PRELIMINARY STORMWATER MANAGEMENT REPORT

FOR

CRATER AVENUE
at
4910 CRATER AVENUE
KEIZER, OREGON

November 22, 2024

PREPARED BY:

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APPENDICES

I. PURPOSE OF REPORT

This report describes the proposed improvements compliance with the City of Keizer's Design Standards (February 2023) – Chapter 400.

II. PROJECT DESCRIPTION

The site is located at 4910 Crater Avenue N. To the north and south the are private properties. The property will have one proposed two-story triplex with 2-car garage and 3-car parking garage located on the western side of the lot and a two story quadplex with 2-car garage and 2-car parking pad located on the eastern side of the lot.

A. EXISTING CONDITION

On the existing site, currently there is one single family residential home on the property. The site has a gravel driveway which leads to the single-family home. Trees can be seen on the east side of the property and one direct west of the residential home.

The existing site has a gentle slope that travels from west to east of the property. The site seems to have some infiltration into the ground.

The existing site is not located within the FEMA flood zone per FEMA flood map 41047C0331G, effective on 1/19/2000.

GEOTECHNICAL FINDINGS:

Geopacific Engineering prepared the Geotechnical Report, Project No. 24-6649, dated October 16, 2024, and concluded the following:

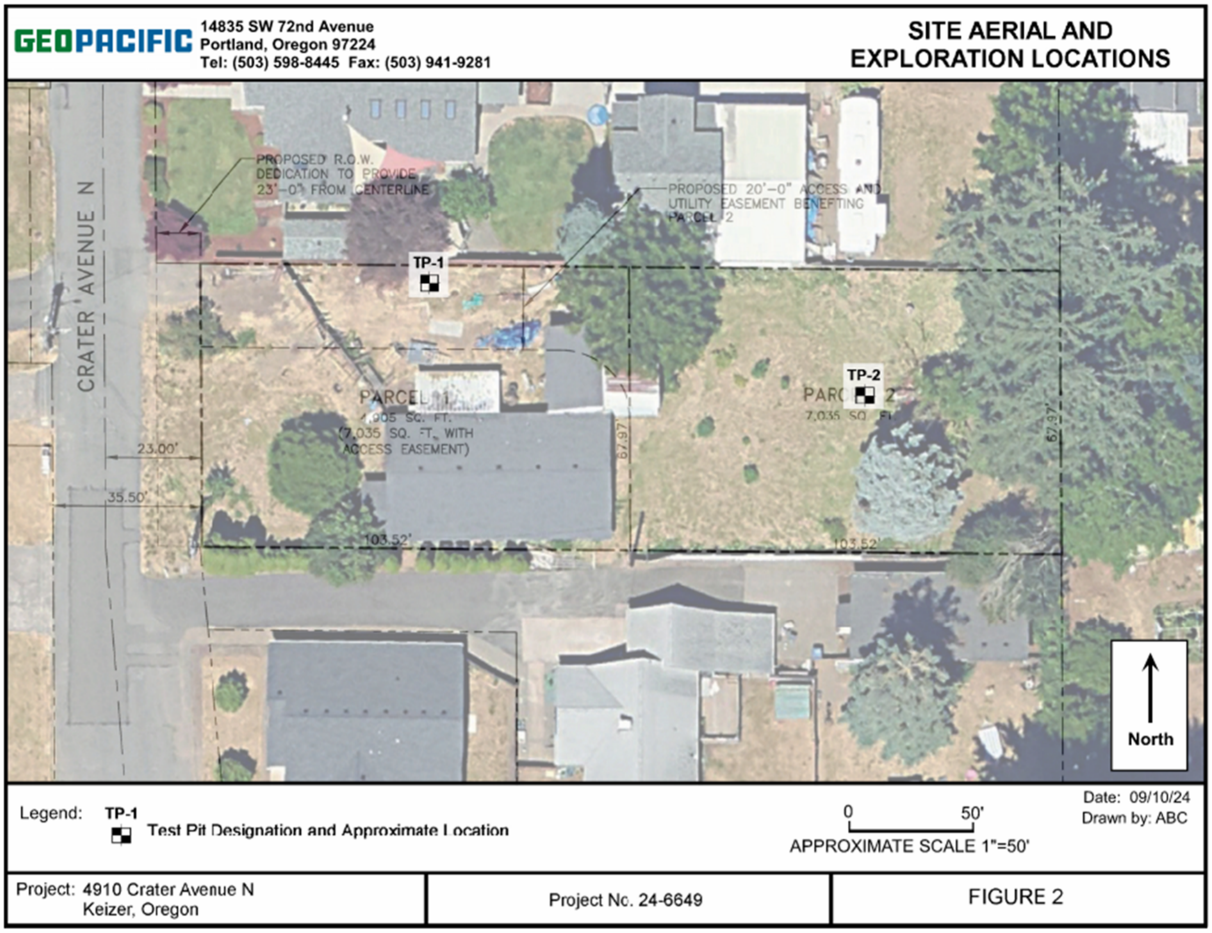
Groundwater was not encountered at an explored depth of 10', however, groundwater is anticipated at a depth of 20' to 40'.

Infiltration testing was performed at a depth of 4' and 8', with a resulting unfactored rate of 2.88 in/hr and 2.16 in/hr. As a result of these findings, infiltration is considered feasible.

Table 1 - Summary of Infiltration Test Results

Test Location	Depth (feet)	Soil Type	Infiltration Rate (in/hr)
TP-2	4.0	Sandy Lean CLAY (CL)	2.88
TP-2	8.0	Poorly Graded SAND with Silt (SP)	2.16

Boring Log Map – Geotechnical Report



B. PROPOSED CONDITION

The proposed development will be divided into two parcels, with a proposed 20' access drive aisle and proposed stormwater improvements. Additionally, Crater Avenue will be improved to it's ultimate right of way, with a 10.5' dedication, new curb, gutter, sidewalk, and parkway, as well as two proposed driveways.

The proposed drainage pattern will generally follow the existing site's gentle slope which travels west to east of the property. The raingarden infiltration systems will be in the southeast and southern portion of the lots. The rain gardens have been sized to fully infiltrate the 100-year storm event. The raingarden infiltration system has been sized for the ultimate future build out design. Additionally, a perimeter curb was placed along the property line where the top of curb elevation is set above the lowest existing street grade elevation, to allow for secondary overland release.

III. METHODOLOGY

The City of Keizer's Design Standards (February 2023) – Chapter 400 were implemented for the design of the onsite stormwater system, as follows:

Projects greater than 5,000 square feet of new or replaced impervious surface are required to meet the full requirements for treatment, flow control, and retention of stormwater as provided below. This proposed project exceeds this new or replaced 5,000 square feet of impervious area.

Stormwater Treatment:
The entire WQE will be required to retain and treat and shall conform to NPDES, TMDL and WPCF requirements and reduce the discharge of the listed pollutants to the Waters of the State. All treatment facilities will be designed to utilize the GSI to the MEF.

Stormwater Retention:
The hierarchy to be followed in determining project specific applicable facility retention requirements based on the Design Infiltration Rates for the site or the Point of Connection as follows:

1. **Design Infiltration Rate greater than 2 inches per hour:**
   The project facility shall retain and treat the entire WQE. The project facility shall retrain all stormwater runoff from design storm events up to and including 100-year design storm event with no released allowed.
2. **Project is in an Unserved Stormwater Area (regardless of design infiltration rate):**
   The project facility shall retain and treat the entire WQE. The project facility shall retrain all stormwater runoff from design storm events up to and including 100-year design storm event with no released allowed.
3. **Design Infiltration Rate between 0.75 inches and 2 inches per hour:**
   The facility shall retrain and treat the entire WQE. In addition, the facility shall retain stormwater runoff for the 5-year, 10-year, 25-year design storm events with an allowable release rate up to the predeveloped 5-year design storm event. Runoff for the 50-year and 100-year design storm events shall be retained with an allowable release rate up to the predeveloped 25-year design storm event.
4. **Design Infiltration Rate less than 0.75 inches per hour:**
   The facility shall retain and treat the entire WQE to the MEF. The facility shall also retrain stormwater runoff for the 5-year, 10-year, 25-year, 50-year, and 100-year design storm events, not allowing any increase in runoff for all storm events listed.
5. **"Critical Basin" Point of Connection (regardless of design infiltration rate):**
   The facility shall retrain and treat the entire WQE to the MEF. The facility shall also retain stormwater runoff for the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year design storm events, not allowing any increase in runoff for all storm events listed.

Flow Control Requirements:

To meet the requirement to retain all stormwater runoff to the MEF, certain sites may need to include flow control to be implemented as part of the design. In other situations, it simply benefits the overall system to provide flow control prior to treatment or retention systems.

GSI FACILITY PLANTING REQUIREMENTS:

SOILS/GROWING MEDIUM REQUIREMENTS

The soil and growing medium installed in the stormwater facility shall meet the following requirements:

1. The minimum depth of the growing medium or amended soil shall be per design – minimum depth is 12 inches.
2. The growing medium or amended soil should be a mix of loamy soil, sand, and compost (30 – 40 percent compost, by volume), and shall be loose, friable, well-mixed, homogenous, free of wood pieces, plastic, and other foreign matter, and have no visible free water when placed in the facility. The pH of the mix shall be between 5 and 8.
3. The final infiltration rate of the bottom of the facility must be tested and confirmed to be equal to or greater than the Design Infiltration Rate for the facility.
4. After planting, the remaining areas of the facility shall be surfaced with 2 to 3 inches of either 1-1/2”-3/4” clean round rock (allowed throughout the facility) and/or hardwood chips (allowed only above the high-water level in the facility).
5. Weed-free certification is required for all imported growing medium, soil, surface material, and seed mixes.

FACILITY PLANTING CALENDAR

To ensure the best chances of successful plant establishment, all planting should take place between October 15th and May 15th; unless regular watering is provided to ensure the plantings are viable in drier months. In addition, air temperatures during planting must be between 32- and 90-degrees Fahrenheit.

IRRIGATION REQUIREMENTS

In-ground automated irrigation systems are required for all GSI facilities and must be designed and installed to meet these requirements:

1. Water the entire plant area of the facility with 1 inch of water per week through from the beginning of July through the end October, throughout the warranty period, to establish the facility plantings.
2. Water infrequently but deeply to help the plants become as drought tolerant as possible.
3. Continued irrigation after the establishment period is at the discretion of the owner.

WARRANTY MAINTENANCE PERIOD

All new public GSI stormwater facilities will be subject to a warranty period including a performance security. These facilities must be maintained to ensure the facility is functioning properly. To successfully complete the warranty period the facility must meet the following conditions:

1. At least 80% of the plants (percentage of cover for grasses, sedges, groundcover and perennials; percentage of trees and shrubs by count) must be alive and in good health.
2. The facility must be free of weeds and invasive plants (as defined by Marion County Soil and Water Conservation District).
3. The facility must be free of trash, debris, and excess dead foliage or clippings.
4. All inlets and outlets shall be clear and operational, without erosion or channelization throughout or downstream of the facility.

Any conditions not met will be required to be remediated prior to release of the performance security.

IV. CALCULATIONS

The development will be designed in accordance with the Design Standards in Division 004, Appendix D. The Santa Barbara Urban Hydrograph (SBUH) method will be the selected methodology used in the computer program HydroCAD Version 10.20. The following parameters were inputted;

Storm Type: Type 1A Rainfall Distribution
Soil Group: Group B

Curve Number:

CURVE NUMBERS	
Pre-Development	**CN**
Brush, Poor	67
Gravel Surface	96
Impervious	98

CURVE NUMBERS	
Post-Development	**CN**
Grass Cover, Fair	69
Impervious	98

Rainfall Depth:

Return Interval	Peak 24-Hour Rainfall
Water Quality Storm Event	1.38 inches
2-YR Storm Event	2.20 inches
5-YR Storm Event	2.70 inches
10-YR Storm Event	3.20 inches
25-YR Storm Event	3.60 inches
50-YR Storm Event	4.10 inches
100-YR Storm Event	4.40 inches

V. SUMMARY

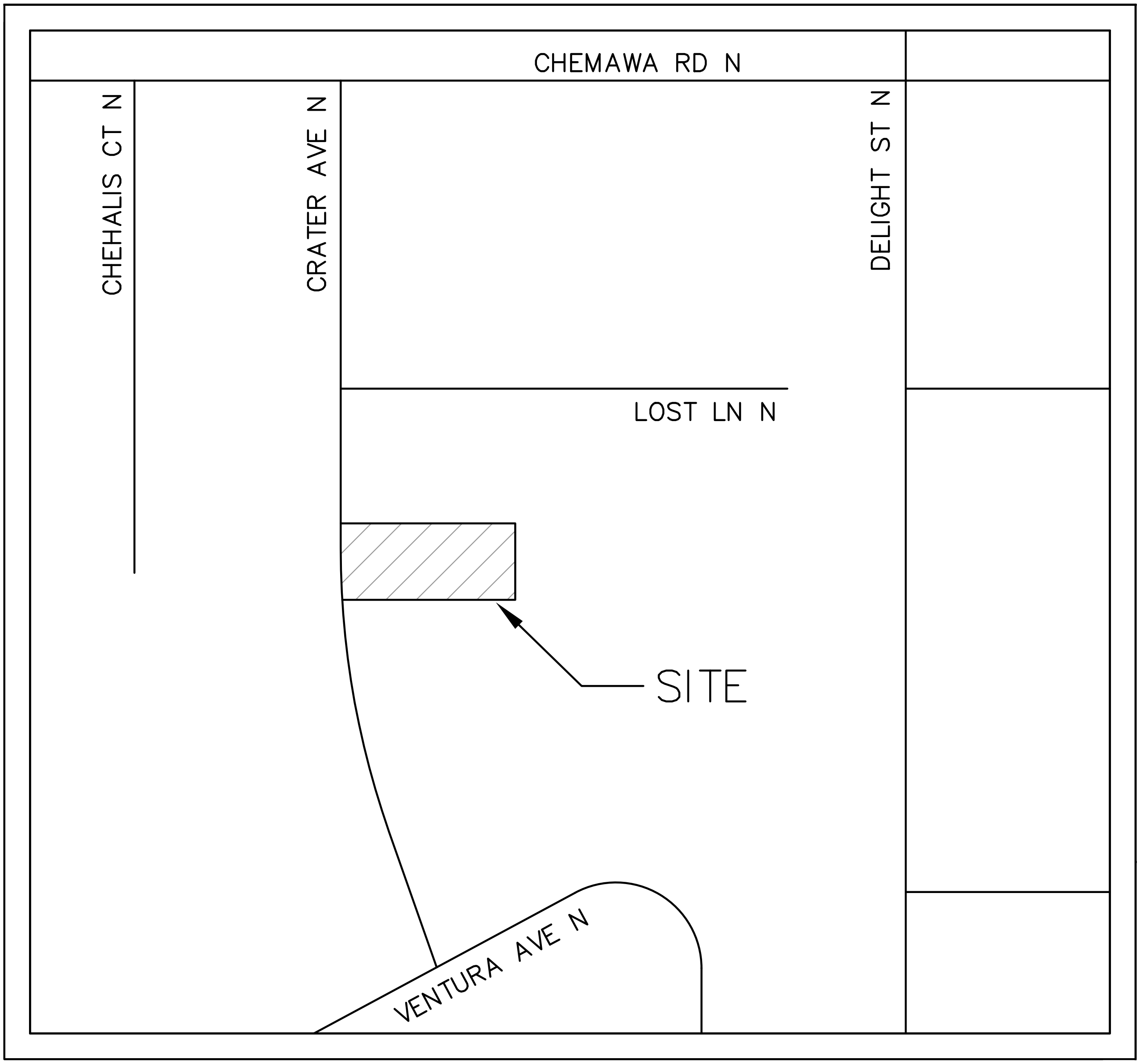
In conclusion, a proposed infiltration planter on each lot. The infiltration planters has been sized to handle the entire 100-year storm event with no overflow. Below is a summary of calculations. Additionally, a perimeter curb was placed along the property line where the top of curb elevation is set above the lowest existing street grade elevation, to allow for secondary overland release.

CATCHMENT AND FACILITY TABLE						
CATCHMENT/ FACILITY ID	TOTAL AREA (SF)/(AC.)	IMPERVIOUS AREA (SF)	PERVIOUS AREA (SF)	OWNERSHIP (PRIVATE/ PUBLIC)	FACILITY TYPE	FACILITY SIZE (BOTTOM) SF
LOT A	9,045	7,236	1,809	PRIVATE	RAIN-GARDEN INFILTRATION	600
LOT B	5,000	4,000	1,000	PRIVATE	RAIN-GARDEN INFILTRATION	450
TOTAL ONSITE	**14,045**	**11,236**	**2,809**			

WATER QUALITY TREATMENT REQUIREMENTS				
CATCHMENT/ FACILITY ID	WQV (IN)	WQV (CF)	80% OF WQV	RAIN GARDEN ALLOWABLE VOLUME
A	1.38	741	592	1,080
B	2.38	392	314	810

PRE VS. POST CONSTRUCTION FLOW RATES		
FACILITY ID	PEAK FLOW RATE (CFS)	
	100 YEAR STORM	
PROJECT SITE	POST (NO BMP)	POST (W/BMP)
LOT A	0.19	0
LOT B	0.11	0

APPENDIX A – MAPS

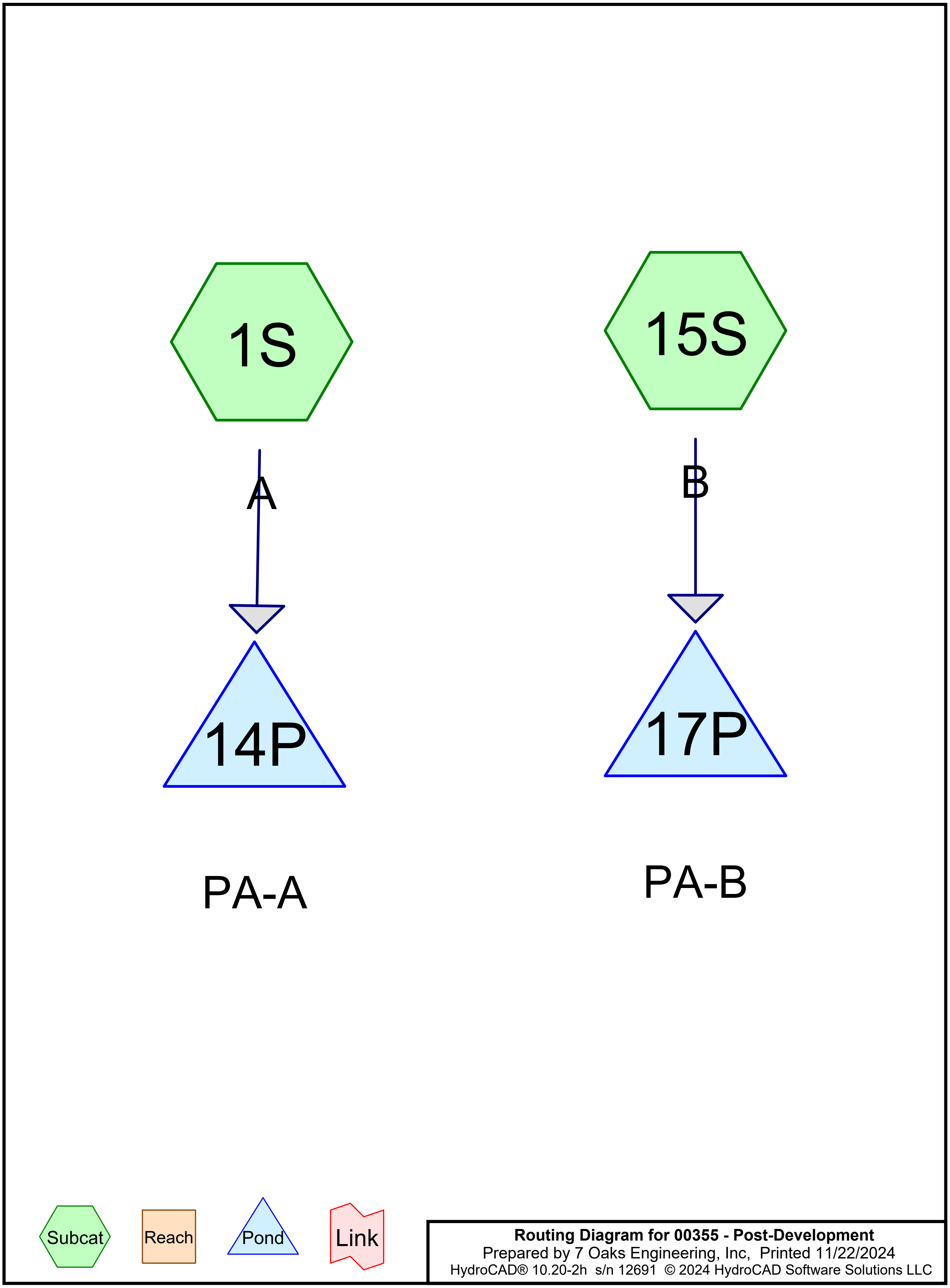


VICINITY MAP
NOT TO SCALE

APPENDIX B – CALCULATIONS

POST DEVELOPMENT HYDROCAD

THE ELEVATIONS SHOWN HEREIN ARE ARBITARY AND USED FOR CALCULATION PURPOSES ONLY



Rainfall Events Listing (selected events)

Event#	Event Name	Storm Type	Curve	Mode	Duration (hours)	B/B	Depth (inches)	AMC
1	100-Yr	Type IA 24-hr		Default	24.00	1	4.40	2

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
0.064	79	<50% Grass cover, Poor, HSG B (1S, 15S)
0.258	98	Paved parking, HSG B (1S, 15S)
0.322	**94**	**TOTAL AREA**

Soil Listing (all nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
0.322	HSG B	1S, 15S
0.000	HSG C	
0.000	HSG D	
0.000	Other	
0.322		**TOTAL AREA**

Ground Covers (all nodes)

HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
0.000	0.064	0.000	0.000	0.000	0.064	<50% Grass cover, Poor	1S, 15S
0.000	0.258	0.000	0.000	0.000	0.258	Paved parking	1S, 15S
0.000	**0.322**	**0.000**	**0.000**	**0.000**	**0.322**	**TOTAL AREA**	

Time span=0.01-48.00 hrs, dt=0.01 hrs, 4800 points
Runoff by SBUH method, Split Pervious/Imperv.
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment1S: A Runoff Area=9,045 sf 80.00% Impervious Runoff Depth=3.79"
Tc=5.0 min CN=79/98 Runoff=0.19 cfs 0.066 af

Subcatchment15S: B Runoff Area=5,000 sf 80.00% Impervious Runoff Depth=3.79"
Tc=5.0 min CN=79/98 Runoff=0.11 cfs 0.036 af

Pond 14P: PA-A Peak Elev=103.98' Storage=1,067 cf Inflow=0.19 cfs 0.066 af
Outflow=0.02 cfs 0.066 af

Pond 17P: PA-B Peak Elev=103.13' Storage=420 cf Inflow=0.11 cfs 0.036 af
Outflow=0.02 cfs 0.036 af

Total Runoff Area = 0.322 ac Runoff Volume = 0.102 af Average Runoff Depth = 3.79"
20.00% Pervious = 0.064 ac 80.00% Impervious = 0.258 ac

Summary for Subcatchment 1S: A

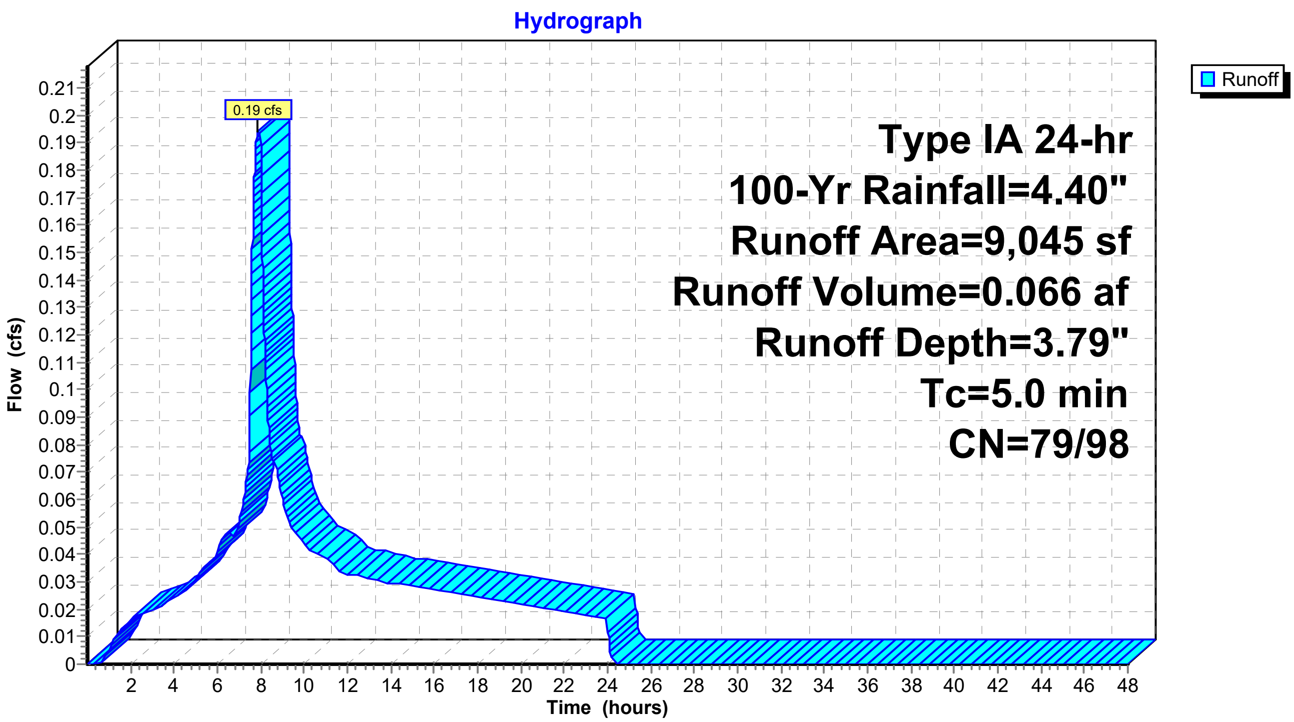
Runoff = 0.19 cfs @ 7.89 hrs, Volume= 0.066 af, Depth= 3.79"
Routed to Pond 14P : PA-A

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.01-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-Yr Rainfall=4.40"

Area (sf)	CN	Description
7,236	98	Paved parking, HSG B
1,809	79	<50% Grass cover, Poor, HSG B
9,045	94	Weighted Average
1,809	79	20.00% Pervious Area
7,236	98	80.00% Impervious Area

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

Subcatchment 1S: A



Summary for Subcatchment 15S: B

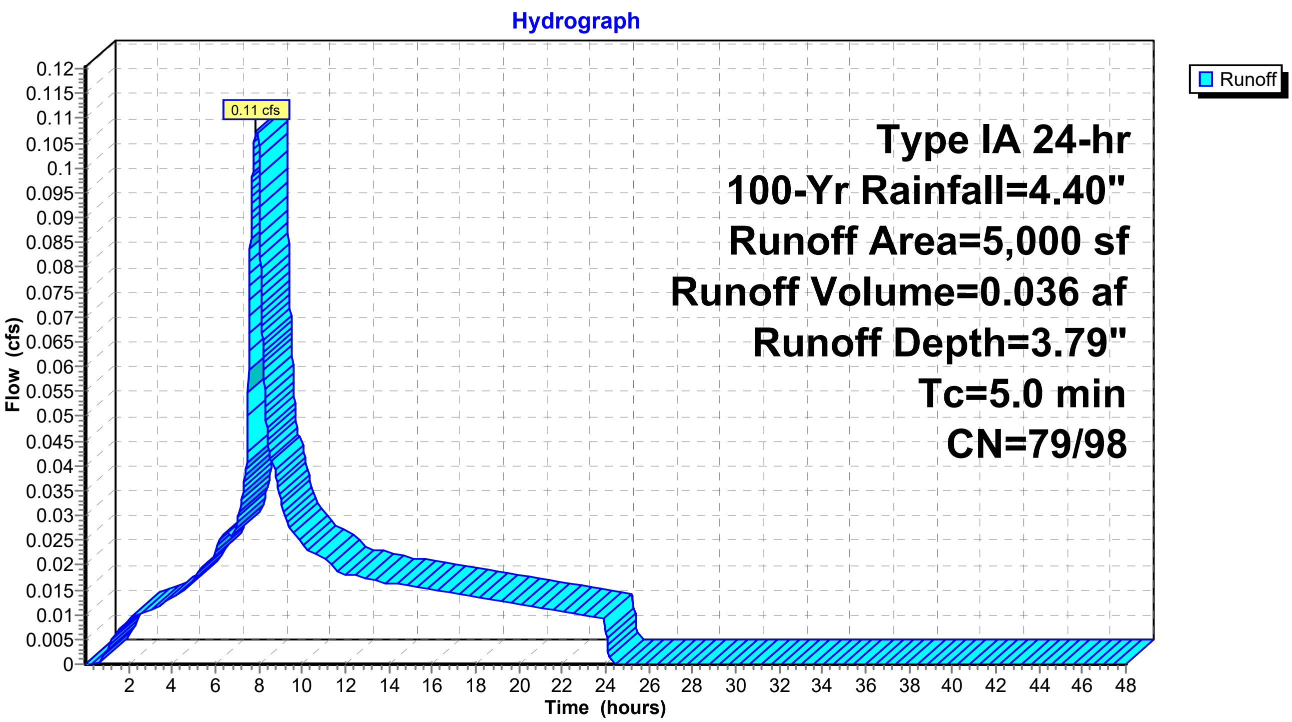
Runoff = 0.11 cfs @ 7.89 hrs, Volume= 0.036 af, Depth= 3.79"
Routed to Pond 17P : PA-B

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.01-48.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-Yr Rainfall=4.40"

Area (sf)	CN	Description
4,000	98	Paved parking, HSG B
1,000	79	<50% Grass cover, Poor, HSG B
5,000	94	Weighted Average
1,000	79	20.00% Pervious Area
4,000	98	80.00% Impervious Area

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

Subcatchment 15S: B



Summary for Pond 14P: PA-A

Inflow Area = 0.208 ac, 80.00% Impervious, Inflow Depth = 3.79" for 100-Yr event
Inflow = 0.19 cfs @ 7.89 hrs, Volume= 0.066 af
Outflow = 0.02 cfs @ 18.59 hrs, Volume= 0.066 af, Atten= 88%, Lag= 642.0 min
Discarded = 0.02 cfs @ 18.59 hrs, Volume= 0.066 af

Routing by Stor-Ind method, Time Span= 0.01-48.00 hrs, dt= 0.01 hrs / 2
Peak Elev= 103.98' @ 18.59 hrs Surf.Area= 600 sf Storage= 1,067 cf

Plug-Flow detention time= 469.6 min calculated for 0.066 af (100% of inflow)
Center-of-Mass det. time= 469.6 min (1,143.5 - 673.9)

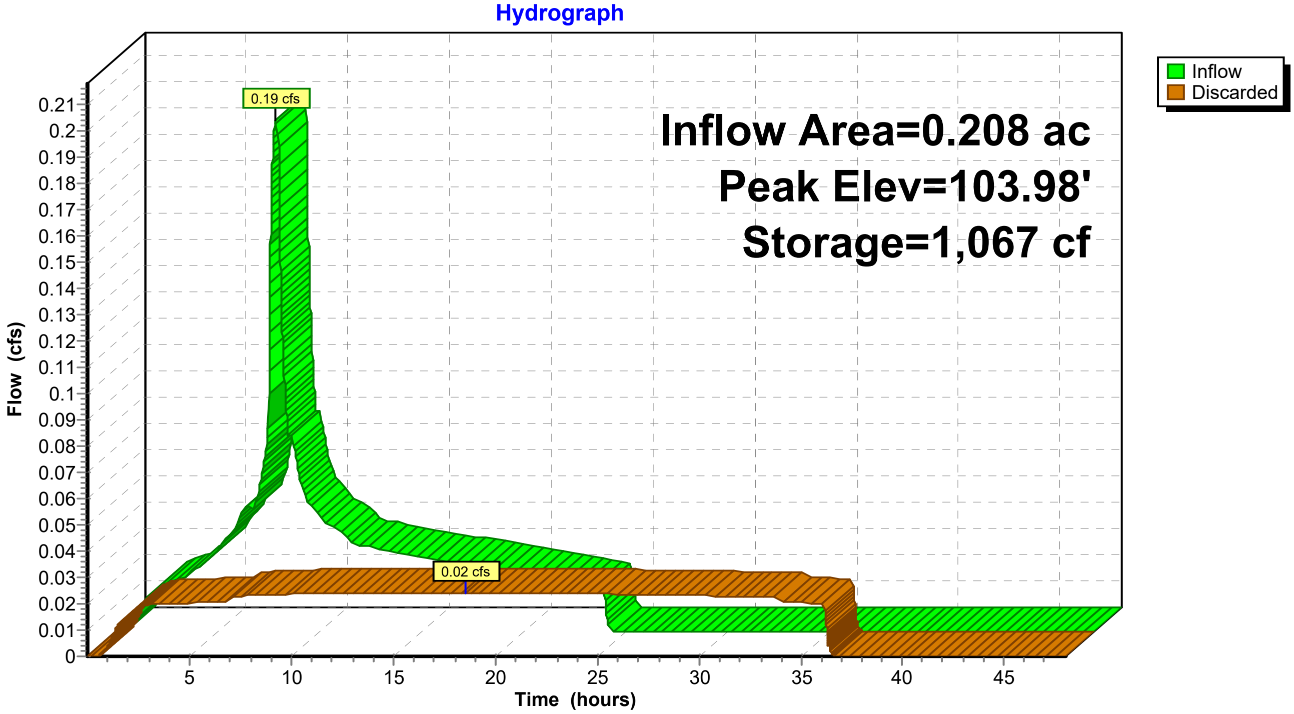
Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	1,080 cf	**Custom Stage Data (Irregular)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Perim. (feet)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
100.00	600	150.0	0.0	0	0	600
101.00	600	150.0	30.0	180	180	750
102.50	600	150.0	0.0	0	180	975
104.00	600	150.0	100.0	900	1,080	1,200

Device	Routing	Invert	Outlet Devices
#1	Discarded	100.00'	**1.440 in/hr Exfiltration over Horizontal area** Conductivity to Groundwater Elevation = 80.00'

Discarded OutFlow Max=0.02 cfs @ 18.59 hrs HW=103.98' (Free Discharge)
└─**1=Exfiltration** (Controls 0.02 cfs)

Pond 14P: PA-A


Hydrograph
Inflow
Discarded
0.19 cfs
0.02 cfs
Inflow Area=0.208 ac
Peak Elev=103.98'
Storage=1,067 cf
Flow (cfs)
0.21
0.2
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
5
10
15
20
25
30
35
40
45
Time (hours)

Summary for Pond 17P: PA-B

Inflow Area = 0.115 ac, 80.00% Impervious, Inflow Depth = 3.79" for 100-Yr event
Inflow = 0.11 cfs @ 7.89 hrs, Volume= 0.036 af
Outflow = 0.02 cfs @ 13.31 hrs, Volume= 0.036 af, Atten= 84%, Lag= 325.6 min
Discarded = 0.02 cfs @ 13.31 hrs, Volume= 0.036 af

Routing by Stor-Ind method, Time Span= 0.01-48.00 hrs, dt= 0.01 hrs / 2
Peak Elev= 103.13' @ 13.31 hrs Surf.Area= 450 sf Storage= 420 cf

Plug-Flow detention time= 256.0 min calculated for 0.036 af (100% of inflow)
Center-of-Mass det. time= 256.1 min (930.0 - 673.9)

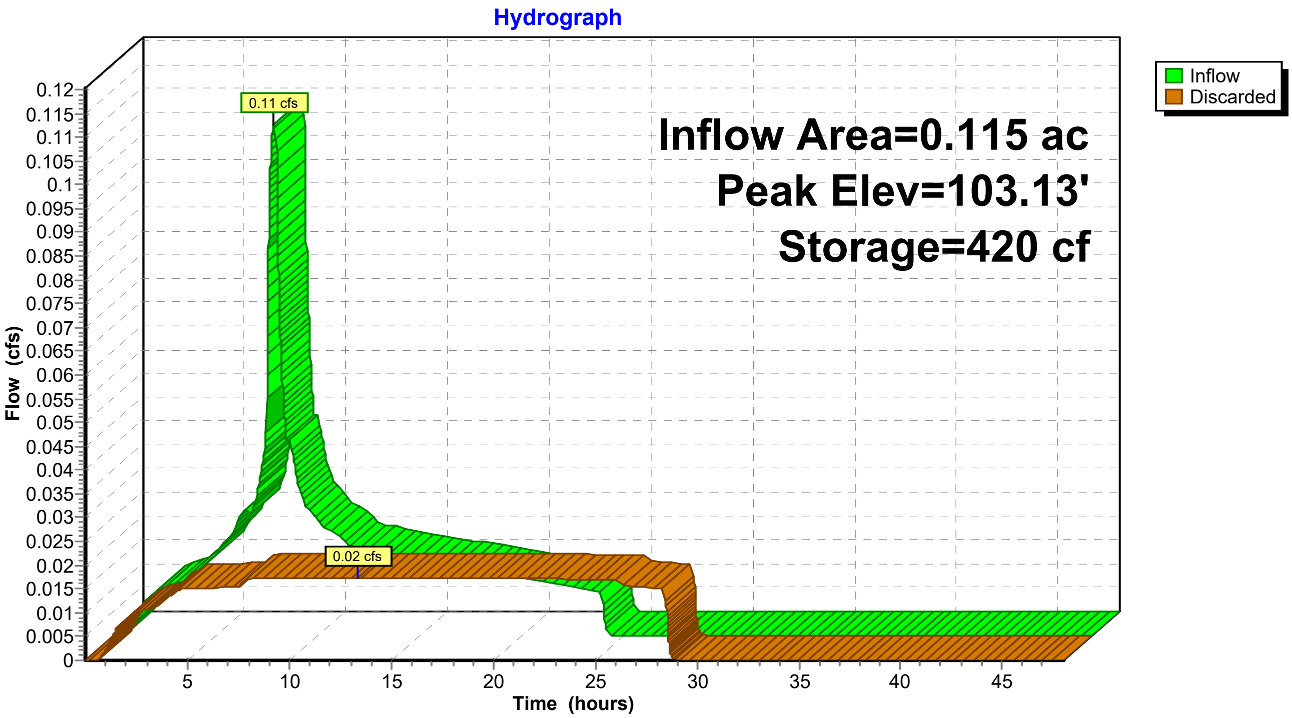
Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	810 cf	**Custom Stage Data (Irregular)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Perim. (feet)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
100.00	450	150.0	0.0	0	0	450
101.00	450	150.0	30.0	135	135	600
102.50	450	150.0	0.0	0	135	825
104.00	450	150.0	100.0	675	810	1,050

Device	Routing	Invert	Outlet Devices
#1	Discarded	100.00'	**1.440 in/hr Exfiltration over Horizontal area** Conductivity to Groundwater Elevation = 80.00'

Discarded OutFlow Max=0.02 cfs @ 13.31 hrs HW=103.13' (Free Discharge)
└─**1=Exfiltration** (Controls 0.02 cfs)

Pond 17P: PA-B


Hydrograph
Inflow
Discarded
0.11 cfs
0.02 cfs
Inflow Area=0.115 ac
Peak Elev=103.13'
Storage=420 cf
Flow (cfs)
Time (hours)
0.12
0.115
0.11
0.105
0.1
0.095
0.09
0.085
0.08
0.075
0.07
0.065
0.06
0.055
0.05
0.045
0.04
0.035
0.03
0.025
0.02
0.015
0.01
0.005
0
5
10
15
20
25
30
35
40
45

APPENDIX C – PLANS

PRELIMINARY DEVELOPMENT PLANS

AT 4910 CRATER AVENUE
KEIZER, OR.

PLANNING:
CASCADIA PLANNING + DEVELOPMENT SERVICES
PO BOX 1920
SILVERTON, OREGON 97381
503.804.1089
STEVE@CASCADIAPD.COM

CIVIL ENGINEER:
7 OAKS ENGINEERING, INC.
KIM JOHNSON, P.E.
345 WESTFIELD ST. #107
SILVERTON, OR. 97381
503.308.8554
KIM@7OAKSENGINEERING.COM

SURVEY:
BARKER SURVEYING
3657 KASHMIR WAY SE
SALEM, OR. 97137
503.588.8800
GREG@BARKERWILSON.COM

UTILITY PURVEYORS:

WATER:
CITY OF KEIZER
930 CHEMAWA RD.
KEZIER, OR. 97303

STORM DRAIN:
CITY OF KEIZER–PUBLIC WORKS
930 CHEMAWA RD.
KEZIER, OR. 97303

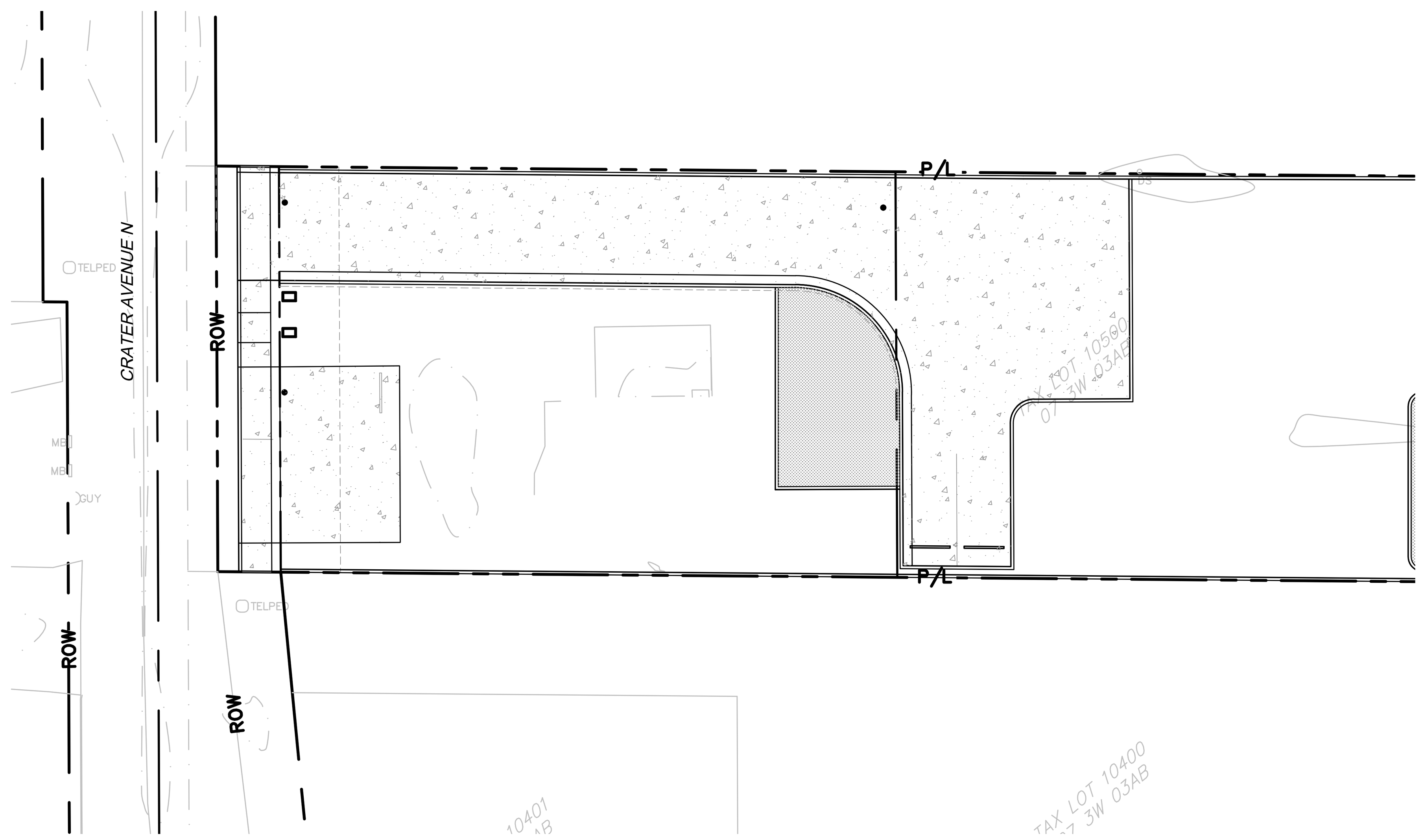
SEWER:
CITY OF SALEM
555 LIBERTY STREET SE
SALEM, OREGON.
503.588.6311

FIRE:
CITY OF KEIZER
930 CHEMAWA RD.
KEZIER, OR. 97303

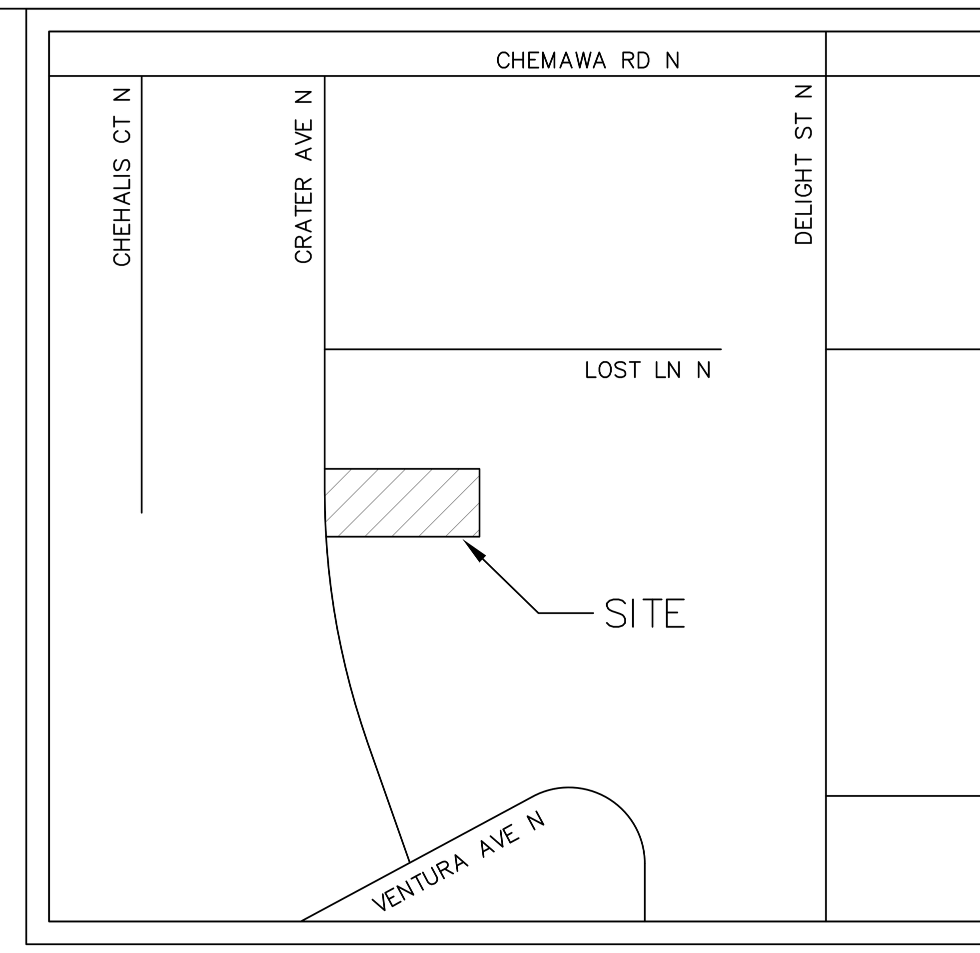
ELECTRIC:
PORTLAND GENERAL ELECTRIC
KEN SPENCER
KENNETH.SPENCER@PGN.COM
503.970.7200

NATURAL GAS:
NORTHWEST NATURAL GAS COMPANY
3123 BROADWAY ST NE
SALEM, OR. 97303
503.585.6611

ROADWAYS:
CITY OF KEIZER–PUBLIC WORKS
930 CHEMAWA RD.
KEZIER, OR. 97303



INDEX MAP
SCALE:1":20'



VICINITY MAP
NOT TO SCALE

GENERAL NOTES:

1. ALL CONSTRUCTION, MATERIALS, AND WORKMANSHIP SHALL CONFORM TO THE LATEST STANDARDS AND PRACTICES OF THE CITY OF KEIZER, THE OREGON STRUCTURAL SPECIALITY CODE (BUILDING CODE), OREGON PLUMBING SPECIALITY (PLUMBING CODE), AND THE OREGON FIRE CODE (FIRE CODE), LATEST EDITIONS.
2. ALL PERMIT AND LICENSES NECESSARY FOR THE EXECUTION AND COMPLETION OF THE WORK SHALL BE SECURED BY THE CONTRACTOR PRIOR TO COMMENCING CONSTRUCTION.
3. ALL EXCAVATORS MUST COMPLY WITH THE RULES ADOPTED BY THE OREGON UTILITY NOTIFICATION CENTER, INCLUDING NOTIFICATION OF ALL OWNERS OF UNDERGROUND UTILITIES AT LEAST 48 BUSINESS HOURS, BUT NOT MORE THAN 10 BUSINESS DAYS, BEFORE COMMENCING AN EXCAVATION. THOSE RULES ARE SET FORTH IN OAR 952–001–0010 THROUGH OAR 952–001–0090 AND ORS 757.541 TO 757.57. THE TELEPHONE NUMBER FOR THE OREGON UTILITY NOTIFICATION CENTER IS 503.232.1987 AND THE LOCAL "CALL 48 HOURS BEFORE YOU DIG NUMBER" IS 503.246.6699.
4. THE LOCATION OF EXISTING UNDERGROUND UTILITIES SHOWN ON THE PLAN IS FOR INFORMATION ONLY AND IS NOT GUARANTEED TO BE ACCURATE. CONTRACTOR SHALL VERIFY ELEVATIONS OF ALL UNDERGROUND UTILITY CONNECTION POINTS PRIOR TO COMMENCING WITH CONSTRUCTION AND SHALL BRING ANY DISCREPANCIES TO THE ATTENTION OF 7 OAKS ENGINEERING, INC. POTHOLE ALL CROSSINGS AS NECESSARY BEFORE CONSTRUCTION TO PREVENT GRADE AND ALIGNMENT CONFLICTS.
5. 7 OAKS ENGINEERING, INC. ASSUMES NO RESPONSIBILITY FOR ANY DISCREPANCIES ENCOUNTERED BETWEEN THE CURRENT FIELD CONDITIONS AND THE INFORMATION SHOWN ON THE SURVEY MAP (PERFORMED BY FORTY FIVE NORTH SURVEYING, LLC). THE CONTRACTOR IS RESPONSIBLE FOR REPORTING ANY DISCREPANCIES TO THE OWNER'S REPRESENTATIVE.

ABBREVIATIONS:

PL	PROPERTY LINE
FF	FINISHED FLOOR
TC	TOP OF CURB
FS	FINISHED SURFACE
FL	FLOW LINE
FG	FINISHED GRADE
GB	GRADE BREAK
CL	CENTERLINE
R	RIDGE LINE
R/W	RIGHT OF WAY
WV	WATER VALVE
PR.	PROPOSED
NAP	NOT A PART
FT	FEET
EV	ELECTRIC VEHICLE
CAV	CLEAN AIR VEHICLE
STD.	STANDARD
AC.	ACRES
CUP	CONDITIONAL USE PERMIT
EX.	EXISTING
TYP.	TYPICAL
MIN.	MINIMUM
SS	SANITARY SEWER
SD	STORM DRAIN
CF	CURB FACE
WM	WATER METER
FDC	FIRE DEPARTMENT CONNECTION
APN	ACCESSOR'S PARCEL MAP
SQ.FT	SQUARE FEET
INV.	INVERT
BF	BACKFLOW
CFS	CUBIC FEET PER SECOND
SCH.	SCHEDULE
PVC	POLYVINYL CHLORIDE
SDR	SPECIAL DRAWING RIGHT
PSI	POUNDS PER SQUARE INCH
NFPA	NATIONAL FIRE PREVENTION ASSOCIATION
CB	CATCH BASIN
D	DIAMETER
VCP	VITRIFIED CLAY PIPE

SHEET INDEX:

NOTICE TO EXCAVATORS:
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DIAL 811 or 1-800-332-2344

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345 WESTFIELD ST. #107
SILVERTON, OR. 97381
503.308.8554
ADMIN@7OAKSENGINEERING.COM

STAMP:
REGISTERED PROFESSIONAL ENGINEER
94115PE
PRELIMINARY
OREGON
NOV 13, 2018
KIMBERLY JOHNSON

NO	DATE	ISSUE DESCRIPTION

QC BY:
DRAWN BY:

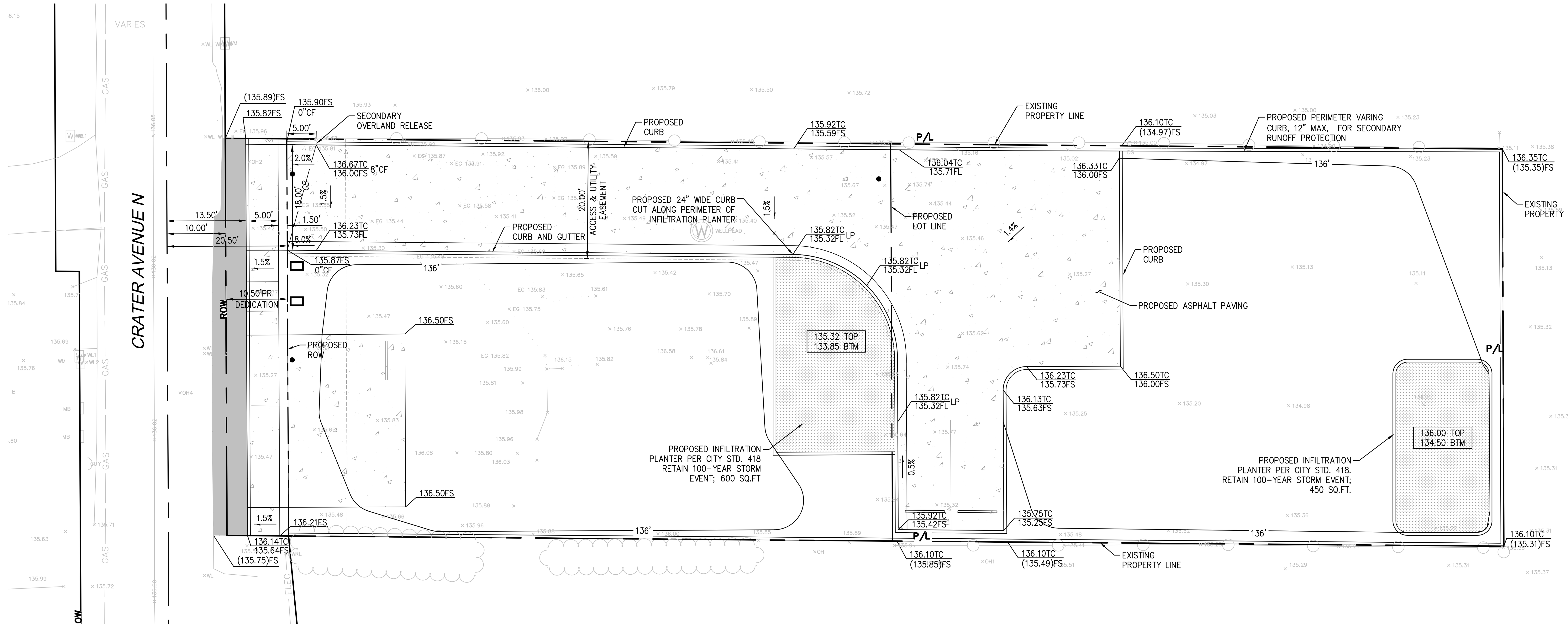
PROJECT NAME: CRATER AVENUE
PROJECT ADDRESS: 4910 CRATER AVENUE
KEIZER, OREGON

SHEET TITLE: TITLE SHEET

DATE: 11/22/2024

SHEET NUMBER: 1

JOB #00355



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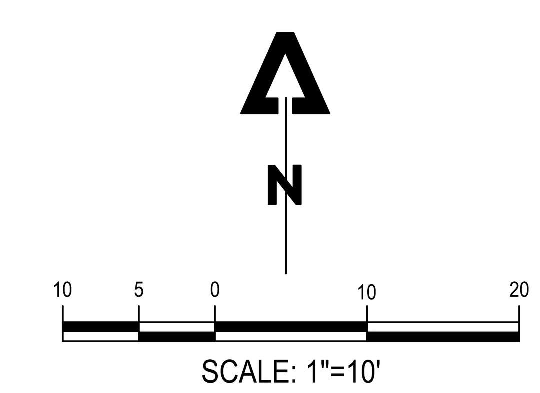

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OREGON
NOV 13, 2018
KIMBERLY JOHNSON

NO	DATE	ISSUE DESCRIPTION

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CONTRACTOR SHALL VERIFY ALL EXISTING CONDITIONS PRIOR TO PROCEEDING WITH CONSTRUCTION AND NOTIFY ARCHITECT IMMEDIATELY OF ANY DISCREPANCIES OR CONFLICTS.

QC BY:
DRAWN BY:

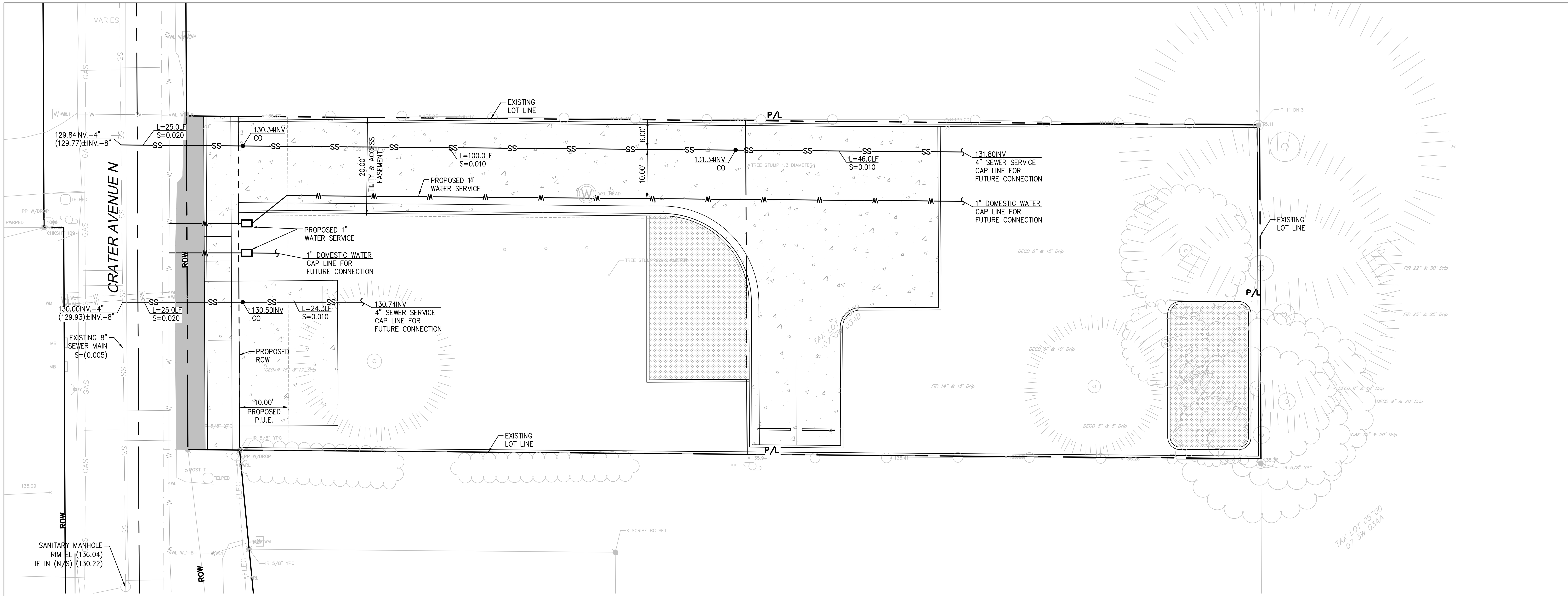
PROJECT NAME:
CRATER AVENUE
PROJECT ADDRESS:
4910 CRATER AVENUE
KEIZER, OREGON

SHEET TITLE:
PRELIMINARY GRADING PLAN

DATE: 11/22/2024

SHEET NUMBER:
2

JOB #00355



LEGEND:

SD	EXISTING DRAIN LINE
SS	PROPOSED SEWER
W	PROPOSED WATER
•	PROPOSED SEWER CLEANOUT
▭	PROPOSED WATER METER

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N

10 5 0 10 20

SCALE: 1"=10'

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QC BY: DRAWN BY:

PROJECT NAME: CRATER AVENUE

PROJECT ADDRESS: 4910 CRATER AVENUE

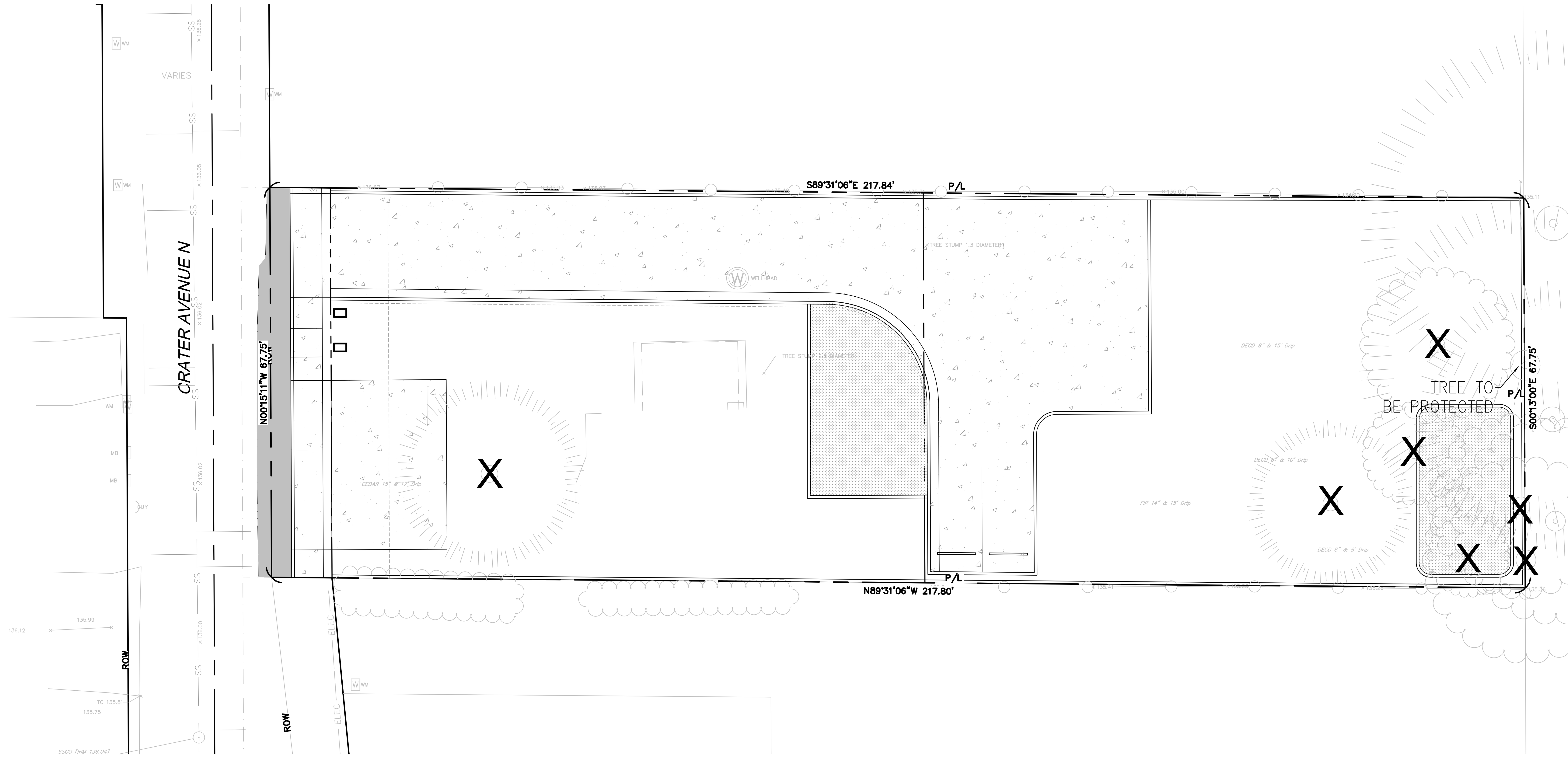
KEIZER, OREGON

SHEET TITLE:
PRELIMINARY UTILITY PLAN

DATE: 11/22/2024

SHEET NUMBER: 3

JOB #00355



TREE PRESERVATION ORDINANCE:

1. TREES IN THE PUBLIC RIGHT-OF-WAY
   - 1.1. MAINTENANCE AND PROTECTION OF TREES. TREES LOCATED IN THE PUBLIC-RIGHT-OF-WAY, INCLUDING PARKING STRIPS BETWEEN STREETS AND SIDEWALKS, SHALL BE MAINTAINED BY THE IMMEDIATELY ABUTTING PROPERTY OWNER ("OWNER") AND SHALL NOT BE CUT DOWN, REMOVED, TOPPED OR TRANSPLANTED WITHOUT FIRST OBTAINING A PERMIT FROM THE CITY OF KEIZER. "TOPPING" IS DEFINED AS THE SEVERE CUTTING BACK OF TRUNK AND/OR LIMBS TO STUBS WITHIN THE TREE'S CROWN TO SUCH A DEGREE SO AS TO REMOVE THE NORMAL CANOPY AND DISFIGURE THE TREE. IT SHALL BE UNLAWFUL TO INTENTIONALLY DAMAGE, DEFACE OR ATTACH ANY ROPE, WIRE NAIL, SIGN, POSTER, OR ANY OTHER MANMADE OBJECT TO ANY TREE LOCATED IN THE PUBLIC RIGHT OF WAY.
2. REMOVAL OF TREES. REMOVAL OF TREES LOCATED NEAR OR WITHIN THE PUBLIC RIGHT OF WAY IS THE RESPONSIBILITY OF THE OWNER AND IS REQUIRED IN THE FOLLOWING CIRCUMSTANCES:
   - 2.0.1. WHEN THE DAMAGE OCCURRING TO THE SURROUNDING LAW, STREET, SIDEWALK, OR/OR CURB.
   - 2.0.2. WHEN A TREE IS DAMAGED OR SICK OR CREATING IMMINENT DANGER TO PERSONS OR PROPERTY.
   - 2.0.3. WHEN A TREE IS INFECTED BY OR HARBORS ANY INJURIOUS FUNGUS, INSECT OR DISEASE WHICH CONSTITUTES A POTENTIAL THREAT TO OTHER TREES WITHIN THE CITY.
   - 2.0.4. WHEN A TREE BY REASON OF ITS NATURE IS OR MAY BECOME INJURIOUS TO SEWERS, ELECTRIC POWER LINES, GAS LINES, WATER LINES, OR PUBLIC IMPROVEMENTS OF ANY TYPE.
   - 2.0.5. STUMPS OF TREES IN THE PUBLIC RIGHT-OF-WAY MAY REMAIN IF THEY DO NOT INTERFERE WITH THE INTEGRITY OF SAVED TREES, SO LONG AS SAID STUMPS SHALL BE REMOVED TO A DEPTH OF EIGHT INCHES BELOW THE LEVEL OF THE SIDEWALK OR IF NO SIDEWALK EXISTS, EIGHT INCHES BLOW THE GRADE OF THE CENTER LINE OF THE RIGHT-OF-WAY. STUMPS SHALL BE CUT AT GROUND LEVEL IF REMOVAL OF THE STUMP OR THE CUTTING OF HTE STUMP BELOW THE SURFACE SHALL HAVE A HARMFUL EFFECT ON ANY SAVED TREE OR TREES.
3. INTERFERENCE WITH STREET LIGHTING AND TRAFFIC DEVICES. EVERY OWNER OF ANY TREE LOCATED IN OR OVERHANGING ANY STREET OR PUBLIC RIGHT-OF-WAY WITHIN THE CITY OF KEIZER SHALL PRUNE ALL BRANCHES SO THAT BRANCHES SHALL NOT:
   - 3.1. OBSTRUCT VIEW OF ANY PUBLIC OR PRIVATE INTERSECTION PURSUANT TO KEIZER ZONING ORDINANCE SECTION 17.18.
   - 3.2. OBSTRUCT THE VIEW OF ANY TRAFFIC CONTROL DEVICE OR SIGN.
   - 3.3. SEVERELY OBSTRUCT THE LIGHT OF ANY STREETLIGHT
   - 3.4. PROTRUDE INTO THE AREA THIRTEEN (13) FEET ABOVE THE STREET SURFACE OR EIGHT (8) FEET ABOVE THE SIDEWALK SURFACE.
4. NUISANCE DECLARED. THE CITY OF KEIZER HEREBY DECLARES THAT ALL 10 TREES DESCRIBED IN SUBSECTIONS (B) AND (C) ABOVE ARE DEEMED TO BE 11 NUISANCES AND ARE SUBJECT TO ABATEMENT.
5. TREES EXEMPTED. THE CITY OF KEIZER HEREBY DECLARES THAT ALL 10 TREES DESCRIBED IN SUBSECTIONS (B) AND (C) ABOVE ARE DEEMED TO BE 11 NUISANCES AND ARE SUBJECT TO ABATEMENT.
6. PLANTING TREES IN PUBLIC RIGHT-OF-WAY. ANY TREE REMOVED FROM THE PUBLIC RIGHT-OF-WAY MUST BE REPLACED WITH A NEW TREE. IT SHALL BE UNLAWFUL TO PLAT ANY TREE IN PUBLIC RIGHT-OF-WAY WITHOUT FIRST OBTAINING WRITTEN PERMISSION FROM THE CITY OF KEIZER. APPLICATIONS FOR REMOVAL/PLANTING OF TREES SHALL BE COMBINED INTO ONE FORM. NO FEE SHALL BE CHARGED FOR THE PERMITTING PROCESS.

LEGEND:

X DENOTES REMOVAL OF TREES

○ DENOTES TREE TO BE PRESERVED

N

SCALE: 1"=10'

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NOV 13, 2018
KIMBERLY JOHNSON

NO	DATE	ISSUE DESCRIPTION

QC BY: DRAWN BY:

PROJECT NAME: CRATER AVENUE

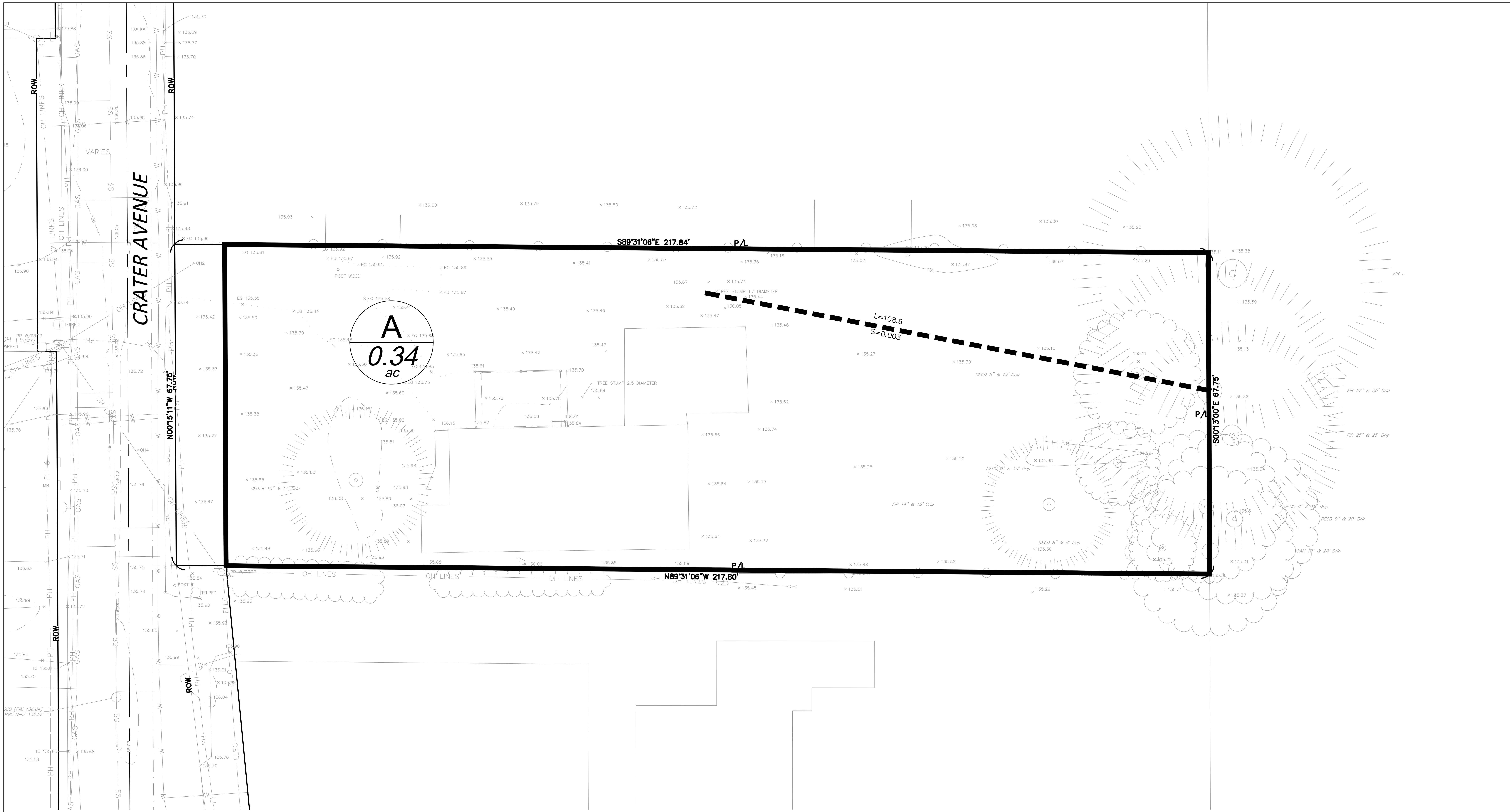
PROJECT ADDRESS: 4910 CRATER AVENUE KEIZER, OREGON

SHEET TITLE: TREE PRESERVATION PLAN

DATE: 11/22/2024

SHEET NUMBER: 4

JOB #00355



LEGEND:

DRAINAGE AREA

A .34 ac DRAINAGE AREA CALLOUT

SCALE: 1"=10'

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QC BY: DRAWN BY:

PROJECT NAME: CRATER AVENUE

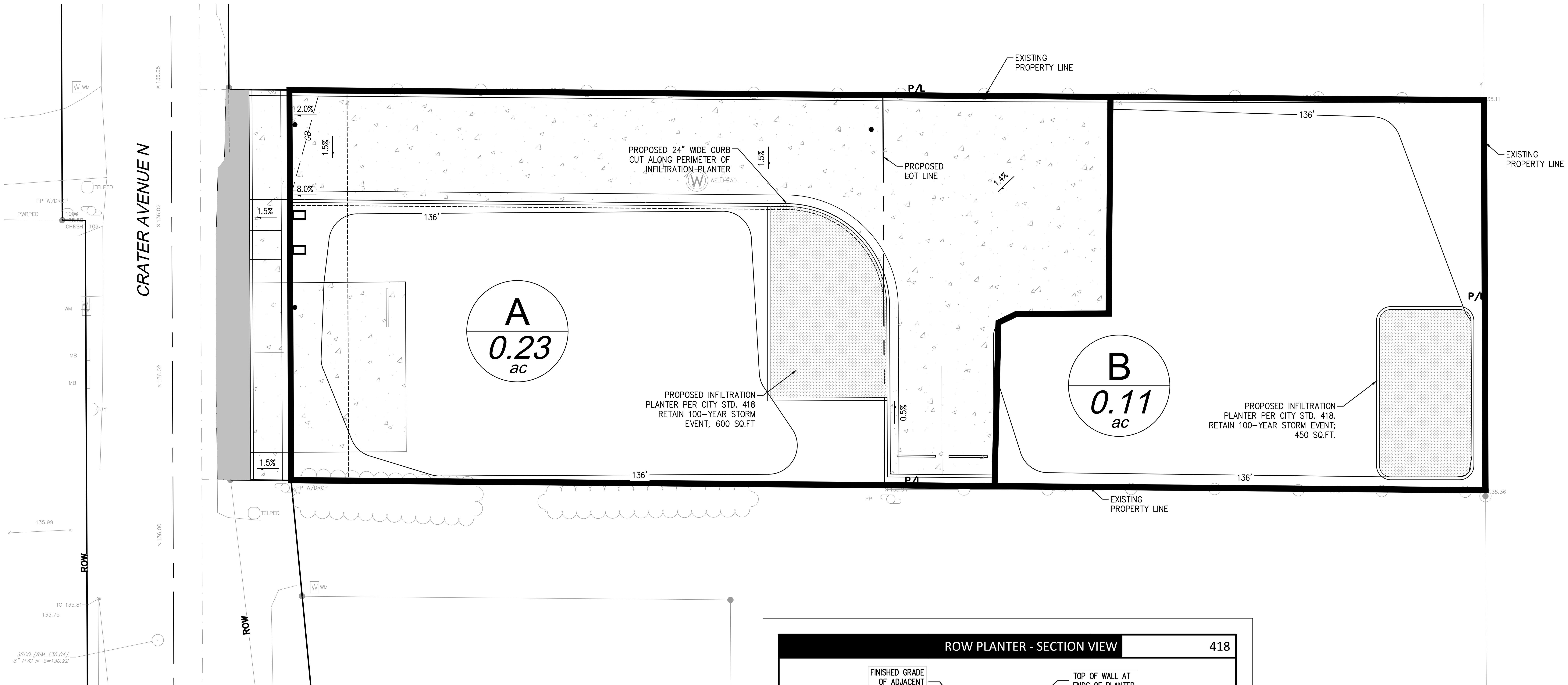
PROJECT ADDRESS: 4910 CRATER AVENUE KEIZER, OREGON

SHEET TITLE: EXISTING STROMWATER PLAN

DATE: 11/22/2024

SHEET NUMBER: 5

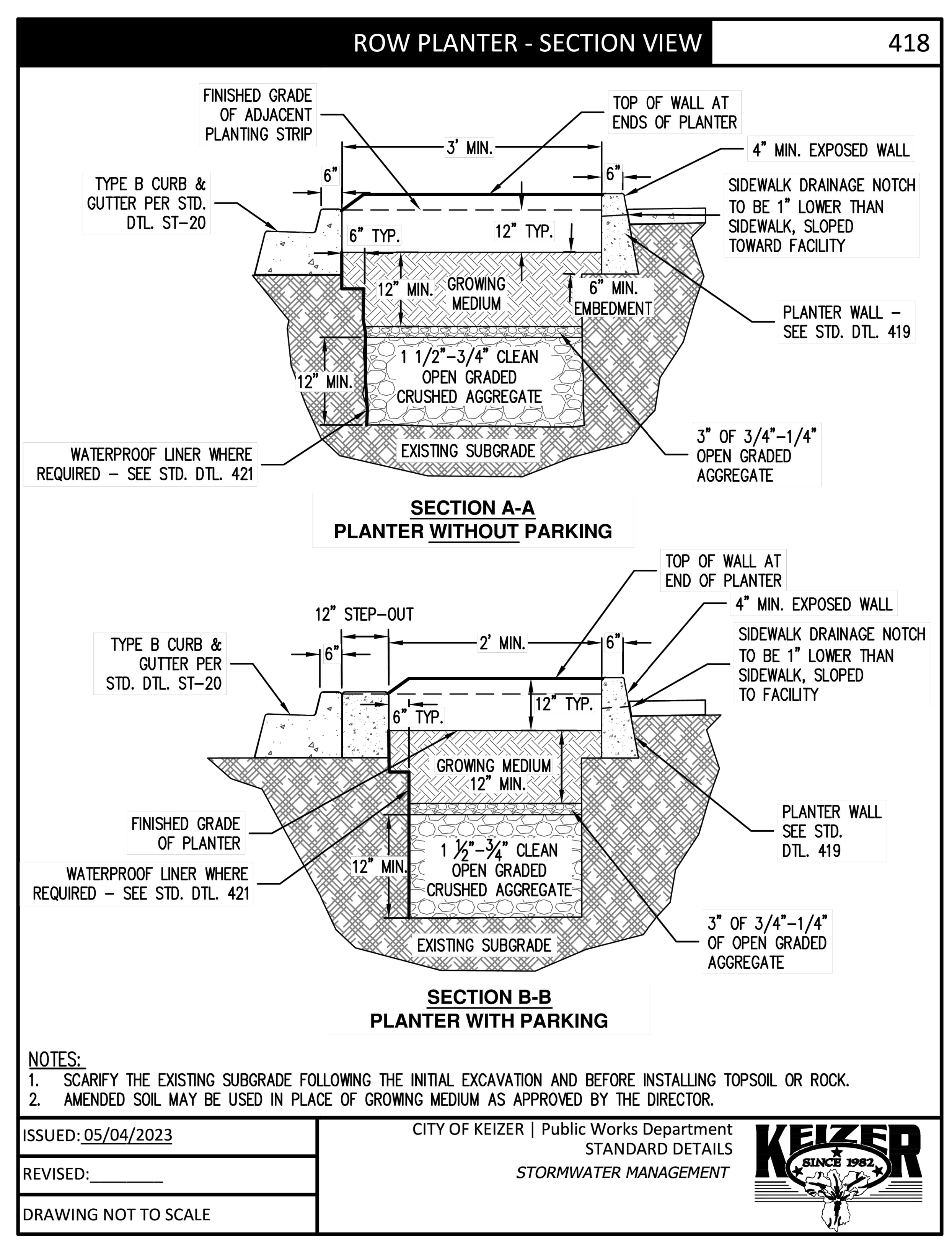
JOB #00355



PRE VS. POST CONSTRUCTION FLOW RATES		
FACILITY ID	PEAK FLOW RATE (CFS)	
	100 YEAR STORM	
PROJECT SITE	POST (NO BMP)	POST (W/BMP)
LOT A	0.19	0
LOT B	0.11	0

WATER QUALITY TREATMENT REQUIREMENTS				
CATCHMENT/ FACILITY ID	WQV (IN)	WQV (CF)	80% OF WQV	RAIN GARDEN ALLOWABLE VOLUME
A	1.38	741	592	1,080
B	2.38	392	314	810

CATCHMENT/ FACILITY ID	TOTAL AREA (SF)/(AC.)	IMPERVIOUS AREA (SF)	PERVIOUS AREA (SF)	OWNERSHIP (PRIVATE/ PUBLIC)	FACILITY TYPE	FACILITY SIZE (BOTTOM) SF
LOT A	9,045	7,236	1,809	PRIVATE	RAIN-GARDEN INFILTRATION	600
LOT B	5,000	4,000	1,000	PRIVATE	RAIN-GARDEN INFILTRATION	450
TOTAL ONSITE	**14,045**	**11,236**	**2,809**			



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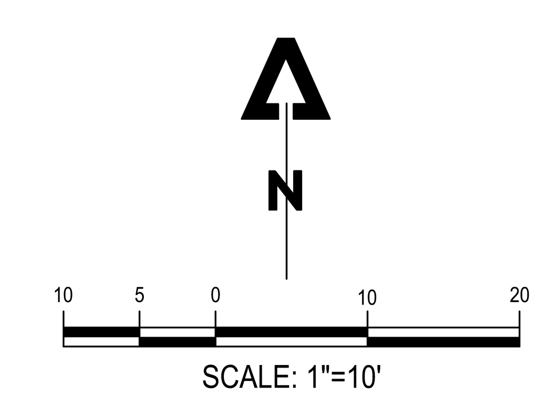

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STAMP:
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94115PE
PRELIMINARY
OREGON
NOV 13, 2018
KIMBERLY JOHNSON

NO	DATE	ISSUE DESCRIPTION

QC BY: DRAWN BY:

PROJECT NAME: CRATER AVENUE
PROJECT ADDRESS: 4910 CRATER AVENUE
KEIZER, OREGON

SHEET TITLE: PRELIMINARY STORMWATER PLAN

DATE: 11/22/2024

SHEET NUMBER: 6

JOB #00355

APPENDIX D – GEOTECHNICAL REPORT

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Geotechnical Engineering Report

Project Information:	4910 Crater Avenue N GeoPacific Project No. 24-6649 October 16, 2024
Site Location:	4910 Crater Avenue N Marion County Tax Lot 073W03AB 10500 Keizer, Oregon
Client:	Steve Hurley Banner Homes 2547 Aerial Way SE Salem, OR 97302 C/O: Steve Kay Cascadia Planning + Development Services, Email: steve@cascadiapd.com

TABLE OF CONTENTS

List of Appendices

List of Figures

1.0 PROJECT INFORMATION

This report presents the results of a geotechnical engineering study conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above-referenced project. The purpose of our investigation was to evaluate subsurface conditions at the site, assess potential geotechnical hazards at the property, and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific Proposal No. P-8886 dated August 15, 2024 and your subsequent authorization of our proposals and *General Conditions for Geotechnical Services*.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located at 4910 Crater Avenue N in Keizer, Marion County, Oregon. The site is rectangular in shape and is approximately 0.35 acres in size. Topography at the is relatively level with site elevations ranging from 139 to 140 feet amsl. The property is currently occupied by a single-family home and associated outbuildings. The site is bordered by residential properties to the east, north, and south and by Crater Avenue N to the west.

Based on review of a preliminary site plan provided by the client, plans for site development include the construction of a 2-lot residential partition with associated interior driveways and underground utilities. We anticipate that structures will be one to two stories, and that cuts and fills will be on the order of 4 feet or less. We understand that subsurface disposal of stormwater is desired at the site.

3.0 REGIONAL GEOLOGIC SETTING

The subject site lies within the Willamette Valley/Puget Sound lowland, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. A series of discontinuous faults subdivide the Willamette Valley into a mosaic of fault-bounded, structural blocks (Yeats et al., 1996). Uplifted structural blocks form bedrock highlands, while down-warped structural blocks form sedimentary basins.

The site is underlain by the Quaternary age (last 1.6 million years) Willamette Formation, a catastrophic flood deposit associated with repeated glacial outburst flooding of the Willamette Valley (Allison, 1953; Beaulieu et al., 1974; Gannett and Caldwell, 1998). The last of these outburst floods occurred approximately 10,000 years ago. These deposits typically consist of horizontally layered, micaceous, silt to coarse sand forming poorly-defined to distinct beds less than 3 feet thick (Yeats et al., 1996).

The Willamette Formation is underlain by the Columbia River Basalt Formation (Allison, 1953; Beaulieu et al., 1974; Gannett and Caldwell, 1998). The Miocene aged (about 14.5 to 16.5 million years ago) Columbia River Basalts are a thick sequence of lava flows which form the crystalline basement of the Willamette Valley (Beeson el al., 1989). The basalts are composed of dense, finely crystalline rock that is commonly fractured along blocky and columnar vertical joints. Individual basalt flow units typically range from 25 to 125 feet thick and interflow zones are typically vesicular, scoriaceous, brecciated, and sometimes include sedimentary rocks.

4.0 REGIONAL SEISMIC SETTING

At least one major fault zone capable of generating damaging earthquakes is thought to exist in the vicinity of the subject site. This fault zone is known as the Cascadia Subduction Zone.

4.1 Cascadia Subduction Zone

The Cascadia Subduction Zone is a 680-mile-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 4 cm per year (Goldfinger et al., 1996). A growing body of geologic evidence suggests that prehistoric subduction zone earthquakes have occurred (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). This evidence includes: (1) buried tidal marshes recording episodic, sudden subsidence along the coast of northern California, Oregon, and Washington, (2) burial of subsided tidal marshes by tsunami wave deposits, (3) paleoliquefaction features, and (4) geodetic uplift patterns on the Oregon coast. Radiocarbon dates on buried tidal marshes indicate a recurrence interval for major subduction zone earthquakes of 250 to 650 years with the last event occurring 300 years ago (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). The inferred seismogenic portion of the plate interface lies approximately along the Oregon Coast at depths of between 20 and 40 kilometers below the surface.

5.0 FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Our site-specific explorations for this report were conducted on September 5, 2024. Explorations consisted of two exploratory test pits excavated with a medium-sized excavator to depths of approximately 10 feet below the ground surface (bgs). The approximate locations of our explorations are presented on Figure 2. It should be noted that our explorations were located in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate.

A GeoPacific engineer continuously monitored the field exploration program and logged the explorations. Soils observed in the explorations were classified in general accordance with the Unified Soil Classification System (USCS). During exploration, we also noted geotechnical conditions such as soil consistency, moisture, and groundwater conditions. After the completion of each exploration, test pits were backfilled loosely with on-site soil. Logs of the explorations are attached to this report. The following report sections are based on the exploration program conducted on Ellingson Farm property and summarize subsurface conditions encountered at the site.

5.1 Soil Descriptions

Gravel Driveway Section: Directly underlying the ground surface in test pit TP-1, we observed approximately 8 inches of ¾”-0 crushed aggregate. This material appeared to be part of a previously utilized driveway.

Topsoil Horizon: The ground surface in the location of test pit TP-2 was directly underlain by a moderately organic topsoil horizon. The topsoil generally consisted of brown silt (OL-ML) that contained fine roots throughout. In test pit TP-2, the topsoil horizon extended to a depth of approximately 10 inches below the ground surface.

Disturbed Native: Underlying the topsoil horizon in test pit TP-2, we observed a layer of disturbed native soil. The disturbed native soil consisted of SILT (ML) which extended to total depth of approximately 16 inches bgs.

Catastrophic Flood Deposits: Beneath the gravel driveway section in test pit TP-1 and beneath the disturbed native soil in test pit TP-2, we encountered Catastrophic Flood Deposit soils. The near-surface Catastrophic Flood Deposit soils consisted of brown Sandy Lean CLAY (CL) which was generally stiff in consistency. The Sandy Lean CLAY (CL) soils graded to medium dense Poorly Graded SAND with Silt (SP) at depths between 6 and 7 feet bgs. The Catastrophic Flood Deposit soils extended beyond the maximum depth of our exploration (10 feet).

5.2 Shrink-Swell Potential

Low plasticity fine-grained and course-grained soils were encountered within the upper 10 feet of the test pit explorations conducted at the site. Based upon our observations and our local experience with the soil layers in the vicinity of the subject site, the shrink-swell potential of the soil types is considered to be low. Special design measures are not considered necessary to minimize the risk of uncontrolled damage of foundations as a result of potential soil expansion at this site.

5.3 Groundwater and Soil Moisture

On September 5, 2024, soils encountered in our explorations were generally dry to moist. Groundwater seepage was not encountered within our test pit explorations, which extended to a maximum depth of 10 feet bgs. Local well logs in the vicinity of the subject site indicate that static groundwater is present at depths of approximately 20 to 40 feet bgs (Oregon Water Resources Department, 2024).

Experience has shown that temporary perched storm-related groundwater conditions often occur within the surface soils over fine-grained native deposits such as those beneath the site, particularly during the wet season. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors. Perched groundwater may be encountered in localized areas and seeps and springs may exist in areas not explored and may become evident during site grading and removal of undocumented fill. If groundwater springs or seeps are observed to be present during site grading, subdrains may be needed to control and divert subsurface water. The geotechnical engineer should be alerted by the earthworks contractor to any subsurface springs that become apparent during site grading for additional evaluation and recommendations.

5.4 Infiltration Testing

Soil infiltration testing was performed using the encased falling-head method in test pit exploration TP-2 at depths of 4.0 and 8.0 feet bgs. The approximate locations of subsurface explorations are presented on Figure 2. Infiltration locations were presoaked prior to testing. The water level was measured to the nearest 0.01 inch from a fixed point and the change in water level was recorded at regular intervals until three successive measurements showing a consistent infiltration rate were achieved. Table 1 summarizes the results of the infiltration tests. Soils at the test locations were observed and sampled in order to characterize the subsurface profile.

Table 1 - Summary of Infiltration Test Results

Test Location	Depth (feet)	Soil Type	Infiltration Rate (in/hr)
TP-2	4.0	Sandy Lean CLAY (CL)	2.88
TP-2	8.0	Poorly Graded SAND with Silt (SP)	2.16

Infiltration rates have been reported without applying a factor of safety. Adequate separation from confining layers and groundwater levels should be maintained for stormwater management.

5.5 Subgrade Resilient Modulus & PDCP Testing Data

GeoPacific understands that development at the site will include construction of a new private roadway to provide access to the proposed homes. On September 5, 2024, subgrade soil strength testing was performed using a KSE DCP K-100 Model with a 17.6 lbs hammer portable dynamic cone penetrometer (PDCP), in the vicinity of the proposed private roadway. PDCP testing was conducted in the location of test pit TP-1. Approximate exploration locations are presented on Figure 2 of this report. Results of the PDCP testing are presented in the appendix to this report. Table 2 summarizes the results of our PDCP testing.

Table 2: PDCP Test Results

Field Test Designation	Test Pit Designation	Material Tested	Depth Interval of Test (in.) bgs	Average Penetration Per Blow (mm)	Soil Resilient Modulus MR (psi)
PDCP-1	TP-1	Sandy Lean CLAY (CL)	1.2 – 35.5	14.0	5,981

6.0 CONCLUSIONS AND RECOMMENDATIONS

Our site investigation indicates that the proposed construction appears to be geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. The primary geotechnical concern associated with the site development is the of a disturbed layer of soil across the site. During our investigation, we observed a layer of disturbed soil in the eastern portion of the site extending to a depth of approximately 16 inches bgs. In areas where structures or engineered fill are planned, this disturbed layer, where encountered, should be root-picked, tilled, moisture-conditioned, and recompacted or should be completely removed. The disturbed layer of soil may be adequate to remain under roadways. GeoPacific should be consulted to evaluate the subgrade in roadway areas during construction.

The following report sections provide recommendations for site development and construction in accordance with the current applicable codes and local standards of practice.

6.1 Site Preparation Recommendations

Areas of proposed construction and areas to receive fill should be cleared of any organic and inorganic debris, disturbed soil, and loose stockpiled soils. Inorganic debris and organic materials from clearing should be removed from the site. Organic-rich soils and root zones should then be stripped from construction areas of the site or where engineered fill is to be placed. The average depth of stripping of existing organic topsoil is estimated to be approximately 10 inches at the site but may be deeper in the vicinity of trees and bushes.

The final depth of soil removal should be determined by the geotechnical engineer or designated representative during site inspection while stripping/excavation is being performed. Stripped topsoil should be removed from areas proposed for placement of engineered fill and structures. Any remaining topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative.

In areas of roadways, structures, or where engineered fill material is proposed, undocumented fills and any subsurface structures (dry wells, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be completely removed and the excavations backfilled with engineered fill. During our investigation, we observed a layer of disturbed soil extending to approximately 16 inches bgs. In areas where structures or engineered fill are planned, this disturbed layer should be root-picked, tilled, moisture-conditioned, and recompacted or should be completely removed. The disturbed layer of soil may be adequate to remain under private roadways. GeoPacific should be consulted to evaluate the subgrade in roadway areas during construction by proof rolling, probing, and/or potholing.

Site earthwork may be impacted by wet weather conditions. Stabilization of subgrade soils may require aeration and re-compaction. If subgrade soils are found to be difficult to stabilize, over-excavation, placement of granular soils, or cement treatment of subgrade soils may be feasible options. GeoPacific should be onsite to observe preparation of subgrade soil conditions prior to placement of engineered fill.

6.2 Engineered Fill

All grading for the proposed construction should be performed as engineered grading in accordance with the applicable building code at the time of construction with the exceptions and additions noted herein. Site grading should be conducted in accordance with the requirements outlined in the 2021 International Building Code (IBC), and 2022 Oregon Structural Specialty Code (OSSC), Chapter 18 and Appendix J. Areas proposed for fill placement should be prepared as described in the section of this report titled Site Preparation. During our investigation, we observed a layer of disturbed soil extending to approximately 16 inches bgs. In areas where structures or engineered fill are planned, this disturbed layer should be root-picked, tilled, moisture-conditioned, and recompacted or should be completely removed. Site preparation, soil stripping, and grading activities should be observed and documented by a geotechnical engineer or his representative. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill.

Onsite native soils appear to be suitable for use as engineered fill. Soils containing greater than 5 percent organic content should not be used as structural fill. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 3 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 12 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 95 percent of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Soils should be moisture conditioned to within two percent of optimum moisture. Field density testing should conform to ASTM D2922 and D3017, or D1556. All engineered fill should be observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency.

Site earthwork may be impacted by soil moisture and wet weather conditions. Earthwork in wet weather would likely require extensive use of additional crushed aggregate, cement or lime treatment, or other special measures, at considerable additional cost compared to earthwork performed under dry-weather conditions.

6.3 Excavating Conditions and Utility Trench Backfill

We anticipate that onsite soils can generally be excavated using conventional heavy equipment. Shallow groundwater may be encountered. Perched groundwater conditions coupled with silty soils could cause sidewall caving in excavations. These conditions could make utility trenching difficult, especially in the winter months, and adequate shoring should be maintained.

Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926) or be shored. The existing native Sandy Lean CLAY (CL) soils encountered above 6 to 7 feet bgs classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. The existing native Poorly Graded SAND with Silt (SP) soils encountered below 6 to 7 feet bgs classify as Type C Soil and temporary excavation side slope inclinations as steep as 1.5H:1V may be assumed for planning purposes. This cut slope inclination is applicable to excavations above the water table only. If native ash soils are encountered in temporary excavations, GeoPacific should be consulted to evaluate the soils and provide additional recommendations.

Shallow, perched groundwater may be encountered at the site and should be anticipated in excavations and utility trenches. Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

Underground utility pipes should be installed in accordance with the procedures specified in ASTM D2321 and jurisdictional standards. We recommend that structural trench backfill be compacted to at least 90 percent of the maximum dry density obtained by the Modified Proctor (ASTM D1557, AASHTO T-180) or equivalent. Initial backfill lift thicknesses for a ¾”-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, at least one density test is taken for every 4 vertical feet of backfill on each 100-lineal-foot section of trench.

6.4 Erosion Control Considerations

During our field exploration program, we generally did not observe soil or topographic conditions which are considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw wattles, fiber rolls, and silt fences. If used, these erosion control devices should remain in place throughout site preparation and construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets. Areas of exposed soil requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

6.5 Wet Weather Earthwork

Soils underlying the site are likely to be moisture sensitive and will be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will require expensive measures such as cement treatment or imported granular material to compact areas where fill may be proposed to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean engineered fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.
- Material used as engineered fill should consist of clean, granular soil containing less than 5 percent passing the No. 200 sieve. The fines should be non-plastic. Alternatively, cement treatment of on-site soils may be performed to facilitate wet weather placement.
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials.
- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved.
- Geotextile silt fences, straw wattles, and fiber rolls should be strategically located to control erosion.

If cement or lime treatment is used to facilitate wet weather construction, GeoPacific should be contacted to provide additional recommendations and field monitoring.

6.6 Spread Foundations

We anticipate that the proposed residential structures will be one to two stories, constructed with typical spread foundations and wood framing, with maximum structural loading on column footings and continuous strip footings on the order of 10 to 35 kips, and 2 to 4 kips respectively. We anticipate maximum cuts and fills will be on the order of 4 feet or less.

The proposed structures may be supported on shallow foundations bearing on medium stiff to stiff, native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. During our investigation, we observed a layer of disturbed soil extending to approximately 16 inches bgs. In areas where structures or engineered fill are planned, this disturbed layer should be root-picked, tilled, moisture-conditioned, and recompacted or should be completely removed. Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 12 inches below exterior grade. If soft soil conditions are encountered at footing subgrade elevation, they should be removed and replaced with compacted crushed aggregate.

The anticipated allowable soil bearing pressure is 1,500 lbs/ft^2 for footings bearing on competent, native soil and/or engineered fill. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. For loads heavier than 35 kips, the geotechnical engineer should be consulted. If heavier loads than described above are proposed, it may be necessary to over-excavate point load areas and replace with additional compacted crushed aggregate to achieve a higher allowable bearing capacity. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

Footing excavations should penetrate through topsoil, undocumented fill material, and any disturbed soil to competent subgrade that is suitable for bearing support. During our investigation, we observed a layer of disturbed soil extending to approximately 16 inches bgs. In areas where structures or engineered fill are planned, this disturbed layer should be root-picked, tilled, moisture-conditioned, and recompacted or should be completely removed. All footing excavations should be trimmed neat, and all loose or softened soil should be removed from the excavation bottom prior to placing reinforcing steel bars. Due to the moisture sensitivity of on-site native soils, foundations constructed during the wet weather season may require over-excavation of footings and backfill with compacted, crushed aggregate.

6.7 Concrete Slabs-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as described in Section 6.1, Site Preparation Recommendations and Section 6.6, Spread Foundations. Care should be taken during excavation for foundations and floor slabs, to avoid disturbing subgrade soils. If subgrade soils have been adversely impacted by wet weather or otherwise disturbed, the surficial soils should be scarified to a minimum depth of 8 inches, moisture conditioned to within about 3 percent of optimum moisture content and compacted to engineered fill specifications. Alternatively, disturbed soils may be removed and the removal zone backfilled with additional crushed rock.

For evaluation of the concrete slab-on-grade floors using the beam on elastic foundation method, a modulus of subgrade reaction of 150 kcf (87 pci) should be assumed for the stiff, fine grained soils anticipated to be present at foundation subgrade elevation following adequate site preparation as described above. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of 8 inches of 1½”-0 crushed aggregate beneath the slab. The total thickness of crushed aggregate will be dependent on the subgrade conditions at the time of construction and should be verified visually by proof-rolling. Under-slab aggregate should be compacted to at least 95 percent of its maximum dry density as determined by ASTM D698 (Standard Proctor) or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. Appropriate design professionals should be consulted regarding vapor barrier and damp proofing systems, ventilation, building material selection and mold prevention issues, which are outside GeoPacific’s area of expertise.

6.8 Footing and Roof Drains

The outside edge of perimeter footings should be provided with a drainage system consisting of 3-inch diameter, slotted, flexible plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining gravel or 1 1/2" - 3/4" drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. Water collected from the footing drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. Down spouts and roof drains should not be connected to the foundation drains in order to reduce the potential for clogging. The footing drains should include clean-outs to allow periodic maintenance and inspection. Grades around the proposed structure should be sloped such that surface water drains away from the building.

Perimeter footing drains are recommended to prevent detrimental effects of surface water runoff on foundations – not to dewater groundwater. Footing drains should not be expected to eliminate all potential sources of water entering a basement or beneath a slab-on-grade. An adequate grade to a low point outlet drain in the crawlspace is required by code.

6.9 Permanent Below-Grade Foundation Walls

Lateral earth pressures against below-grade foundation retaining walls will depend upon the inclination of any adjacent slopes, type of backfill, degree of wall restraint, method of backfill placement, degree of backfill compaction, drainage provisions, and magnitude and location of any adjacent surcharge loads. At-rest soil pressure is exerted on a retaining wall when it is restrained against rotation. In contrast, active soil pressure will be exerted on a wall if its top is allowed to rotate or yield a distance of roughly 0.001 times its height or greater.

If the subject retaining walls will be free to rotate at the top, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf for level backfill against the wall. For restrained wall, an at-rest equivalent fluid pressure of 52 pcf should be used in design, again assuming level backfill against the wall. These values assume that the recommended drainage provisions are incorporated, and hydrostatic pressures are not allowed to develop against the wall.

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the Mononobe-Okabe equation and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 6.5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend a passive earth pressure of 320 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and GeoPacific should be contacted for additional recommendations.

A coefficient of friction of 0.42 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

The above recommendations for lateral earth pressures assume that the backfill behind the subsurface walls will consist of properly compacted structural fill, and no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the surcharge pressure should be added. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice.

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a 12 to 18-inch wide zone of sand and gravel containing less than 5 percent passing the No. 200 sieve against the walls. A 3-inch minimum diameter perforated, plastic drain-pipe should be installed at the base of the walls and connected to a suitable discharge point to remove water in this zone of sand and gravel. The drain-pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

Wall drains are recommended to prevent detrimental effects of surface water runoff on foundations – not to dewater groundwater. Drains should not be expected to eliminate all potential sources of water entering a basement or beneath a slab-on-grade. An adequate grade to a low point outlet drain in the crawlspace is required by code. Underslab drains are sometimes added beneath the slab when placed over soils of low permeability and shallow, perched groundwater.

Water collected from the wall drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. Down spouts and roof drains should not be connected to the wall drains in order to reduce the potential for clogging. The drains should include clean-outs to allow periodic maintenance and inspection. Grades around the proposed structure should be sloped such that surface water drains away from the building.

GeoPacific should be contacted during construction to verify subgrade strength in wall keyway excavations, to verify that backslope soils are in accordance with our assumptions, and to take density tests on the wall backfill materials.

Structures should be located a horizontal distance of at least 1.5H away from the back of the retaining wall, where H is the total height of the wall. GeoPacific should be contacted for additional foundation recommendations where structures are located closer than 1.5H to the top of any wall.

6.10 Stormwater Management

We understand that plans for project development may include stormwater management facilities, and that it may be desired to incorporate subsurface disposal of stormwater. The native Sandy Lean CLAY (CL) observed in our test pit explorations from approximately 1 to 6 feet bgs exhibited an infiltration rate of 2.88 inches per hour. The native Poorly Graded SAND with Silt (SP) observed in our test pit explorations from approximately 6 to 10 feet bgs exhibited an infiltration rate of 2.16 inches per hour.

Infiltration rates presented in this report should not be applied to inappropriate or complex hydrological models such as a closed basin without extensive further studies. Adequate separation from the confining layers and groundwater levels should be maintained for stormwater management. Evaluating environmental implications of stormwater disposal at this site are beyond the scope of this study. Stormwater management systems should be constructed as specified by the designer and/or in accordance with the applicable stormwater design codes. Stormwater exceeding storage capacities will need to be directed to a suitable surface discharge location, away from structures. Stormwater management systems may need to include overflow outlets, surface water control measures and/or be connected to the street storm drain system, if available. In no case should stormwater be allowed to flow uncontrolled over the ground surface. Evaluating environmental implications of stormwater disposal at this site are beyond the scope of this study.

6.11 Pavement Section Design

From discussions with the client and conceptual designs for the project, we understand that the development at the site will include construction of interior driveways and parking areas. We have made assumptions based on the number of proposed lots regarding the amount and size of vehicles in the anticipated traffic. We designed for an anticipated ESAL count of approximately 50,000 over 20 years for the private roadway. We assume that traffic for the interior roadways will primarily consist of passenger vehicles, garbage trucks, and occasional fire trucks, weighing up to 75,000 pounds. For design purposes, we used a resilient modulus of 5,900 based on our PDCP data. Table 3 presents our pavement section design input factors.

Table 3: Flexible Pavement Section Design Input Parameters for Private Roadways

Input Parameter	Design Value
18-kip ESAL Initial Performance Period (20 Years)	50,000
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	90 Percent
Overall Standard Deviation	0.5
Roadbed Soil Resilient Modulus (PSI)	5,900
Required Structural Number	**2.38**

Table 4 presents our minimum dry-weather pavement section for the interior drive area with estimated structural coefficients calculated into a structural number. Design calculations are attached to this report.

Table 4 – Minimum Dry-Weather Flexible Pavement Section for Private Roadways

Material Layer	Section Thickness (in.)	Structural Coefficient	Compaction Standard
Asphaltic Concrete Pavement (ACP)	3	0.42	91% / 92% of Rice Density AASHTO T-209
¾-0 Crushed Aggregate	2	0.10	95% of Modified Proctor AASHTO T-180
1½-0 Crushed Aggregate Base	8	0.10	95% of Modified Proctor AASHTO T-180
Subgrade	12	N/A	95% of Standard Proctor AASHTO T-99, approved native, or equivalent
Calculated Structural Number 2.52			

Any pockets of organic debris or loose fill encountered during subgrade preparation should be removed and replaced with engineered fill (see *Site Preparation* Section). As discussed previously, we observed a layer of disturbed soil extending to approximately 16 inches bgs. The disturbed layer of soil may be adequate to remain under roadways. GeoPacific should be consulted to evaluate the subgrade in roadway areas during construction by proof rolling, probing, and/or potholing. In order to evaluate subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving. If pavement areas are to be constructed during wet weather, the subgrade and construction plan should be reviewed by the project geotechnical engineer at the time of construction so that condition specific recommendations can be provided. The moisture sensitive subgrade soils make the site a difficult wet weather construction project. General recommendations for wet weather pavement sections are provided below.

During placement of pavement section materials, density testing should be performed to verify compliance with project specifications. Generally, one subgrade and one base course test is performed for every 100 to 200 linear feet of roadway.

6.12 Wet Weather Pavement Construction

This section presents our recommendations for wet weather pavement section and construction for new pavement sections at the project. These wet weather pavement section recommendations are intended for use in situations where it is not feasible to compact the subgrade soils to Washington County requirements, due to wet subgrade soil conditions, and/or construction during wet weather. Based on our site review, we recommend a wet weather section with a minimum subgrade deepening of 6 inches to accommodate a working subbase of additional 1½”-0 crushed rock.

Geotextile fabric, Mirafi 500x or equivalent, should be placed on subgrade soils prior to placement of base rock.

In some instances, it may be preferable to use biaxial geogrid or granular subbase materials. GeoPacific should be consulted for additional recommendations if it is desired to pursue alternatives. Cement treatment of the subgrade may also be considered instead of overexcavation. For planning purposes, we anticipate that treatment of the onsite soils would involve mixing cement powder to approximately 6 percent cement content and a mixing depth on the order of 18 inches. Actual depths of cement treatment and cement percentages would be determined during construction, based on the moisture content of the existing soil and the depths of soft soil.

With implementation of the above recommendations, it is our opinion that the resulting pavement section will provide equivalent or greater structural strength than the dry weather pavement section currently planned. However, it should be noted that construction in wet weather is risky and the performance of pavement subgrades depend on a number of factors including the weather conditions, the contractor's methods, and the amount of traffic the road is subjected to. There is a potential that soft spots may develop even with implementation of the wet weather provisions recommended in this letter. If soft spots in the subgrade are identified during roadway excavation, or develop prior to paving, the soft spots should be overexcavated and backfilled with additional crushed rock.

During subgrade excavation, care should be taken to avoid disturbing the subgrade soils. Removals should be performed using an excavator with a smooth-bladed bucket. Truck traffic should be limited until an adequate working surface has been established. We suggest that the crushed rock be spread using bulldozer equipment rather than dump trucks, to reduce the amount of traffic and potential disturbance of subgrade soils.

Care should be taken to avoid overcompaction of the base course materials, which could create pumping, unstable subgrade soil conditions. Heavy and/or vibratory compaction efforts should be applied with caution. Following placement and compaction of the crushed rock to project specifications (95 percent of Modified Proctor), a finish proof-roll should be performed before paving.

The above recommendations are subject to field verification. GeoPacific should be on-site during construction to verify subgrade strength and to take density tests on the engineered fill and base rock materials.

7.0 SEISMIC DESIGN

The Oregon Department of Geology and Mineral Industries (Dogami), Oregon HazVu: 2024 Statewide GeoHazards Viewer indicates that the site is in an area where *violent* ground shaking is anticipated during an earthquake (Dogami HazVu, 2024). Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2021 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2022). We recommend Site Class D be used for design as defined in ASCE 7-16, Chapter 20, and Table 20.3-1 and seismic design category D_0 as defined in 2021 International Residential Code (IRC) Table R301.2.2.1.1. Design values determined for the site using the ATC (Applied Technology Council)

2024 Hazards by Location Online Tool are summarized in Table 5 and are based upon existing soil conditions.

Table 5 - Recommended Earthquake Ground Motion Parameters (ASCE 7-16)

Parameter	Value
Location (Lat, Long), degrees	44.995, -123.036
Risk-Targeted Maximum Considered Earthquake Design Parameters, 2% Exceedance in 50 years (MCE_R):	
Peak Ground Acceleration PGA_M	0.470 g
Short Period, S_s	0.834 g
1.0 Sec Period, S_1	*0.416 g
Soil Factors for Site Class D:	
F_a	1.167
F_v	*1.884
SD_s = 2/3 x F_a x S_s	0.648 g
SD_1 = 2/3 x F_v x S_1	*0.522 g
Seismic Design Category	D (D_0 per 2021 IRC)

*The **F_v** value reported in the above table is a straight-line interpolation of mapped spectral response acceleration at 1-second period, S_1 per Table 1613.2.3(2) of OSSC 2022 with the assumption that Exception 2 of ASCE 7-16 Chapter 11.4.8 is met per the Structural Engineer. If Exception 2 is not met, and the long-period site coefficient (F_v) is required for design, GeoPacific Engineering can be consulted to provide a site-specific procedure as per ASCE 7-16, Chapter 21.

7.1 Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2024 Statewide GeoHazards Viewer indicates that the site is in an area considered to be at a *very high* risk for soil liquefaction during an earthquake. Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to ground shaking caused by strong earthquakes. Soil liquefaction is generally limited to loose sands and granular soils located below the water table, and fine-grained soils with a plasticity index less than 15.

The upper 10 feet of the site was observed to be underlain by silt and sand soils, which generally had a stiff or medium dense consistency. Groundwater seepage was not observed within our explorations. Local well logs indicate that static groundwater is present at a depth of approximately 20 to 40 feet below the existing ground surface (Oregon Water Resources Department, 2024). It is our understanding that for construction of single-family homes, special design or construction measures are not required by code to mitigate the effects of liquefaction.

8.0 UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and their consultants for use in design of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, GeoPacific should be notified for review of the recommendations of this report, and revision of such if necessary.

Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

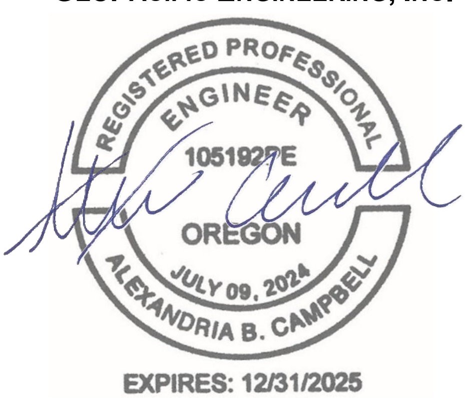
This report should not be relied upon by third parties unless a reliance letter has been issued by GeoPacific specifically to that third party, otherwise the third party should rely upon their own due diligence and geotechnical studies only. Foundations, and wood floors and slab-on-grade performance should be evaluated in accordance with ASCE Guidelines for the Evaluation and Repair of Residential Foundations (ASCE Texas Chapter, 2009) when exceeding L/100 for overall tilting and L/360 for overall deflection across the length of the home, unless superseded by the builder's warranty guidelines. Localized deflections may exceed these tolerances due to other factors such as built-in uneveness.

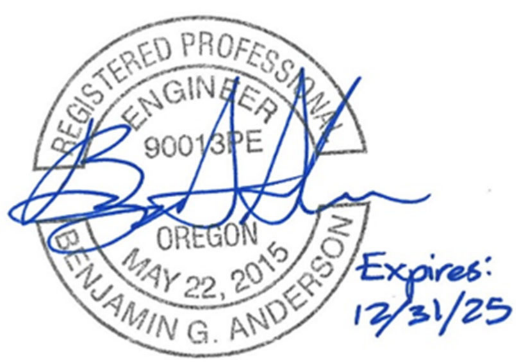
Within the limitations of scope, schedule and budget, GeoPacific attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, expressed or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

GEOPACIFIC ENGINEERING, INC.





Alexandria B. Campbell, P.E.
Staff Engineer

Benjamin G. Anderson, P.E.
Associate Engineer

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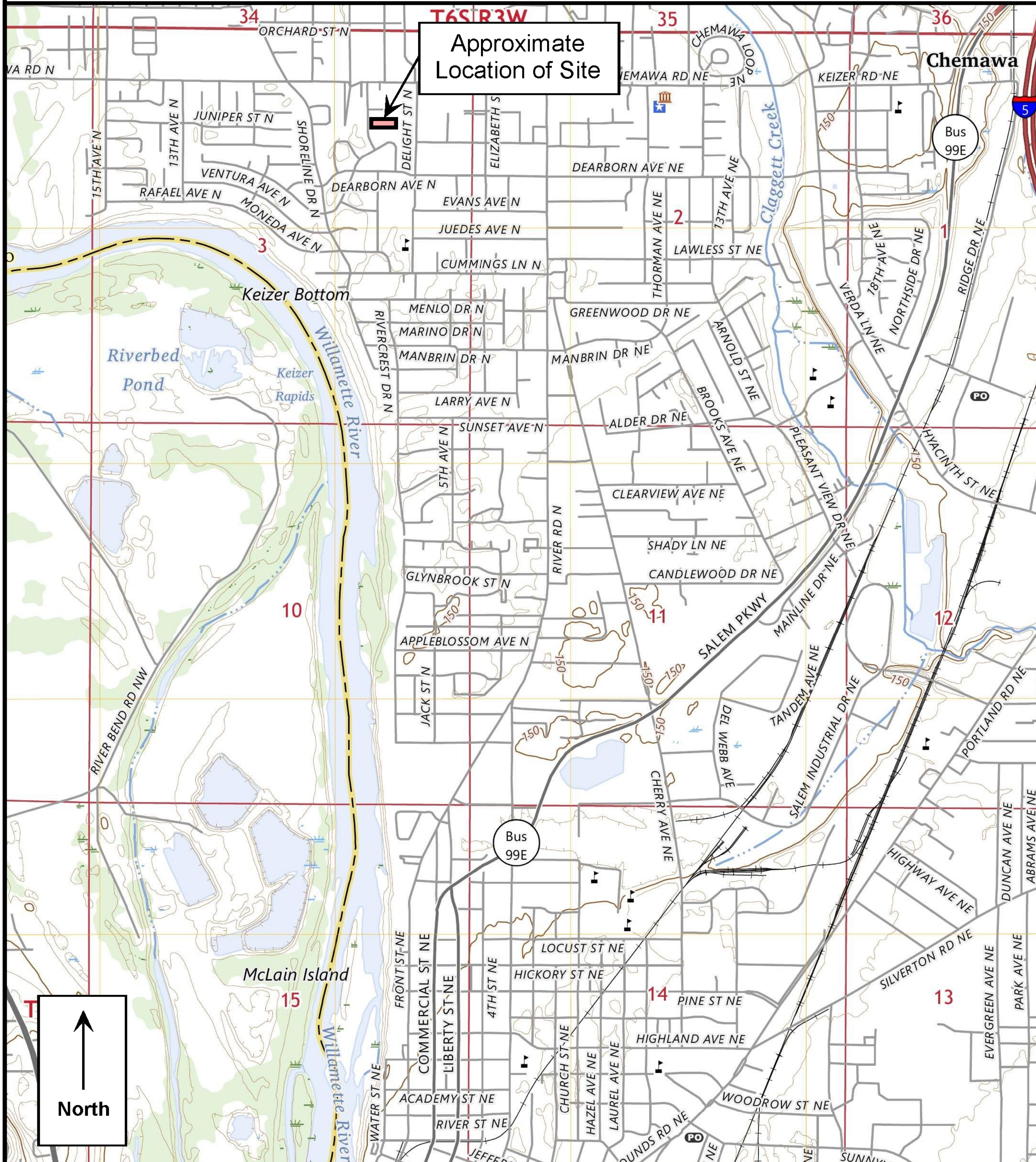
FIGURES

GEOPACIFIC

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

SITE VICINITY MAP



Base map: U.S. Geological Survey 7.5 minute Topographic Map Series, Salem West, Oregon Quadrangle, 2024

Date: 09/10/24

Drawn by: ABC

0 2,000

Approximate Scale: 1" = 2,000'

Project: 4910 Crater Avenue N Keizer, Oregon	Project No. 24-6649	FIGURE 1

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

SITE AERIAL AND EXPLORATION LOCATIONS

Legend: **TP-1** Test Pit Designation and Approximate Location

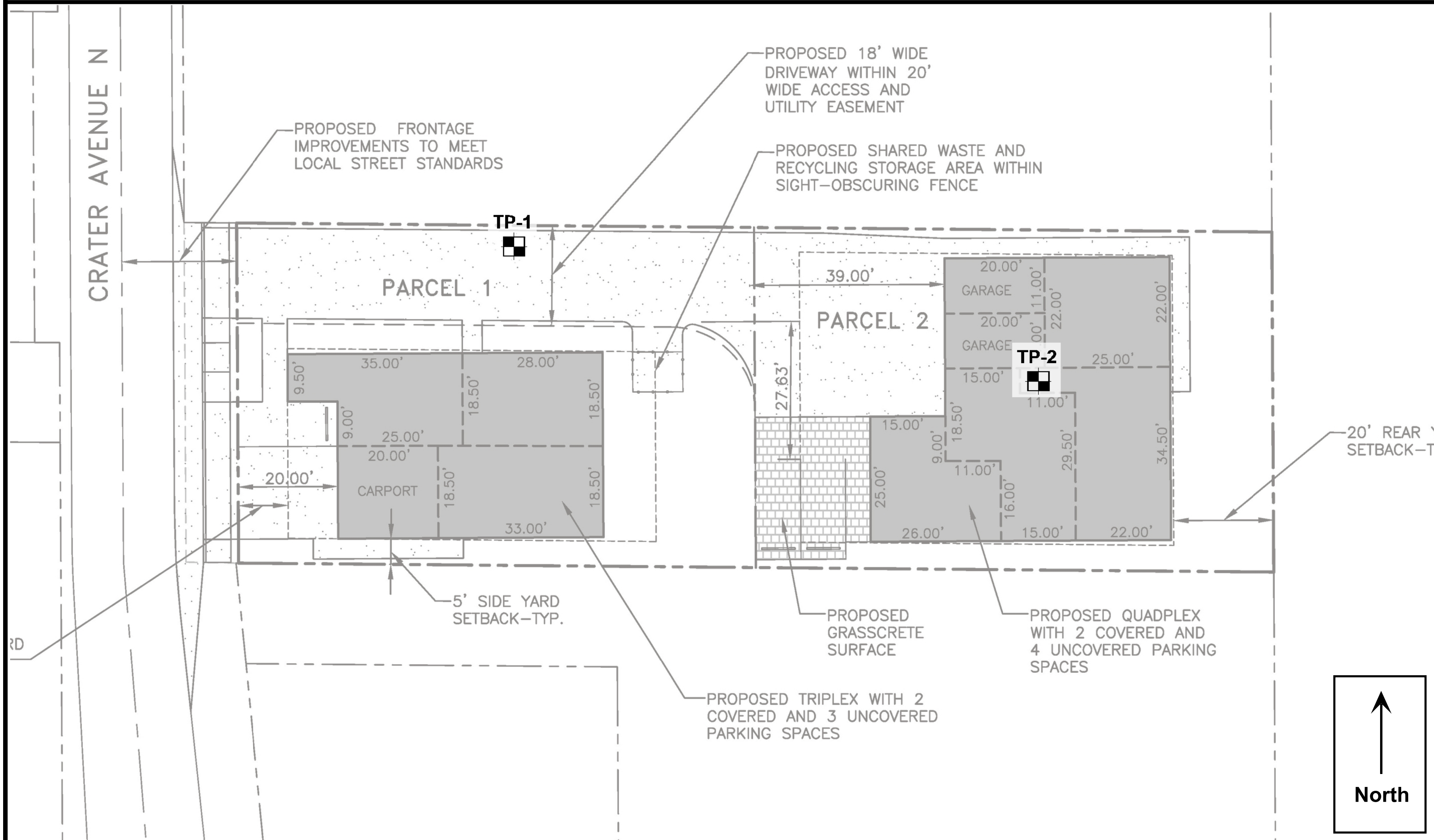
0 50'
APPROXIMATE SCALE 1"=50'

Date: 09/10/24
Drawn by: ABC

Project: 4910 Crater Avenue N Keizer, Oregon	Project No. 24-6649	FIGURE 2

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

SITE PLAN AND EXPLORATION LOCATIONS



Legend: **TP-1** Test Pit Designation and Approximate Location

0 50'
APPROXIMATE SCALE 1"=50'

Date: 09/10/24
Drawn by: ABC

Project: 4910 Crater Avenue N Keizer, Oregon	Project No. 24-6649	FIGURE 3

EXPLORATION LOGS

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: 4910 Crater Avenue N, Keizer, Oregon	Project No. 24-6649	Test Pit No. TP-1

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Torvane Shear (tons/ft^2)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.67							3/4"-0 Crushed Aggregate GRAVEL (GP), gray, 8 inches, dry (Driveway)
0.67–6							Sandy Lean CLAY (ML), brown, stiff, dry to damp (Catastrophic Flood Deposits) Grades to moist Grades to with more sand
6–10							Poorly Graded SAND with Silt (SP), brown, medium dense, moist (Catastrophic Flood Deposits) Grades to with less silt
10							Test pit terminated at 10 feet bgs. No groundwater seepage observed. Excavator: Medium-sized excavator with rock teeth.

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 09/05/24
Logged By: ABC
Surface Elevation: 139 Feet

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: 4910 Crater Avenue N, Keizer, Oregon	Project No. 24-6649	Test Pit No. **TP-2**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Torvane Shear (tons/ft²)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	**Material Description**
							Moderately Organic SILT (OL-ML), brown, organic horizon to 10 inches, dry (Topsoil)
1							SILT (ML), brown, disturbed texture, stiff, dry to damp (Disturbed Native)
2							Sandy Lean CLAY (CL), brown, stiff, dry to damp (Catastrophic Flood Deposits)
							Grades to moist
3							
4				58.4	13.6		Infiltration test conducted at 4 feet. Infiltration rate: 2.88 in/hr
5							
6							Grades to with more sand
7							Poorly Graded SAND with Silt (SP), brown, medium dense, moist (Catastrophic Flood Deposits)
8				8.6	7.2		Infiltration test conducted at 8 feet. Infiltration rate: 2.16 in/hr
9							
10							Test pit terminated at 10 feet bgs. No groundwater seepage observed. Excavator: Medium-sized excavator with rock teeth.
11							
12							
13							
14							
15							
16							
17							

LEGEND

100 to 1,000 g — Bag Sample
5 Gal. Bucket — Bucket Sample
Shelby Tube Sample
Seepage
Water Bearing Zone
Water Level at Abandonment

Date Excavated: 09/05/24
Logged By: ABC
Surface Elevation: 139 Feet

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PORTABLE DYNAMIC CONE PENETROMETER / CALIFORNIA BEARING RATIO CORELATION

Project: Crater Avenue
Project #: 24-6649
Test Designation: PDCP-1
Location: TP-1
Existing A/C Thickness: N/A
Existing B/C Thickness: N/A
Subgrade: Sandy Lean CLAY (CL)

Date: 9/5/2024
Recorded by: ABC

Portable Dynamic Cone Penetrometer: KSE DCP K-100 Model, ASTM D6951, 17.6 lbs Hammer

Shaft Length (in)	Start Depth (in)	Initial Reading (mm)	Initial Reading (in)	Resilient Modulus Correction Factor
36	0	30.48	1.2	0.35

Blows	Depth (In)	Depth (ft)	Depth (mm)	mm/blow	Correlated CBR	Correlated PSI
1	2.5	0.21	63.50	33.0	5.7	4387
1	3.4	0.28	86.36	22.9	8.7	5063
1	3.7	0.31	93.98	7.6	29.7	7772
1	4.1	0.34	104.14	10.2	21.5	6947
1	4.3	0.36	109.22	5.1	46.7	9103
5	5	0.42	127.00	3.6	69.7	10461
5	5.8	0.48	147.32	4.1	60.0	9931
5	6.5	0.54	165.10	3.6	69.7	10461
5	7.5	0.63	190.50	5.1	46.7	9103
5	8.9	0.74	226.06	7.1	32.0	7983
5	10.4	0.87	264.16	7.6	29.7	7772
5	12	1.00	304.80	8.1	27.6	7578
5	14	1.17	355.60	10.2	21.5	6947
5	16.5	1.38	419.10	12.7	16.7	6368
5	19.9	1.66	505.46	17.3	11.9	5648
5	25.4	2.12	645.16	27.9	6.9	4682
5	31.4	2.62	797.56	30.5	6.3	4526
2	35.5	2.96	901.70	52.1	3.4	3673
			Average	14.9	14.0	5981

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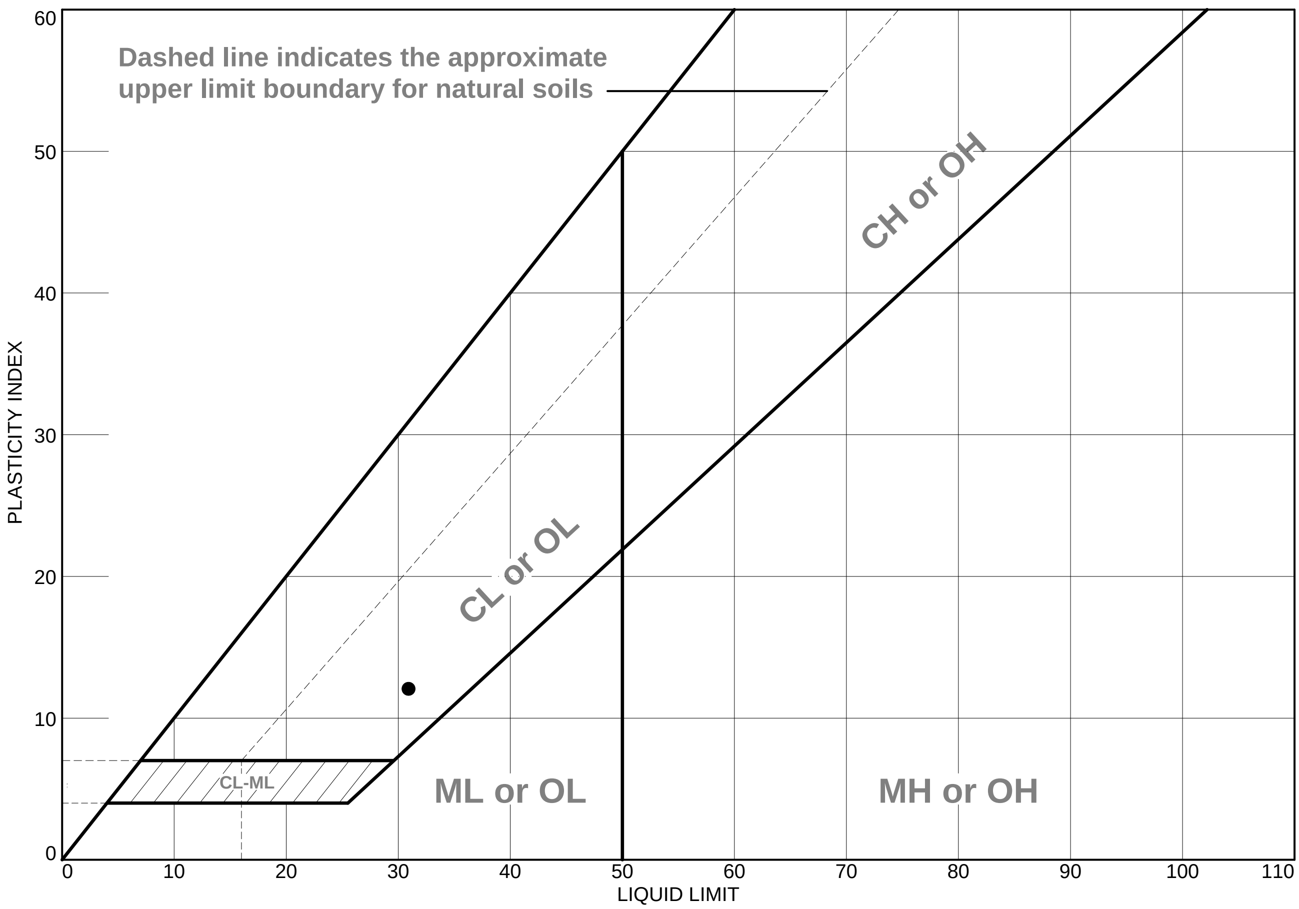
LABORATORY TEST RESULTS

LIQUID AND PLASTIC LIMITS TEST REPORT



	Material Description	Sampled	Tested	Technician	LL	PL	PI	%<#40	USCS
●	Sandy Lean CLAY	ABC 09/05/24		ALL	31	19	12	96.6	CL
■	Poorly Graded SAND with Silt	ABC 09/05/24		ALL	NV	NP	NP	78.6	SP-SM

Project No. 24-6649 **Client:** Banner Homes

Project: 4910 Crater Avenue N Keizer

○**Location:** TP-1 **Depth:** 4ft **Sample Number:** S24-140

□**Location:** TP-1 **Depth:** 8ft **Sample Number:** S24-141

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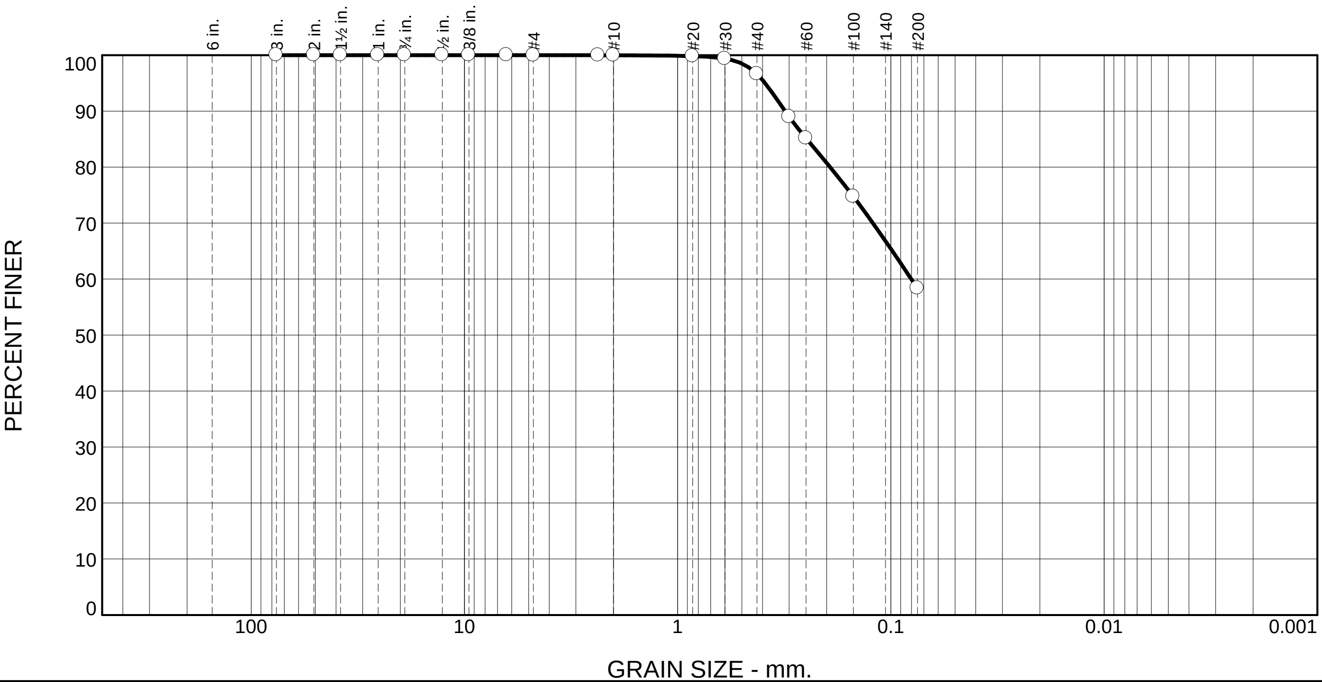
Checked by:

Title:

Figure

Tested By: ALL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	3.4	38.2	58.4	

Test Results (AASHTO T 27 & AASHTO T 11)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3"	100.0		
2"	100.0		
1.5"	100.0		
1"	100.0		
3/4"	100.0		
1/2"	100.0		
3/8"	100.0		
1/4"	100.0		
#4	100.0		
#8	100.0		
#10	100.0		
#20	99.8		
#30	99.4		
#40	96.6		
#50	89.0		
#60	85.2		
#100	74.7		
#200	58.4		

* (no specification provided)

Material Description

Sandy Lean CLAY

Atterberg Limits (ASTM D 4318)

PL= 19 **LL=** 31 **PI=** 12

Classification

USCS (D 2487)= CL **AASHTO (M 145)=** A-6(4)

Coefficients

D_{90}= 0.3136 D_{85}= 0.2478 D_{60}= 0.0802
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture Content = 13.6 Percent

Date Received: **Date Tested:** 09/24/24

Tested By: ALL

Checked By: MTB

Title:

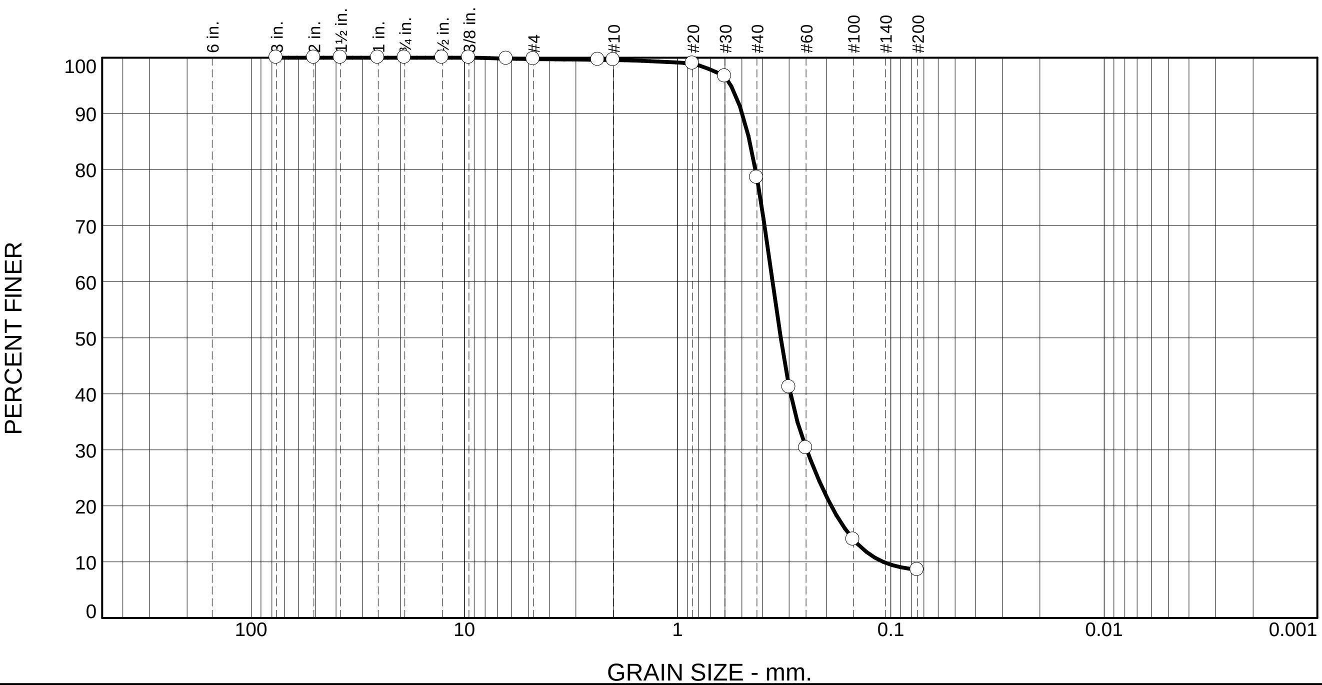
Location: TP-1 **Date Sampled:** ABC 09/05/
Sample Number: S24-140 **Depth:** 4ft

GEOPACIFIC ENGINEERING, INC.

Client: Banner Homes
Project: 4910 Crater Avenue N Keizer

Project No: 24-6649 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.3	0.1	21.0	70.0	8.6	

Test Results (AASHTO T 27 & AASHTO T 11)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3"	100.0		
2"	100.0		
1.5"	100.0		
1"	100.0		
3/4"	100.0		
1/2"	100.0		
3/8"	100.0		
1/4"	99.8		
#4	99.7		
#8	99.6		
#10	99.6		
#20	99.0		
#30	96.7		
#40	78.6		
#50	41.2		
#60	30.4		
#100	14.0		
#200	8.6		

* (no specification provided)

Material Description

Poorly Graded SAND with Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 0.4975 D_{85}= 0.4592 D_{60}= 0.3588
D_{50}= 0.3286 D_{30}= 0.2480 D_{15}= 0.1574
D_{10}= 0.1086 C_u= 3.31 C_c= 1.58

Remarks

Moisture Content = 7.2 Percent

Date Received: **Date Tested:** 09/24/24

Tested By: ALL

Checked By: MTB

Title:

Location: TP-1
Sample Number: S24-141 **Depth:** 8ft
Date Sampled: ABC 09/05/

GEOPACIFIC ENGINEERING, INC.

Client: Banner Homes
Project: 4910 Crater Avenue N Keizer
Project No: 24-6649 **Figure**

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PAVEMENT DESIGN CALCULATIONS

```
==================================================

DARWin(tm) - Pavement Design

A Proprietary AASHTOWARE(tm)
Computer Software Product

---------------------------------------

Flexible Structural Design Module

--------------------------------------------------

Project Description
   24-6649 4910 Crater Avenue N - Private Interior Roadways

Flexible Structural Design Module Data
   18-kip ESALs Over Initial Performance Period: 50,000
                      Initial Serviceability: 4.2
                     Terminal Serviceability: 2.5
                       Reliability Level (%): 90
                 Overall Standard Deviation: .5
          Roadbed Soil Resilient Modulus (PSI): 5,900
                          Stage Construction: 1

                  Calculated Structural Number: 2.38

Specified Layer Design
                                       Layer: 1
                        Material Description: New Asphalt
                   Structural Coefficient (Ai): .44
                     Drainage Coefficient (Mi): 1
                     Layer Thickness (Di) (in): 3.00
                           Calculated Layer SN: 1.32

                                       Layer: 2
                        Material Description: 3/4"-0 Crushed Rock
                   Structural Coefficient (Ai): .12
                     Drainage Coefficient (Mi): 1
                     Layer Thickness (Di) (in): 2.00
                           Calculated Layer SN: .24

                                       Layer: 3
                        Material Description: 1-1/2"-0 Crushed Rock
                   Structural Coefficient (Ai): .12
                     Drainage Coefficient (Mi): 1
                     Layer Thickness (Di) (in): 8.00
                           Calculated Layer SN: .96

                        Total Thickness (in): 13.00
                         Total Calculated SN: 2.52
```

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SITE RESEARCH

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address: 4910 Crater Ave N, Keizer, OR 97303, USA

Coordinates: 44.9954145, -123.0359569

Elevation: 139 ft

Timestamp: 2024-09-18T21:42:08.402Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D

Basic Parameters

Name	Value	Description
S_S	0.834	MCE_R ground motion (period=0.2s)
S_1	0.416	MCE_R ground motion (period=1.0s)
S_{MS}	0.973	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.648	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.167	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.879	Coefficient of risk (0.2s)
CR_1	0.865	Coefficient of risk (1.0s)
PGA	0.387	MCE_G peak ground acceleration
F_{PGA}	1.213	Site amplification factor at PGA
PGA_M	0.47	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
SsRT	0.834	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.948	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.416	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.48	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.665	Factored deterministic acceleration value (1.0s)
PGAd	0.549	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.55155279 g

Recovered targets

Return period: 2450.1577 yrs
Exceedance rate: 0.000408137 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.43 %

Mean (over all sources)

m: 8.3
r: 67.06 km
ε_0: 0.98 σ

Mode (largest m-r bin)

m: 9.34
r: 61.27 km
ε_0: 0.35 σ
Contribution: 16.26 %

Mode (largest m-r-ε_0 bin)

m: 8.86
r: 61.23 km
ε_0: 0.7 σ
Contribution: 12.37 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
sub0_ch_bot.in	Interface							39.69
Cascadia Megathrust - whole CSZ Characteristic		61.27	9.10	0.53	123.702°W	45.000°N	270.79	39.69
sub0_ch_mid.in	Interface							14.63
Cascadia Megathrust - whole CSZ Characteristic		109.67	8.92	1.39	124.356°W	44.742°N	255.33	14.63
coastalOR_deep.in	Slab							8.07
coastalOR_deep.in	Slab							6.49
sub0_ch_top.in	Interface							3.70
Cascadia Megathrust - whole CSZ Characteristic		121.20	8.83	1.58	124.561°W	45.000°N	270.78	3.70
sub2_ch_bot.in	Interface							3.62
Cascadia Megathrust - Goldfinger Case C Characteristic		60.91	8.73	0.78	123.702°W	45.000°N	270.79	3.62
WUSmap_2014_fixSm.ch.in (opt)	Grid							1.95
noPuget_2014_fixSm.ch.in (opt)	Grid							1.95
WUSmap_2014_fixSm.gr.in (opt)	Grid							1.92
noPuget_2014_fixSm.gr.in (opt)	Grid							1.92
sub1_ch_bot.in	Interface							1.81
Cascadia Megathrust - Goldfinger Case B Characteristic		60.75	8.86	0.69	123.702°W	45.000°N	270.79	1.81
sub1_GRb0_bot.in	Interface							1.63
Cascadia floater over southern zone - Goldfinger Case B		64.29	8.47	1.00	123.702°W	45.000°N	270.79	1.63
sub1_GRb1_bot.in	Interface							1.23
Cascadia floater over southern zone - Goldfinger Case B		64.90	8.35	1.08	123.702°W	45.000°N	270.79	1.23

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PHOTOGRAPHIC LOG

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Test Pit TP-1

Test Pit TP-4

APPENDIX E –

OPERATION AND MAINTEANCE MANUAL

WILL BE PROVIDED WITH FINAL REPORT