stripping, structural fill placement, footing excavations and construction as well as retaining wall backfill.

CLOSURE AND LIMITATIONS

This report is intended for the exclusive use of the addressee and/or their representative(s) to use to design and construct the proposed new hotel and commercial structures and their associated site improvements described herein as well as to prepare any related construction documents. The conclusions and recommendations contained in this report are based on site conditions as they presently exist and assume that the explorations are representative of the subsurface conditions between the explorations and/or at other locations across the study area. The data, analyses, and recommendations herein may not be appropriate for other structures and/or purposes. We recommend that parties contemplating other structures and/or purposes contact our office. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report. Additionally, the above recommendations are contingent on Redmond Geotechnical Services, LLC being retained to provide all site inspections and construction monitoring services for this project. Redmond Geotechnical Services, LLC will not assume any responsibility and/or liability for any engineering judgment, inspection and/or testing services performed by others.

It is the owners/developer's responsibility for ensuring that the project designers and/or contractors involved with this project implement our recommendations into the final design plans, specifications and/or construction activities for the project. Further, in order to avoid delays during construction, we recommend that the final design plans and specifications for the project be reviewed by our office to evaluate as to whether our recommendations have been properly interpreted and incorporated into the project.

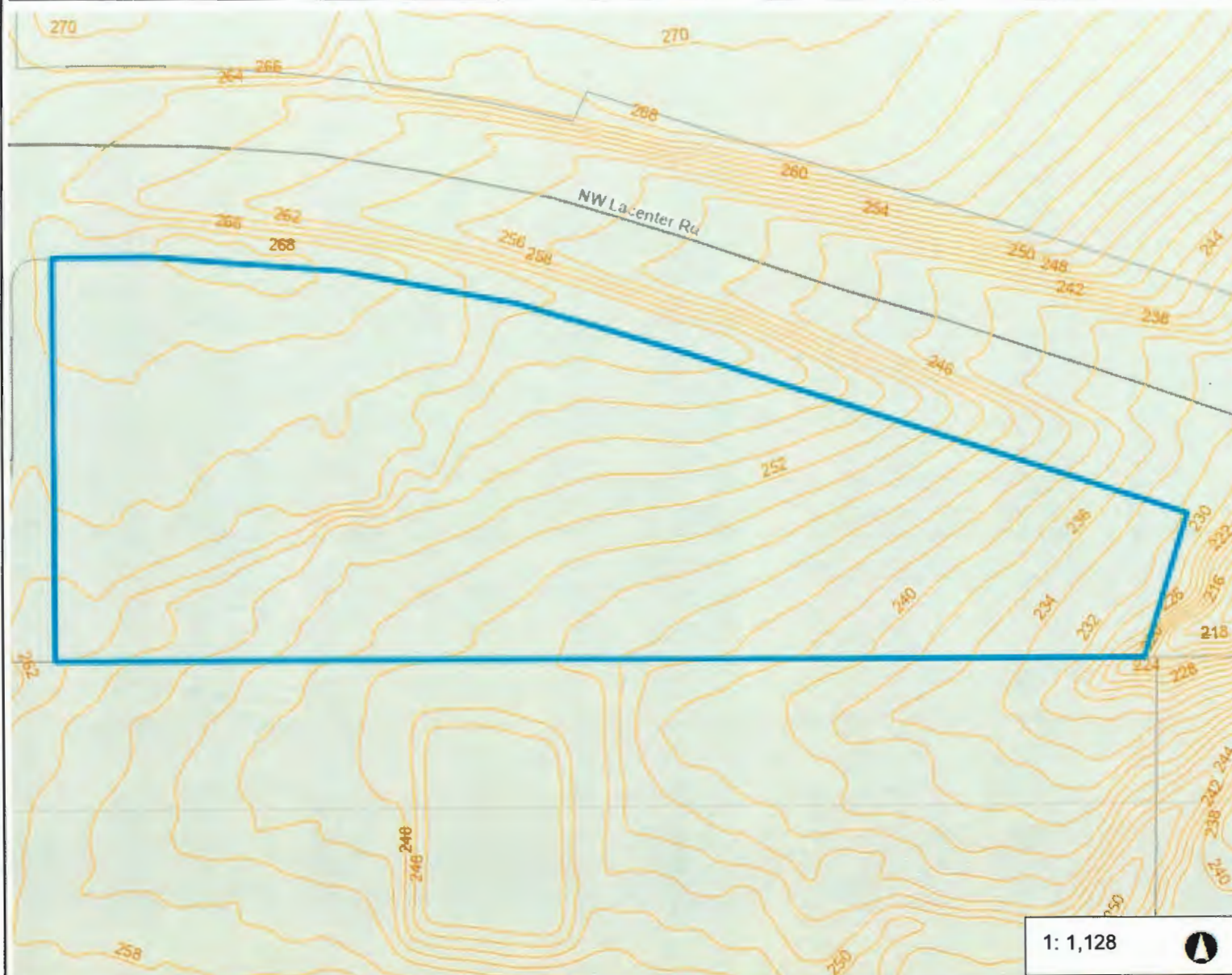
If during any future site grading and construction, subsurface conditions different from those encountered in the explorations are observed or appear to be present beneath excavations, we should be advised immediately so that we may review these conditions and evaluate whether modifications of the design criteria are required. We also should be advised if significant modifications of the proposed site development are anticipated so that we may review our conclusions and recommendations.

LEVEL OF CARE

The services performed by the Geotechnical Engineer for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in the area under similar budget and time restraints. No warranty or other conditions, either expressed or implied, is made.



Parcel #209708000



Legend

- Taxlots
- Contours Lines - 2 ft

Notes:

188.0 0 94.00 188.0 Feet

WGS_1984_Web_Mercator_Auxiliary_Sphere
Clark County, WA. GIS - <http://gis.clark.wa.gov>

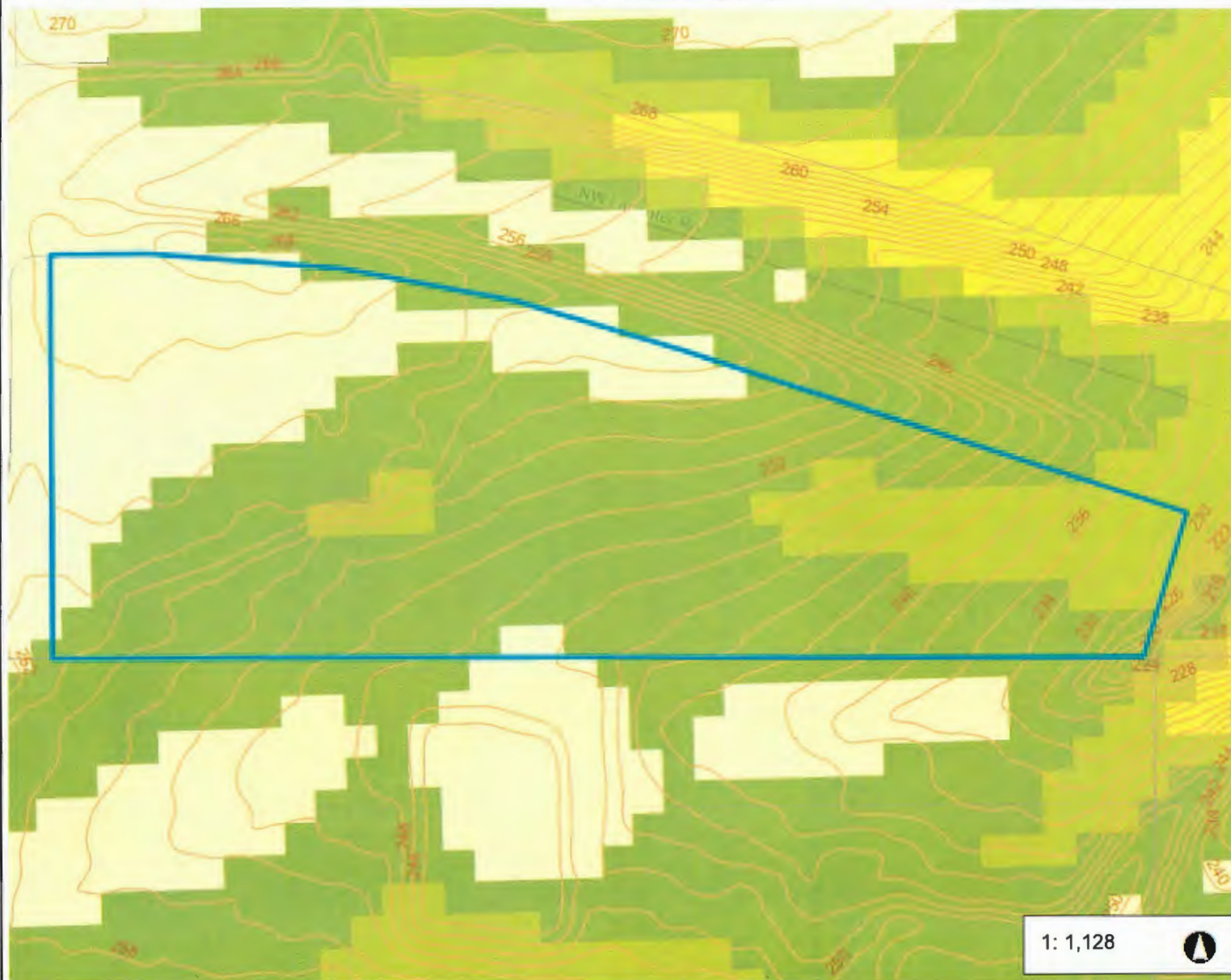
1: 1,128



This map was generated by Clark County's "MapsOnline" website. Clark County does not warrant the accuracy, reliability or timeliness of any information on this map, and shall not be held liable for losses caused by using this information. Taxlot (i.e., parcel) boundaries cannot be used to determine the location of property lines on the ground.



Parcel #209708000



Legend

- ☐ Taxlots
- ☐ Contours Lines - 2 ft
- Percent Slope**
 - 0-5 percent
 - >5-10 percent
 - >10-15 percent
 - >15-25 percent
 - >25-40 percent
 - >40-80 percent
 - >80 percent

Notes:

188.0 0 94.00 188.0 Feet

WGS_1984_Web_Mercator_Auxiliary_Sphere
Clark County, WA. GIS - <http://gis.clark.wa.gov>

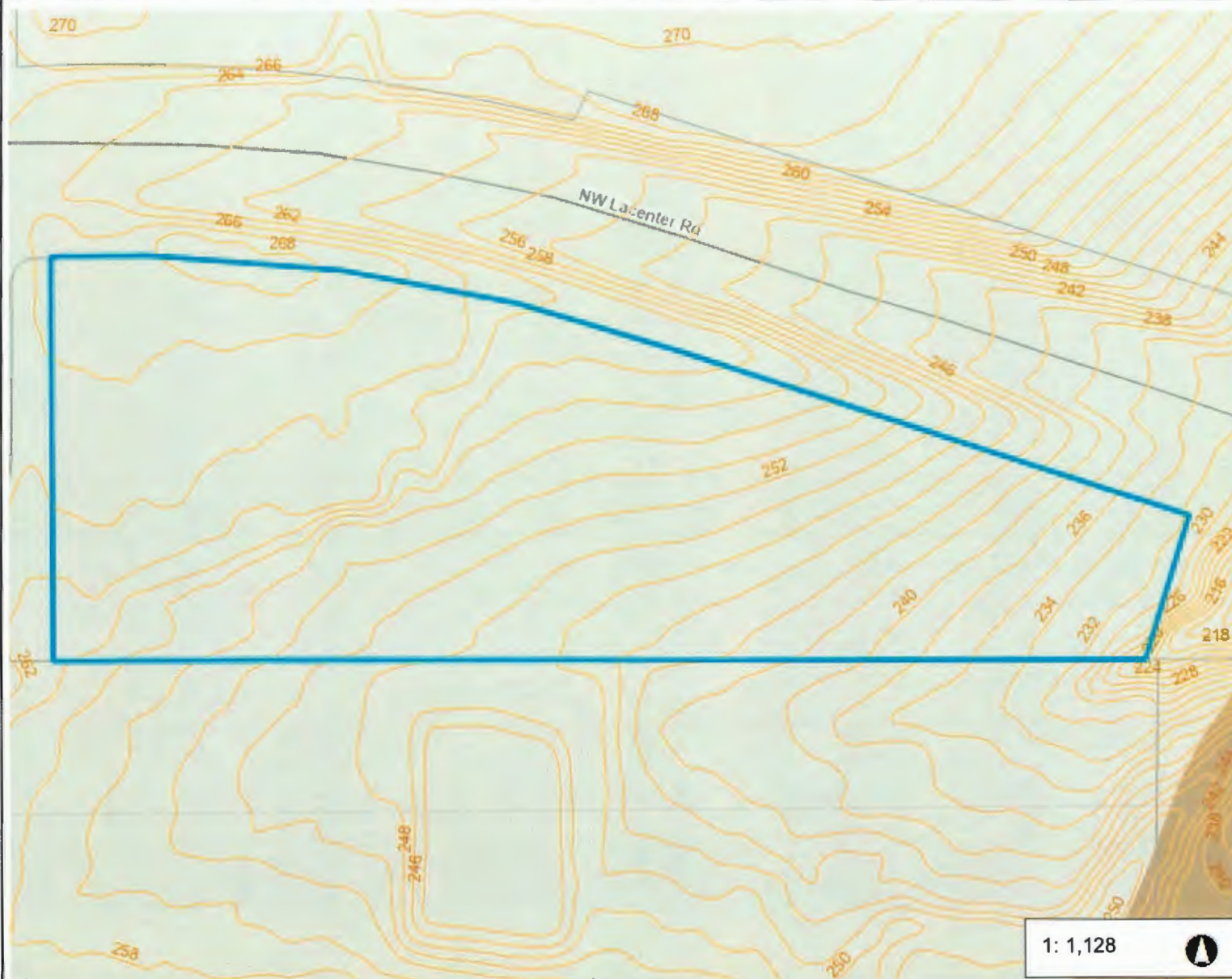
1: 1,128



This map was generated by Clark County's "MapsOnline" website. Clark County does not warrant the accuracy, reliability or timeliness of any information on this map, and shall not be held liable for losses caused by using this information. Taxdot (i.e., parcel) boundaries cannot be used to determine the location of property lines on the ground.



Parcel #209708000



Legend

- Taxlots
- Contours Lines - 2 ft
- Severe Erosion Hazard Areas

Notes:

188.0 0 94.00 188.0 Feet

WGS_1984_Web_Mercator_Auxiliary_Sphere
Clark County, WA. GIS - <http://gis.clark.wa.gov>

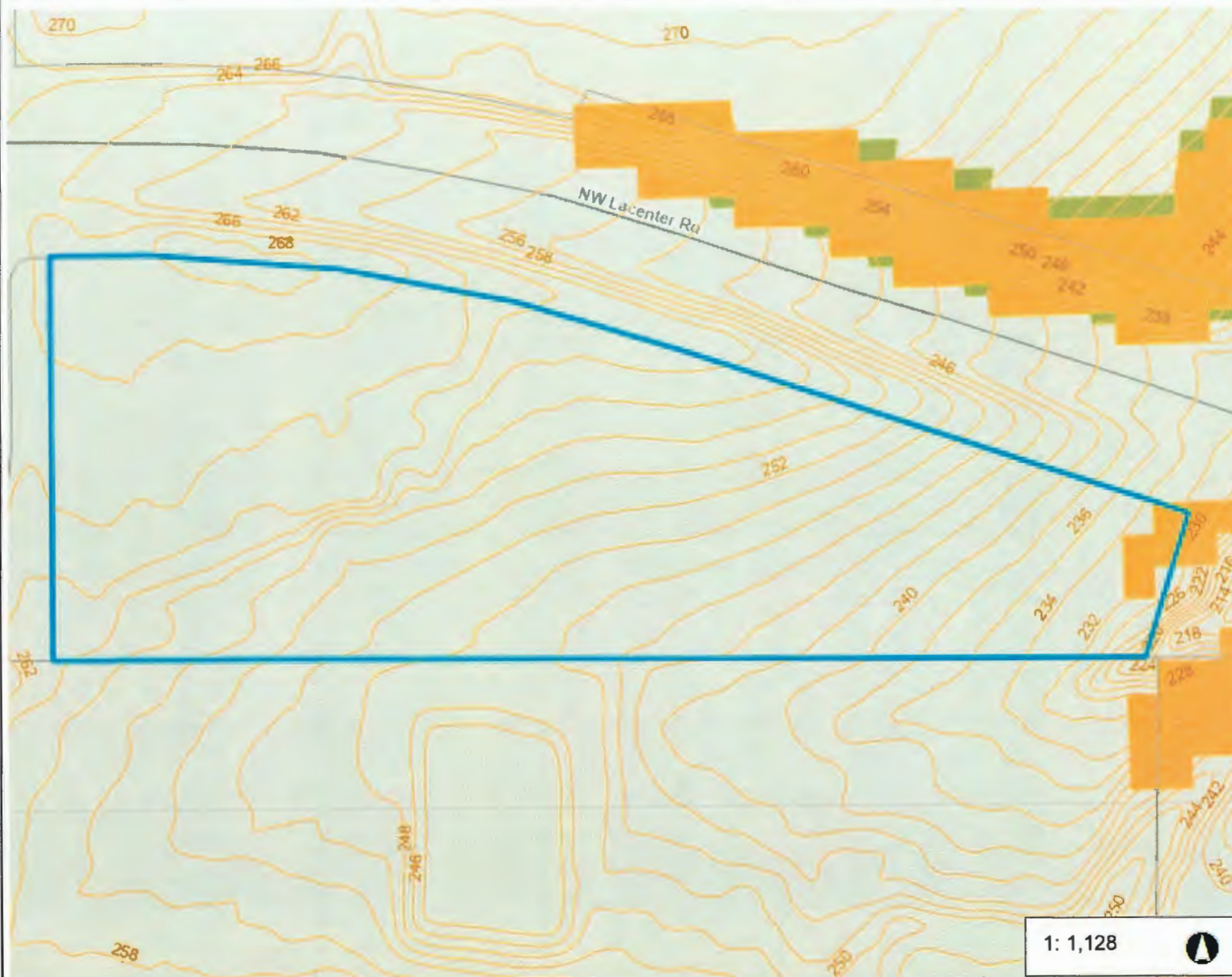
1: 1,128



This map was generated by Clark County's "MapsOnline" website. Clark County does not warrant the accuracy, reliability or timeliness of any information on this map, and shall not be held liable for losses caused by using this information. Taxlot (i.e., parcel) boundaries cannot be used to determine the location of property lines on the ground.



Parcel #209708000



Legend

- Taxlots
- Contours Lines - 2 ft
- Steep Slopes and Landslide H.**
 - Areas of Historic or Active Landslide
 - Areas of Potential Instability
 - Areas of Older Landslide Debris
 - Slopes > 15%
 - Slopes > 25%

Notes:

188.0 0 94.00 188.0 Feet

WGS_1984_Web_Mercator_Auxiliary_Sphere
Clark County, WA. GIS - <http://gis.clark.wa.gov>

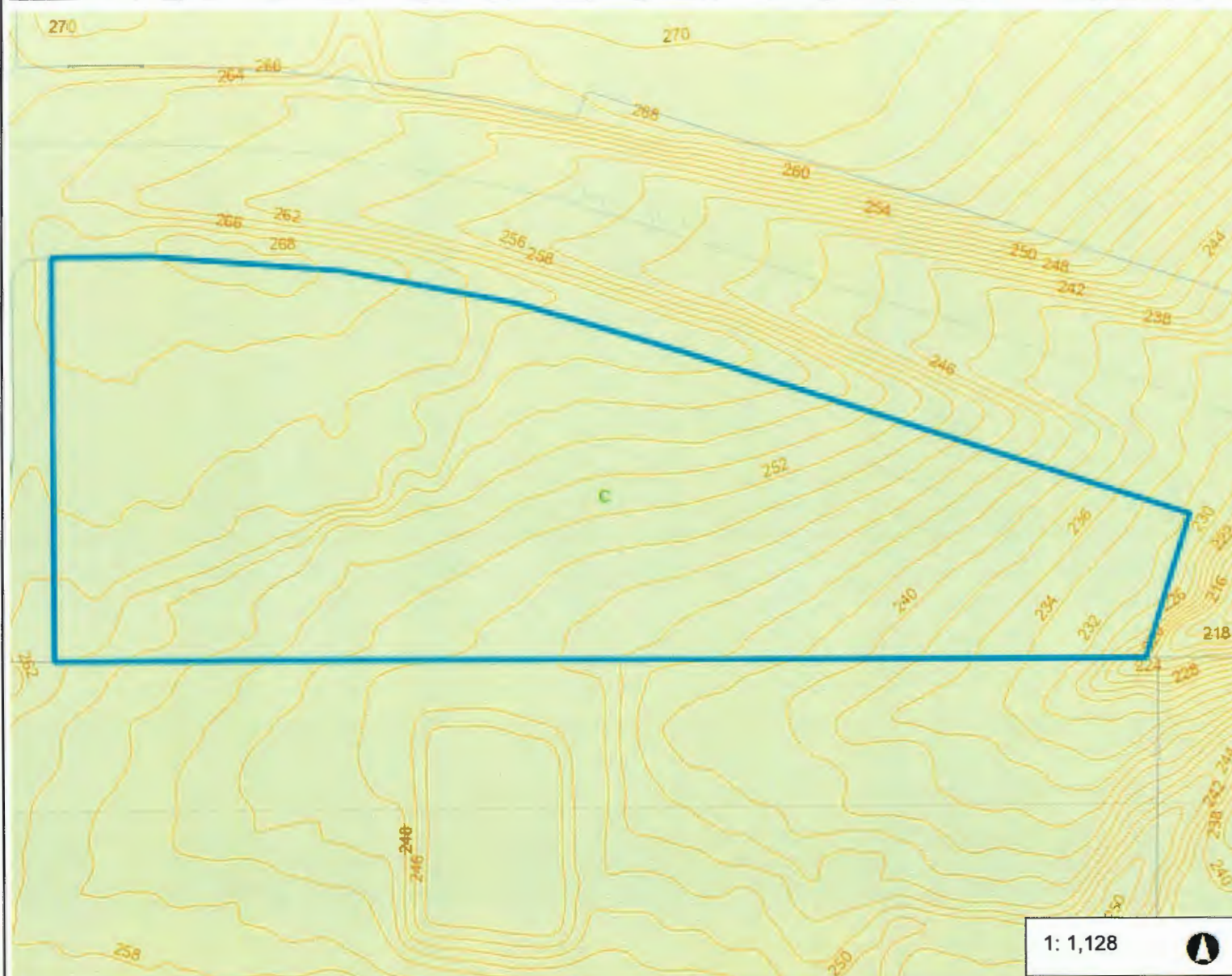
1: 1,128



This map was generated by Clark County's "MapsOnline" website. Clark County does not warrant the accuracy, reliability or timeliness of any information on this map, and shall not be held liable for losses caused by using this information. Taxlot (i.e., parcel) boundaries cannot be used to determine the location of property lines on the ground.



Parcel #209708000



Legend

- ☐ Taxlots
- Contours Lines - 2 ft
- NEHRP Site Classes
 - Site Class E - Highest Relative Haz
 - Site Class D-E
 - Site Class D
 - Site Class C-D
 - Site Class C
 - Site Class B-C
 - Site Class B - Lowest Relative Haz
 - PEAT
 - WATER

Notes:

188.0 0 94.00 188.0 Feet

WGS_1984_Web_Mercator_Auxiliary_Sphere
Clark County, WA. GIS - <http://gis.clark.wa.gov>

1: 1,128



This map was generated by Clark County's "MapsOnline" website. Clark County does not warrant the accuracy, reliability or timeliness of any information on this map, and shall not be held liable for losses caused by using this information. Taxdot (i.e., parcel) boundaries cannot be used to determine the location of property lines on the ground.



Parcel #209708000



Legend

- Taxlots
- Contours Lines - 2 ft
- Liquefaction Susceptibility**
 - Moderate to High
 - Low to Moderate
 - Very Low to Low
 - Very Low
 - Bedrock
 - Peat
 - Water

Notes:

188.0 0 94.00 188.0 Feet

WGS_1984_Web_Mercator_Auxiliary_Sphere
Clark County, WA. GIS - <http://gis.clark.wa.gov>

1: 1,128



This map was generated by Clark County's "MapsOnline" website. Clark County does not warrant the accuracy, reliability or timeliness of any information on this map, and shall not be held liable for losses caused by using this information. Taxdot (i.e., parcel) boundaries cannot be used to determine the location of property lines on the ground.

REFERENCES

- Adams, John, 1984, Active Deformation of the Pacific Northwest Continental Margin: *Tectonics*, v.3, no. 4, p. 449-472.
- Applied Technology Council, ATC-13, 1985, Earthquake Damage Evaluation Data for California.
- Atwater, B.F., 1992, Geologic evidence for earthquakes during the past 2000 years along the Copalis River, southern coastal Washington: *Journal of Geophysical Research*, v. 97, p. 1901-1919.
- Atwater, B.F., 1987a, A periodic Holocene recurrence of widespread, probably coseismic Subsidence in southwestern Washington: *EOS*, v. 68, no. 44.
- Atwater, B.F., 1987b, Evidence for great Holocene earthquakes along the outer coast of Washington State: *Science*, v. 236, no. 4804, pp. 942-944.
- Campbell, K.W., 1990, Empirical prediction of near-surface soil and soft-rock ground motion for the Diablo Canyon Power Plant site, San Luis Obispo County, California: Dames & Moore report to Lawrence Livermore National Laboratory.
- Carver, G.A., and Burke, R.M., 1987, Late Holocene paleoseismicity of the southern end of the Cascadia Subduction zone [abs.]: *EOS*, v. 68, no. 44, p. 1240.
- Chase, R.L., Tiffin, D.L., Murray, J.W., 1975, The western Canadian continental margin: In Yorath, C.J., Parker, E.R., Glass, D.J., editors, *Canada's continental margins and offshore petroleum exploration: Canadian Society of Petroleum Geologists Memoir 4*, p. 701-721.
- Crouse, C.B., 1991a, Ground motion attenuation equations for earthquakes on the Cascadia Subduction Zone: *Earthquake Spectra*, v. 7, no. 2, pp. 201-236.
- Crouse, C.B., 1991b, Errata to Crouse (1991a), *Earthquake Spectra*, v. 7, no. 3, p. 506.
- Dariento, M.E., and Peterson, C.D., 1987, Episodic tectonic subsidence recorded in late Holocene salt marshes, northern Oregon central Cascadia margin: *Tectonics*, v. 9, p. 1-22.
- Dariento, M.E., and Peterson, C.D., 1987, Episodic tectonic subsidence recorded in late Holocene salt marshes northwest Oregon [abs]: *EOS*, v. 68, no. 44, p. 1469.
- EERI (Earthquake Engineering Research Institute), 1993, The March 25, 1993, Scotts Mill Earthquake, Western Oregon's Wake-Up Call: *EERI Newsletter*, Vol. 27, No. 5, May.
- Geomatrix, 1995 Seismic Design Mapping, State of Oregon: Final Report to Oregon Department of Transportation, January.

Geologic Map Series (GMS-49), Map of Oregon Seismicity, 1841-1986 dated 1986.

Geologic Map of the Ridgefield Quadrangle, Clark and Cowlitz Counties, Washington (Scientific Investigations Map 2844) by Russell C. Evarts dated 2004.

Grant, W.C., and McLaren, D.D., 1987, Evidence for Holocene Subduction earthquakes along the northern Oregon coast [abs]: EOS v. 68, no. 44, p. 1239.

Grant, W.C., Atwater, B.F., Carver, G.A., Darienzo, M.E., Nelson, A.R., Peterson, C.D., and Vick, G.S., 1989, Radiocarbon dating of late Holocene coastal subsidence above the Cascadia Subduction zone-compilation for Washington, Oregon, and northern California, [abs]: EOS Transactions of the American Geophysical Union, v. 70, p. 1331.

International Conference of Building Officials (ICBO), 1994, Uniform Building Code: 1994 Edition, Whittier, CA. 1994.

Joyner, W.B., and Boore, D.M., 1998, Measurement, characterization and prediction of strong ground motion: Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation, ASCE Geotech. Special Publ. No. 20, p. 43-102.

Riddihough, R.P., 1984, Recent movements of the Juan de Fuca plate system: Journal of Geophysical Research, v. 89, no. B8, p. 6980-6994.

Youngs, R.R., Day, S.M., and Stevens, J.L., 1998, Near field ground motions on rock for large Subduction earthquakes: Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation, ASCE Geotech. Special Publ. No. 20, p. 445-462.

Appendix "A"

Test Boring/Test Pit Logs and Laboratory Test Data

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATION

Subsurface conditions at the site were explored by excavating seven (7) exploratory test pits (TH-#1 through TH-#7) on February 6, 2024 and drilling three (3) exploratory borings on February 17, 2024. The approximate location of the test pit and test boring explorations are shown in relation to the existing site features and/or site improvements on the Site Exploration Plan, Figure No. 3.

The test pits and/or test borings were excavated and/or drilled with track-mounted excavating and/or Geoprobe drilling equipment in general conformance with ASTM Methods in Vol. 4.08, D-1586-94 and D-1587-83. The test pits were excavated to depths ranging from about 6.0 to 7.0 feet beneath existing site and/or surface grades while the test borings were drilled to depths of between 21.5 and 26.5 feet beneath the existing site and/or surface grades. Detailed logs of the test borings and test pits are presented on the Boring Logs and Log of Test Pits, Figure No's. A-5 through A-11. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-4.

The exploration program was coordinated by a field engineer who monitored the excavating and/or drilling exploration activity, obtained representative samples of the subsurface soils encountered, classified the soils by visual and textural examination, and maintained continuous logs of the subsurface conditions. Disturbed and/or undisturbed samples of the subsurface soils were obtained at appropriate depths and/or intervals and placed in plastic bags and/or with a thin walled ring sample.

Groundwater was encountered within two (2) of the exploratory test borings (B-#1 and B-#2) at the time of the field work at depths of between 15 and 20 feet beneath existing surface grades.

LABORATORY TESTING

Pertinent physical and engineering characteristics of the soils encountered during our subsurface investigation were evaluated by a laboratory testing program to be used as a basis for selection of soil design parameters and for correlation purposes. Selected tests were conducted on representative soil samples. The program consisted of tests to evaluate the existing (in-situ) moisture-density, maximum dry density and optimum moisture content, Atterberg Limits and gradational characteristics as well as consolidation, direct shear strength and "R"-value tests.

Dry Density and Moisture Content Determinations

Density and moisture content determinations were performed on both disturbed and relatively undisturbed samples from the test pit and test boring explorations in general conformance with ASTM Vol. 4.08 Part D-216. The results of these tests were used to calculate existing overburden pressures and to correlate strength and compressibility characteristics of the soils. Test results are shown on the test pit and test boring logs at the appropriate sample depths.

Maximum Dry Density

One (1) Maximum Dry Density and Optimum Moisture Content test was performed on a representative sample of the on-site clayey, sandy silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. This test was conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-12.

Atterberg Limits

Two (2) Liquid Limit (LL) and Plastic Limit (PL) tests were performed on representative samples of the clayey, sandy silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-4318-85. These tests were conducted to facilitate classification of the soils and for correlation purposes. The test results appear on Figure No. A-13.

Gradation Analysis

Two (2) Gradation analyses were performed on representative samples of the clayey, sandy silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-422. The test results were used to classify the soil in accordance with the Unified Soil Classification System (USCS). The test results are shown graphically on Figure No. A-14.

Consolidation Test

One (1) Consolidation test was performed on a representative sample of the clayey, sandy silt subgrade soil to assess the compressibility characteristics of the underlying subgrade soils in accordance with ASTM Vol. 4.08 Part D-2435-80.

Conventional loading increments of 100, 200, 400, ... 12,800 psf were applied after the 100 percent time of primary consolidation was identified for each loading increment. The samples were unloaded and allowed to rebound after the completion of the loading sequence. Deflection versus time readings were recorded for all load increments from 100 through 12,800 psf. The deflection corresponding to 100 percent primary consolidation was plotted on the consolidation strain versus consolidation pressure curve, which is presented on Figure No. A-15.

Direct Shear Strength Test

One (1) Direct Shear Strength test was performed on an undisturbed and/or remolded sample at a continuous rate of shearing deflection (0.02 inches per minute) in accordance with ASTM Vol. 4.08 Part D-3080-79. The test results were used to determine engineering strength properties and are shown graphically on Figure No. A-16.

"R"-Value Tests

One (1) "R"-value test was performed on a remolded subgrade soil sample in accordance with ASTM Vol. 4.08 Part D-2844. The test results were used to help evaluate the subgrade soils supporting and performance capabilities when subjected to traffic loading. The test results are shown on Figure No. A-17.

The following figures are attached and complete the Appendix:

Figure No. A-4	Key To Exploratory Test Pit Logs
Figure No's. A-5 through A-11	Log of Test Borings/Test Pits
Figure No. A-12	Maximum Dry Density
Figure No. A-13	Atterberg Limits Test Results
Figure No. A-14	Gradation Test Results
Figure No. A-15	Consolidation Test Results
Figure No. A-16	Direct Shear Strength Test Results
Figure No. A-17	Results of "R"-Value Tests
Figure No. A-18	Field Infiltration Test Results

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

DEFINITION OF TERMS

U.S. STANDARD SERIES SIEVE								CLEAR SQUARE SIEVE OPENINGS		
200		40		10		4		3/4"	3"	12"
SILTS AND CLAYS		SAND			GRAVEL		COBBLES	BOULDERS		
		FINE	MEDIUM	COARSE	FINE	COARSE				

GRAIN SIZES

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT [†]
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50


CLAYS AND PLASTIC SILTS	STRENGTH [‡]	BLOWS/FOOT [†]
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

RELATIVE DENSITY

[†] Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

[‡] Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

CONSISTENCY

 REDMOND GEOTECHNICAL SERVICES PO Box 20547 • PORTLAND, OREGON 97294	KEY TO EXPLORATORY BORING LOGS Unified Soil Classification System (ASTM D-2487)		
	LA CENTER HOTEL SITE PARCEL #20970800, NW LA CENTER ROAD		
	PROJECT NO.	DATE	Figure A-4
	1171.009.G	4/16/24	

DRILLING COMPANY: Western States			RIG: Geoprobe		DATE: 2/17/24	
BORING DIAMETER: 3.0		DRIVE WEIGHT: 140#		DROP: 30"		ELEVATION: 240'±

DEPTH (FEET)	BAG SAMPLE	DRIVE SAMPLE BLOWS/FOOT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION BORING NO. B-#1
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
1					ML	Medium to gray-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT Becomes medium stiff to stiff Becomes sandier with depth Becomes wet to saturated
2	X	6				
3						
4	X	7				
5						
6						
7	X					
8						
9	X	9				
10						
11						
12						
13	X	8				
14						
15						
16						
17						
18	X	13				
19						
20						
21						
22						
23						
24						
25	X	18				
26						
27						
28						
29						
30						

BORING LOG		
PROJECT NO. 1171.009.G	LA CENTER HOTEL SITE	FIGURE NO. A-5

DRILLING COMPANY: Western States			RIG: Geoprobe		DATE: 2/17/24	
BORING DIAMETER: 3.0"		DRIVE WEIGHT: 140#		DROP: 30"		ELEVATION: 252' ±

DEPTH (FEET)	BAG SAMPLE	DRIVE SAMPLE BLOWS/FOOT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION BORING NO. B-#2
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
7	X	7			ML	Medium to gray-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT
12	X	12				Becomes stiff
15	X	15				
19	X	19				Becomes wet to saturated
						Total Depth = 21.5 feet Groundwater encountered at a depth of 20 to 21 feet at time of exploration

BORING LOG		
PROJECT NO. 1171.009.G	LA CENTER HOTEL SITE	FIGURE NO. A-6

DRILLING COMPANY: Western States

RIG: Geoprobe

DATE: 2/17/24

BORING DIAMETER: 3.0"

DRIVE WEIGHT: 140#

DROP: 30"

ELEVATION: 262'±

DEPTH (FEET)	BAG SAMPLE	DRIVE SAMPLE BLOWS/FOOT	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION BORING NO. B-#3
0					AC	<u>FILL</u> : Black, moist, moderately compacted, Asphalt Grindings
	X	6			ML/ SM	<u>FILL</u> : Dark gray-brown, very moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with organics and occasional gravel
5	X	7			ML	<u>NATIVE GROUND</u> : Dark brown, very moist, soft to medium stiff, organic, sandy, clayey SILT (Old Topsoil Zone)
					ML	Medium to reddish-brown, very moist, medium stiff, clayey, sandy SILT
10	X	11				Becomes medium stiff to stiff
15	X	15				
20	X	19				Becomes very moist to wet
						Total Depth = 21.5 feet No groundwater encountered at time of exploration
25						
30						

BORING LOG

PROJECT NO. 1171.009.G

LA CENTER HOTEL SITE

FIGURE NO. A-7

BACKHOE COMPANY: Inland Company

BUCKET SIZE: 24 inches

DATE: 2/06/24

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#1 ELEVATION 225'±
0					ML	Dark brown, wet, very soft, organic, sandy, clayey SILT (Topsoil)
	X			24.4	ML	Medium to gray-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT
5	X			22.7		
Total Depth = 7.0 feet No groundwater encountered at time of exploration						
10						
15						
TEST PIT NO. TH-#2 ELEVATION 240'±						
0					ML	Dark brown, wet, very soft, organic, sandy, clayey SILT (Topsoil)
	X			25.2	ML	Medium to gray-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT
5						
Total Depth = 6.0 feet No groundwater encountered at time of exploration						
10						
15						

LOG OF TEST PITS

PROJECT NO. 1171.009.G

LA CENTER HOTEL SITE

FIGURE NO. A-8

BACKHOE COMPANY: Inland Company

BUCKET SIZE: 24 inches

DATE: 2/06/24

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#3 ELEVATION 251'±
0					ML	Dark brown, wet, very soft, organic, sandy, clayey SILT (Topsoil)
X				24.0	ML	Medium to gray-brown with gray mottling, very moist, medium stiff, clayey, sandy SILT
5						
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						
TEST PIT NO. TH-#4 ELEVATION 257'±						
0					ML	Dark brown, wet, very soft, organic, sandy, clayey SILT (Topsoil)
X				22.6	ML	Medium to reddish-brown, moist to very moist medium stiff, clayey, sandy SILT
5	X			22.2		
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1171.009.G

LA CENTER HOTEL SITE

FIGURE NO. A-9

BACKHOE COMPANY: Inland Company

BUCKET SIZE: 24 inches

DATE: 2/06/24

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#5 ELEVATION 261'±
0					SM/ GM	<u>FILL</u> : Medium to dark gray brown, very moist, moderately compacted, silty, gravelly SAND to sandy GRAVEL with fragments of brick and concrete rubble
5					ML/ SM	<u>FILL</u> : Dark gray-brown, very moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with organics and occasional gravel
					ML	<u>NATIVE GROUND</u> : Dark brown, very moist, soft, organic, sandy, clayey SILT (Old Topsoil Zone)
10					ML	Medium to reddish-brown, very moist, medium stiff, clayey, sandy SILT
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

						TEST PIT NO. TH-#6 ELEVATION 261'±
0					AC	<u>FILL</u> : Black, moist, poorly compacted, Asphalt Grindings
					ML/ SM	<u>FILL</u> : Dark gray-brown, very moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with organics and occasional gravel
5	X			26.8	ML	<u>NATIVE GROUND</u> : Dark brown, very moist, soft, organic, sandy, clayey SILT (Old Topsoil Zone)
					ML	Medium to reddish-brown, very moist, medium stiff, clayey, sandy SILT
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1171.009.G

LA CENTER HOTEL SITE

FIGURE NO. A-10

BACKHOE COMPANY: Inland Company

BUCKET SIZE: 24 inches

DATE: 2/06/24

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#7 ELEVATION 260'±
0					AC	<u>FILL</u> : Black, moist, poorly compacted, Asphalt Grindings
					ML, SM	<u>FILL</u> : Medium to dark brown, moist to very moist, clayey, sandy SILT to silty SAND with organics and debris (i.e., brick and concrete concrete rubble
5					ML	<u>NATIVE GROUND</u> : Dark brown, very moist, soft, organic, sandy, clayey SILT (Old Topsoil Zone)
					ML	Medium to reddish-brown, very moist, medium stiff, clayey, sandy SILT
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO.						ELEVATION
0						
5						
10						
15						

LOG OF TEST PITS

PROJECT NO. 1171.009.G

LA CENTER HOTEL SITE

FIGURE NO. A-11

MAXIMUM DENSITY TEST RESULTS

SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#4 @ 2.5'	Medium to reddish-brown, clayey, sandy SILT (ML)	108.0	16.0

EXPANSION INDEX TEST RESULTS

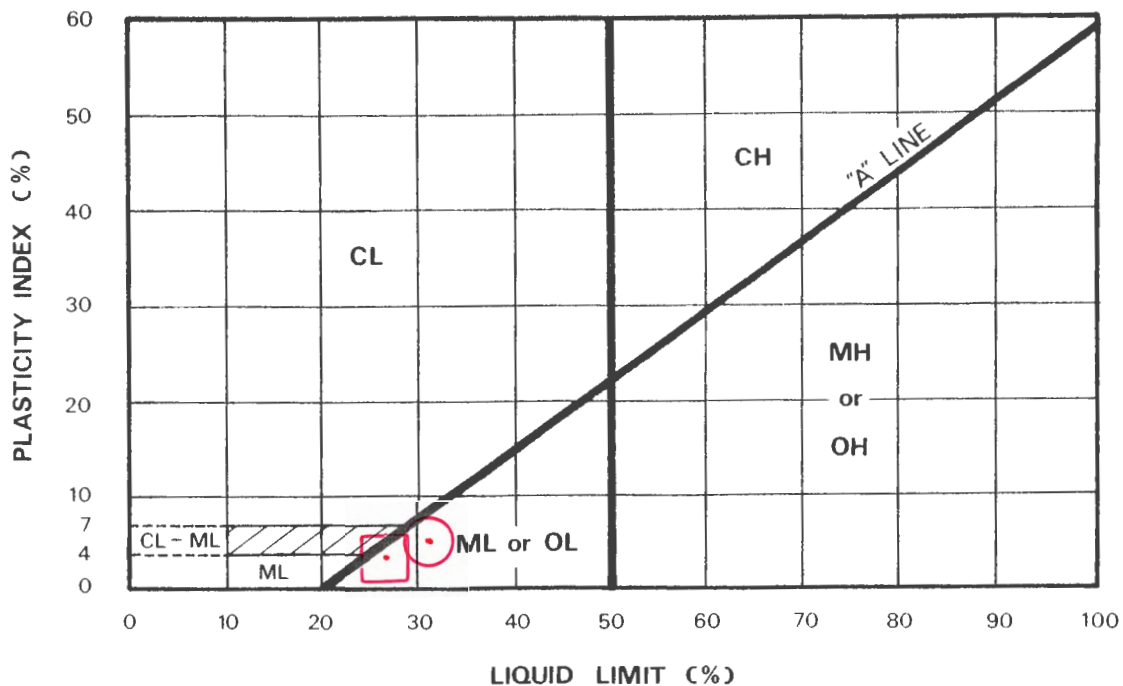
SAMPLE LOCATION	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (%)	EXPANSION INDEX	EXPANSIVE CLASS.

MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 1171.009.G

LA CENTER HOTEL SITE

FIGURE NO.: A-12



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
	TH-#1	2.0	24.4	27.7	3.9	76.2		ML
	TH-#4	2.5	22.6	30.4	6.2	76.4		ML

PLASTICITY CHART AND DATA

LA CENTER HOTEL SITE
PARCEL #209708000, NW LA CENTER ROAD

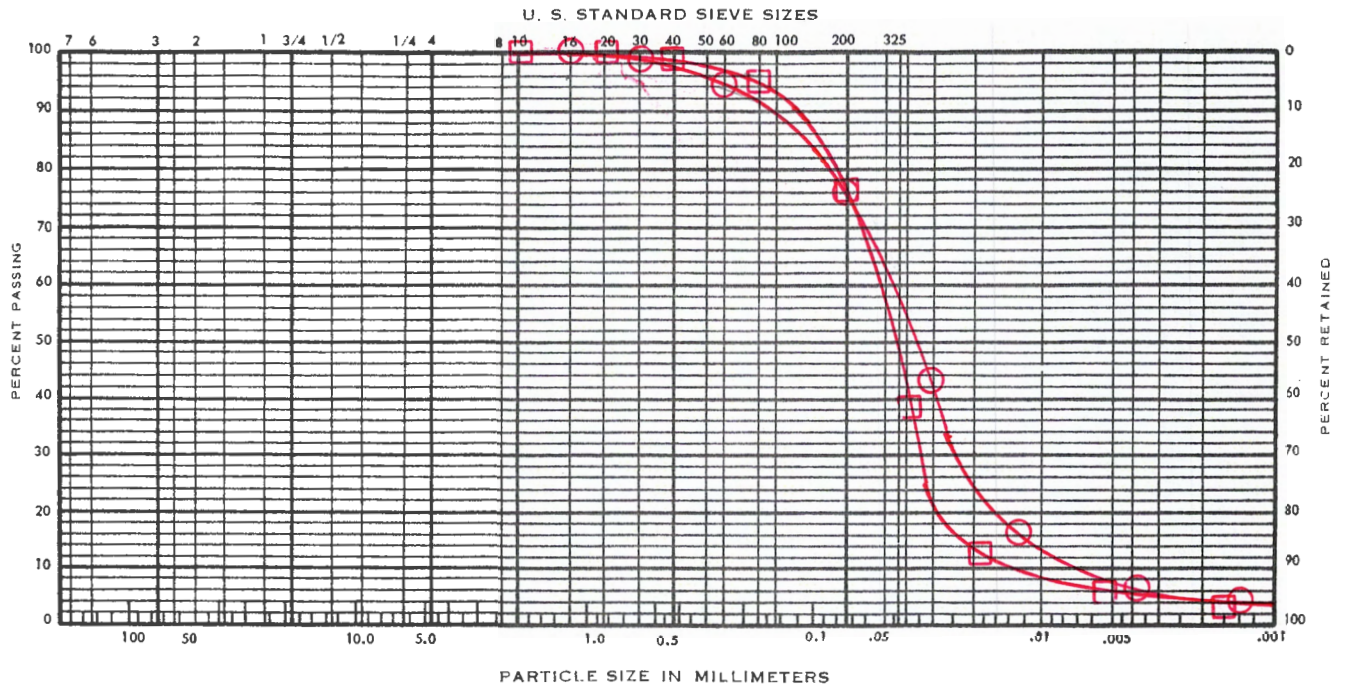
PROJECT NO.
1171.009.G

DATE
4/16/24

Figure A-13

UNIFIED SOIL CLASSIFICATION SYSTEM

(ASTM D 422-72)



COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	ELEV. (feet)	UNIFIED SOIL CLASSIFICATION SYMBOL	SAMPLE DESCRIPTION
	TH-#1	2.0		ML	Medium to gray-brown, clayey, sandy SILT
	TH-#4	2.5		ML	Medium to reddish-brown, clayey, sandy SILT

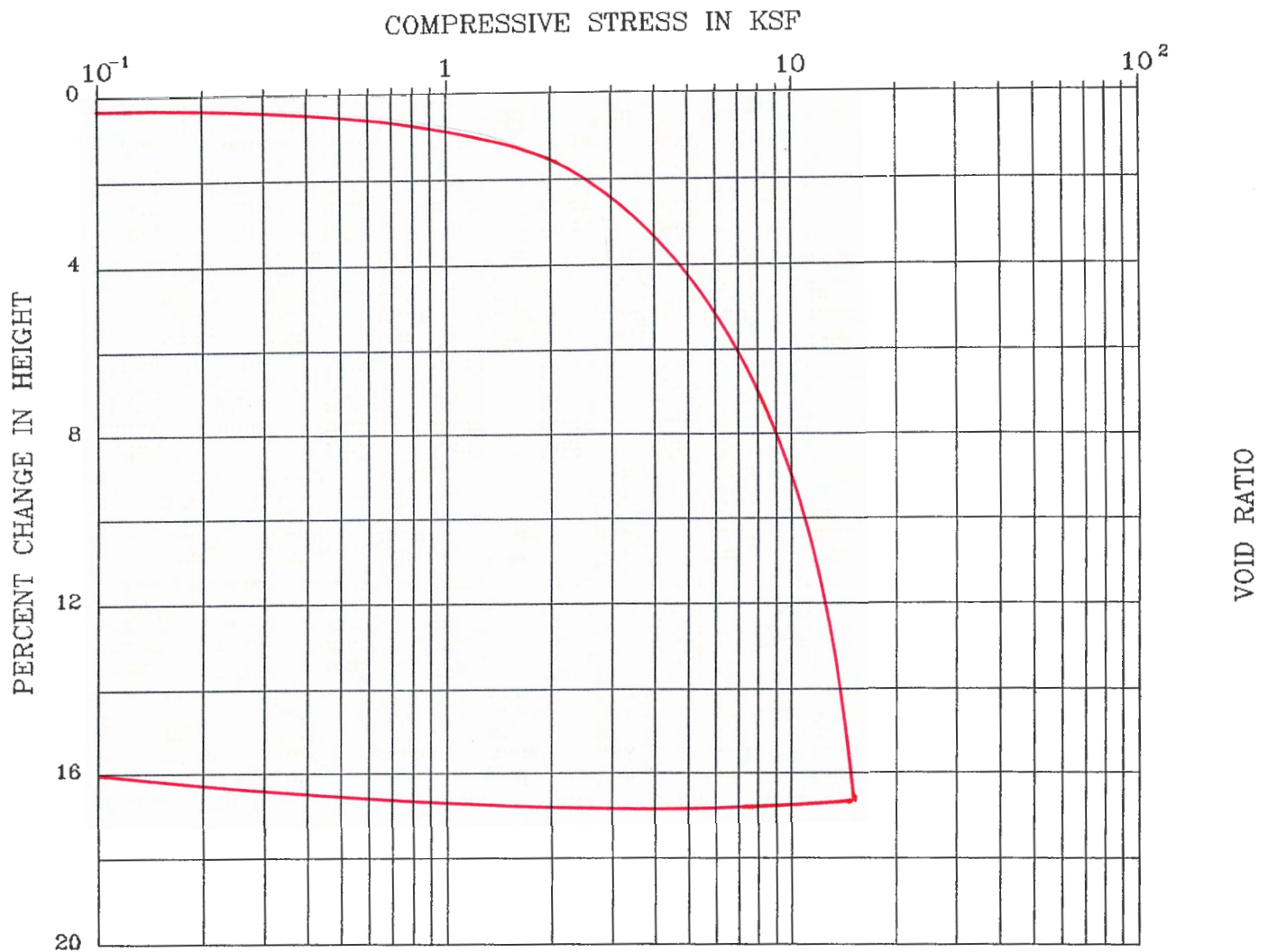


PO Box 20547 • PORTLAND, OREGON 97294

GRADATION TEST DATA

LA CENTER HOTEL SITE
PARCEL #209708000, NW LA CENTER ROAD

PROJECT NO.	DATE	FIGURE
1171.009.G	4/16/24	A-14



BORING : B-#1
 DEPTH (ft) : 3.5
 SPEC. GRAVITY : 2.5 (assumed)

DESCRIPTION : clayey, sandy SILT (ML)
 LIQUID LIMIT : 27.7
 PLASTIC LIMIT : 23.8

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	24.5	93.6	88.3	
FINAL	11.3	109.2	96.3	



PO Box 20547 • PORTLAND, OREGON 97294

CONSOLIDATION TEST DATA

LA CENTER HOTEL SITE
PARCEL #209708000, NW LA CENTER ROAD

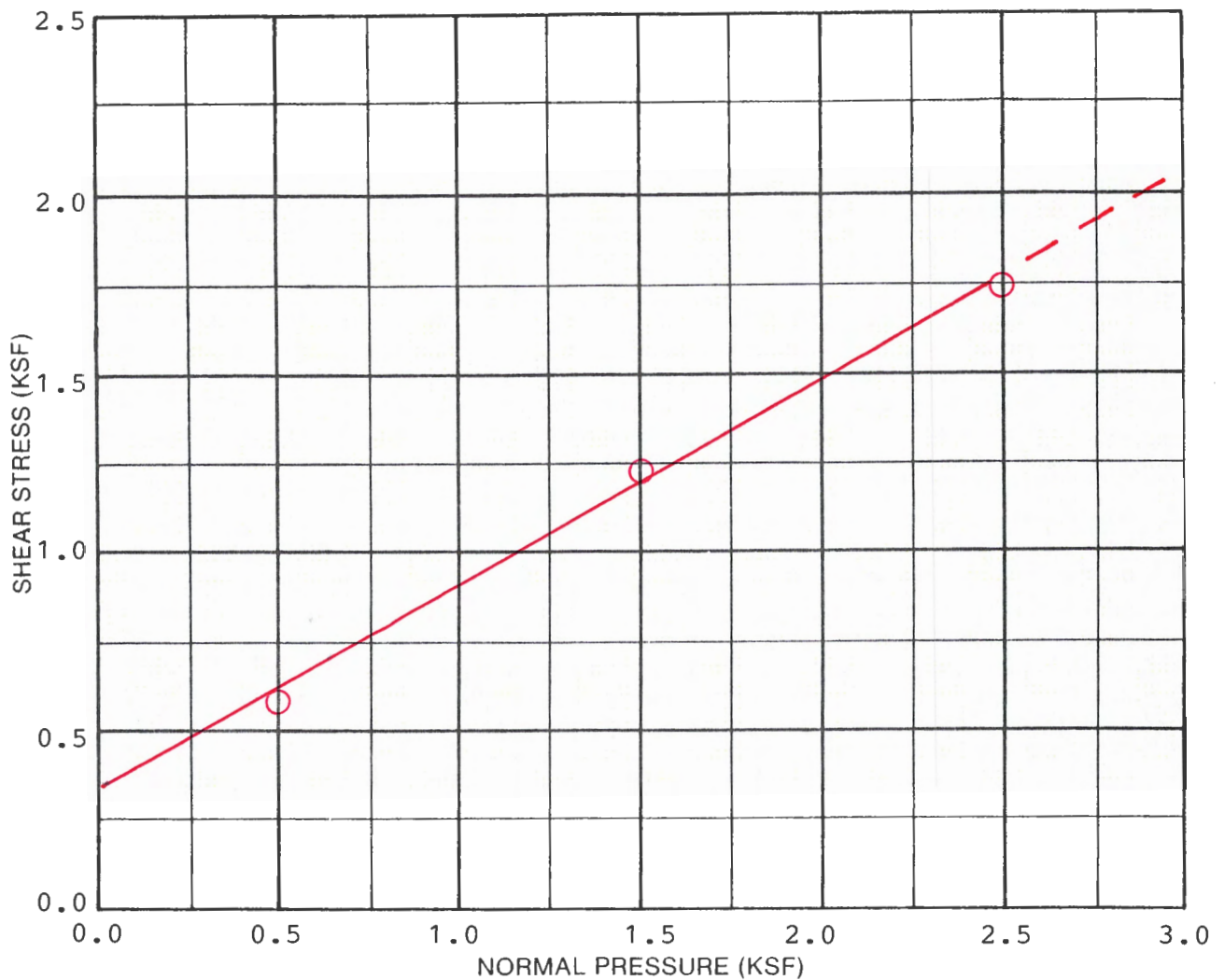
PROJECT NO.

DATE

1171.009.G

4/16/24

Figure A-15



SAMPLE DATA	
DESCRIPTION: Medium to gray-brown, clayey, sandy SILT (ML) (Remolded)	
BORING NO.: TH-#4	
DEPTH (ft.): 2.5	ELEVATION (ft.):
TEST RESULTS	
APPARENT COHESION (C): 350 psf	
APPARENT ANGLE OF INTERNAL FRICTION (ϕ): 28°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.6	1.2	1.7	
INITIAL H ₂ O CONTENT (%)	16.0	16.0	16.0	
FINAL H ₂ O CONTENT (%)	16.8	12.4	8.9	
INITIAL DRY DENSITY (PCF)	98.0	98.0	98.0	
FINAL DRY DENSITY (PCF)	98.7	104.1	108.9	
STRAIN RATE: 0.02 inches per minute				



PO Box 20547 • PORTLAND, OREGON 97294

DIRECT SHEAR TEST DATA

LA CENTER HOTEL SITE
PARCEL #209708000, NW LA CENTER ROAD

PROJECT NO.	DATE
1171.009.G	4/16/24

Figure A-16

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#6

SAMPLE DEPTH: 5.5 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	219	322	431
Expansion Dial (0.0001")	0	0	1
Expansion Pressure (psf)	0	0	3
Moisture Content (%)	24.6	19.4	14.1
Dry Density (pcf)	102.7	107.4	111.5
Resistance Value, "R"	15	29	41
"R"-Value at 300 psi Exudation Pressure = 28			

SAMPLE LOCATION:

SAMPLE DEPTH:

Specimen	A	B	C
Exudation Pressure (psi)			
Expansion Dial (0.0001")			
Expansion Pressure (psf)			
Moisture Content (%)			
Dry Density (pcf)			
Resistance Value "R"			
"R"-Value at 300 psi Exudation Pressure =			

Figure No. A-17

Field Infiltration Test Results

Location: Parcel #209708000	Date: March 6, 2024	Test Hole: TH-#1
Depth to Bottom of Hole: 4.0 feet	Hole Diameter: 6 inches	Test Method: Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E.		
Tester's Company: Redmond Geotechnical Services, LLC		Tester's Contact Number: 503-285-0598
Depth (feet)	Soil Characteristics	
0.0-1.0	Dark brown, sandy, clayey SILT (TOPSOIL)	
1.0-4.0	Medium to gray-brown, clayey, sandy SILT (ML)	

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
10:00	0	36.00	----		Filled w/12" water
10:10	10	36.25	0.25	1.50	
10:20	10	36.48	0.23	1.38	
10:30	10	36.70	0.22	1.32	
10:40	10	36.91	0.21	1.26	
10:50	10	36.12	0.21	1.26	
11:00	10	37.22	0.20	1.20	
11:10	10	37.42	0.20	1.20	
11:20	10	37.62	0.20	1.20	

Infiltration Test Data Table

APPENDIX E

Operations and Maintenance Manual