PRELIMINARY TECHNICAL INFORMATION REPORT

Oak Tree Station

City of Camas, Washington

Prepared for:

KYNE ROB & BONIFE LARRY

3239 NW HOOD CT

CAMAS WA , 98607

Prepared by:

ENGINEERING NORTHWEST PLLC

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7504 NW 10th Avenue

Vancouver, WA 98665

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ENGINEERING NORTHWEST

CERTIFICATION OF ENGINEERING

The technical information and data contained in this report was prepared under the direction and supervision of the undersigned, whose seal, as a professional engineer licensed to practice as such, is affixed below. This TIR meets the minimum requirements of the City of Camas stormwater ordinances and 2019 Stormwater Management Manual for Western Washington.

Contents

SECTION A – PROJECT OVERVIEW

Section A.1 Site Information

The proposed Oak Tree Station food court will be located on the northeast quadrant of the NW Lake Road and NW Friberg-Strunk Street cross section in the City of Camas, Washington (See Vicinity Map). It is on parcel number 176162000. The proposed project will construct a building that will act as a food court, a parking lot, sidewalks, a drive thru coffee kiosk, a patio, concrete pads for the food carts, landscaping, and a storm water facility.

The proposed storm design will meet the requirements of the City of Camas and 2019 Stormwater Management Manual for Western Washington.

Topography

The project site is approximately 3.95 acres in size. The site has modest slopes (0-5%). The highest elevation of 250 feet is located along the west side of the parcel. The lowest elevation of 244 feet is located along the northeast corner of the property.

Hydrology

Stormwater runoff sheet flows from approximately at the high point which is on the east property line. From the high point the stormwater runoff sheet flows west to the adjacent property.

Basin Areas

Impervious and pervious surface areas for the pre-developed, existing and post-developed conditions site are shown in Table A.1. The impervious area includes roofs, the parking lot, sidewalks, the concrete pads, and the concrete patio while pervious area includes landscaping. Pre-Developed conditions are defined as forested.

Due to soil properties in this area, it is unlikely stormwater management by infiltration BMPs will be applicable.

Section B – Minimum Requirements

Section B.1 – Determination of Minimum Requirements

Proposed land disturbance will consist of two buildings, concrete pads, a concrete patio and parking lot. Since the amount of proposed hard surfaces is more than 5,000 square feet, this project is required to meet Minimum Requirements 1-9

TABLE A.1: THRESHOLD DISCHARGE AREA ID Basin 1	Square Feet
Existing hard Surface	*0*
New hard surfaces	*58,048*
Replaced hard surfaces	*0*
Native vegetation converted to lawn/landscape	*25,530*
Native vegetation converted to pasture	*0*
Total land disturbing activity	*84,000*
New and replaced pollution generating hard surfaces (PGHS)	*47,868*
Non-pollution generating surface	*10,180*

The developed basin's effective hard surfaces and their applicability to meeting the Minimum Requirements 6-8 are summarized in Table A.2 below.

TABLE A.2 EFFECTIVE HARD SURFACES			
Hard Surface Area	MR #6 Required (Y/N)	MR#7 Required (Y/N)	MR#8 Required (Y/N)
47,868	Y	Y	N

City of City of Camas stormwater ordinances requires that the project discharge less than the pre-development rate. The pre-development site was modeled in WWHM2012 as forested condition for the proposed project disturbed area.

Section C -Soils Evaluation

The "Soil Survey of Clark County. Washington" indicates the soil at this site consist of the following:

(DoB) Dollar loam, 0 to 5 percent slopes.

Clark County GIS indicates that the site soils are designed as Soil Group 4 – Poorly Drained Soils for use with the Western Washington Hydrology Model (WWHM2012).

Onsite native soil is not suitable for infiltration, thus infiltration testing was not performed onsite.

Section D – Source Control

There are not any prohibited discharges planned for this site. A stormwater pollution plan (SWPPP) will be prepared for the final technical report for the project. The SWPPP will identify and list BMPs for Source Control and will include BMPs to prohibit sediment-laden runoff from leaving the site and impacting any local or State water. In addition, BMPs will be implemented as necessary to prevent pollutants from coming in contact with the stormwater system.

The proposed site is being developed with activities that are pollution generating. The following BMP categories have some degree of applicability, in particular, BMPs for Landscaping and Lawn/Vegetation Management and Maintenance of Stormwater Drainage and Treatment System.

All source control BMPs in the private street will be the responsibility of the parcel owners per their established maintenance procedures. The stormwater facilities will be privately owned and maintained in a manner consistent with the Stormwater Facility Maintenance Manual and BMPs for landscaping and Lawn/Vegetation Management.

The parcel owners will also be responsible for source control BMPs related to installing and maintaining landscaping and roof downspout systems on this parcel. This responsibility includes the prevention of introduction of pollutants into their system(s). The application of appropriate maintenance measures will also provide source control.

Additional Reference: SMMWW, Volume IV, Chapter 2 - Selection of Operational and Structural Source Control BMPs; 2.2 Pollutant Source-Specific BMPs

BMPs for Dust Control at Disturbed Land Areas and Unpaved Roadways and Parking Lots

BMPs for Landscaping and Lawn/ Vegetation Management

BMPs for Maintenance of Stormwater Drainage and Treatment Systems BMPs for Urban Streets

Section E - Onsite Stormwater Management BMPs

An Erosion Control Plan will be developed for the implementation of BMPs to manage stormwater during grading activities. These BMPs will be shown on the erosion control plan.

The parcel owners will be responsible for installing and maintaining roof downspout systems consistent with Volume III, Chapter 3.1.1 of the SMMWW.

Section F – Runoff Treatment and Design

1) Basic stormwater treatment is required for the parking lot in this project.

2) Phosphorous removal is also required.

The runoff from areas requiring treatment will be routed to specific Old Castle PerkFilter Systems (or Contech Stormfilter Systems). The systems will be off-line in nature and be sized to treat the off-line flow rate determined from a WWHM2012 analysis.

The management of flows above the WQ flow rates will be directed to the particular storm line system. The existing site will release to a stormwater pipe in NW Lake Road that will take the runoff to a public basin on the adjacent, east parcel.

Initial installation cost and the expenses associated with long-term maintenance are expected to be typical of projects with similar street sections and with runoff from parking lots.

The amount of pollution-generating surfaces is:

From Parking Lot = 1.1 acres

Section G - Flow Control Analysis and Design

The project proposes a large vault to store clean stormwater runoff. A control manhole will meter the stormwater from the storage vault and slowly release runoff to the stormwater pipe in NW Lake Road that will discharge into the public basin on the parcel to the east. The WWHM model for the project show flow duration has passed.

Section I - Wetlands Protection

No wetland in the area.

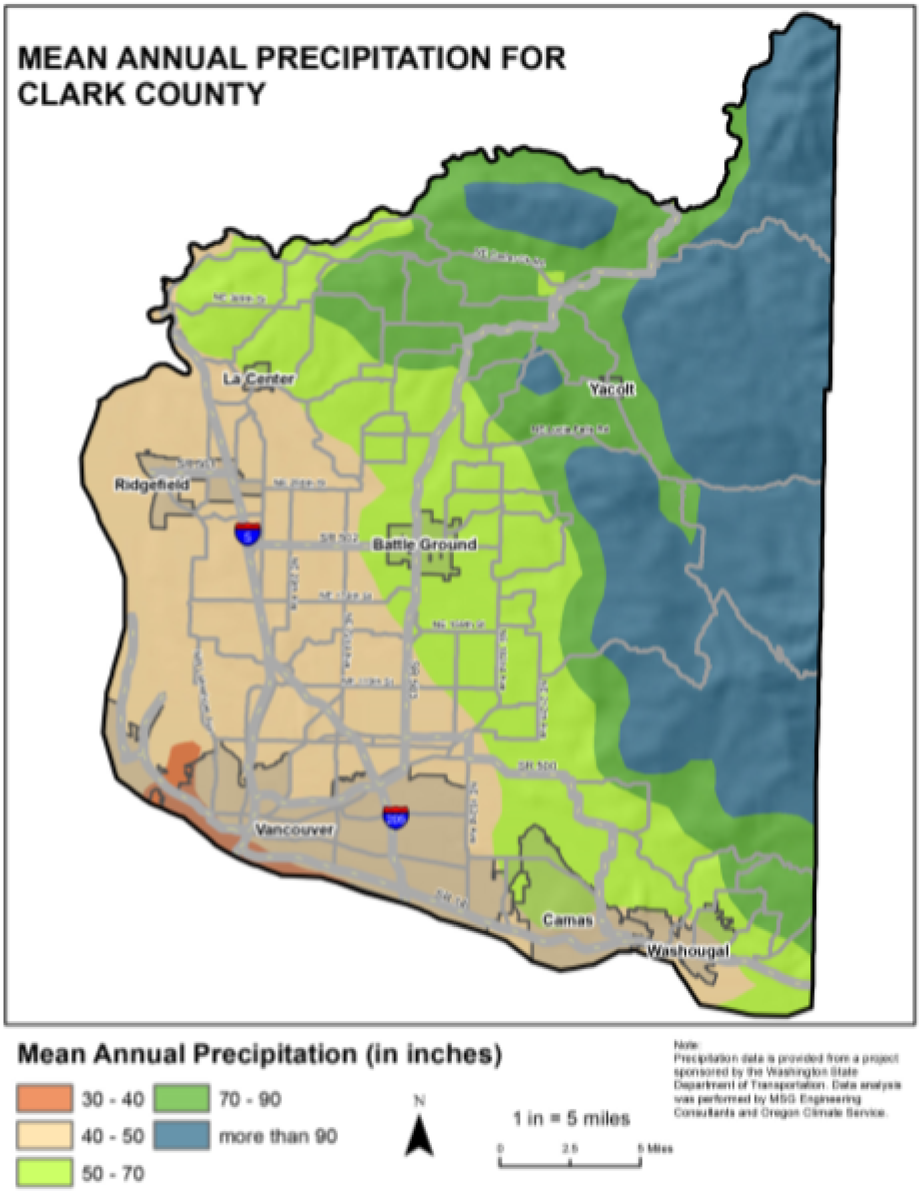
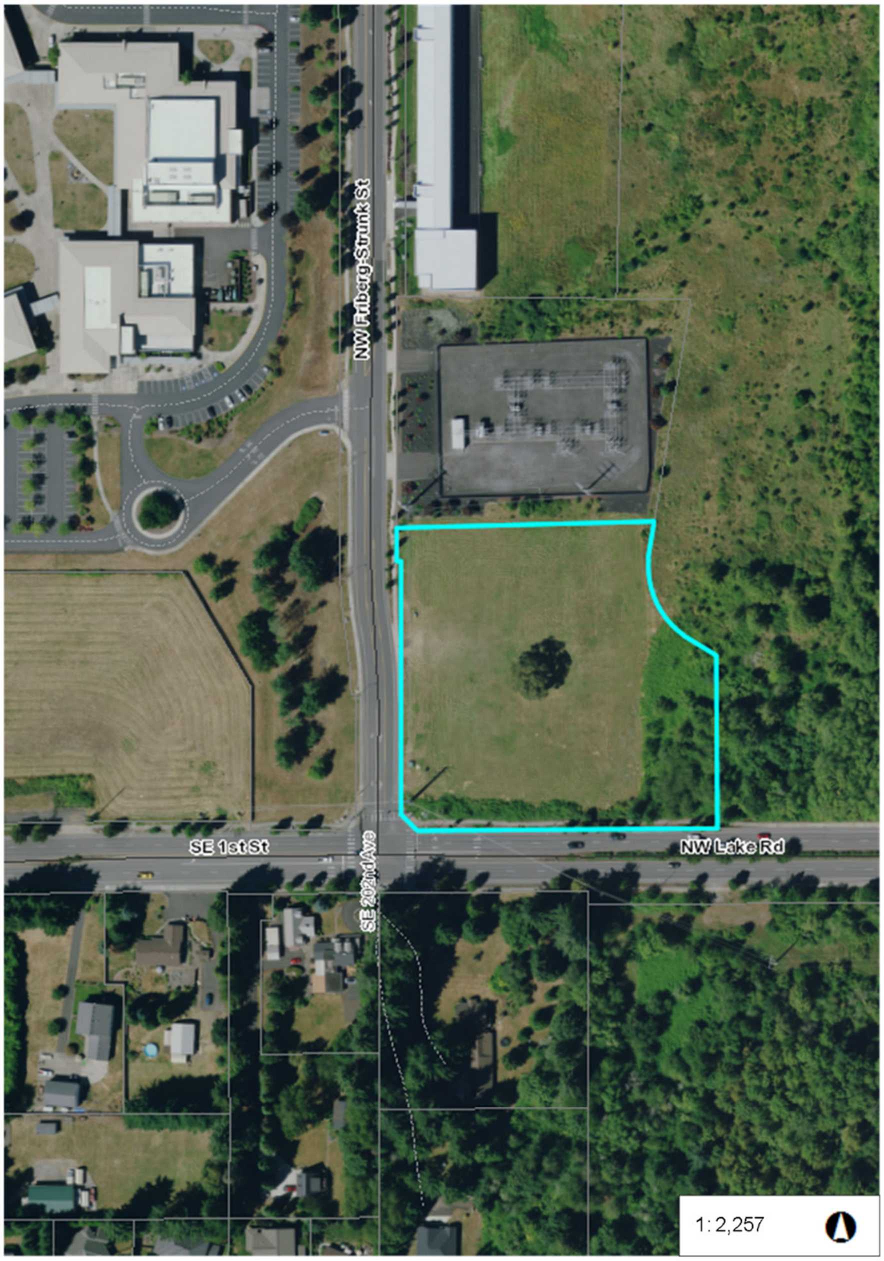
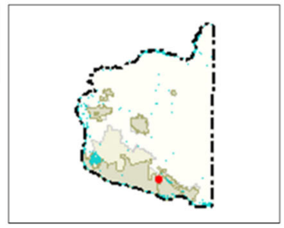


Figure 15: Mean Annual Precipitation in Clark County

APPENDIX A: MAPS

AERIAL





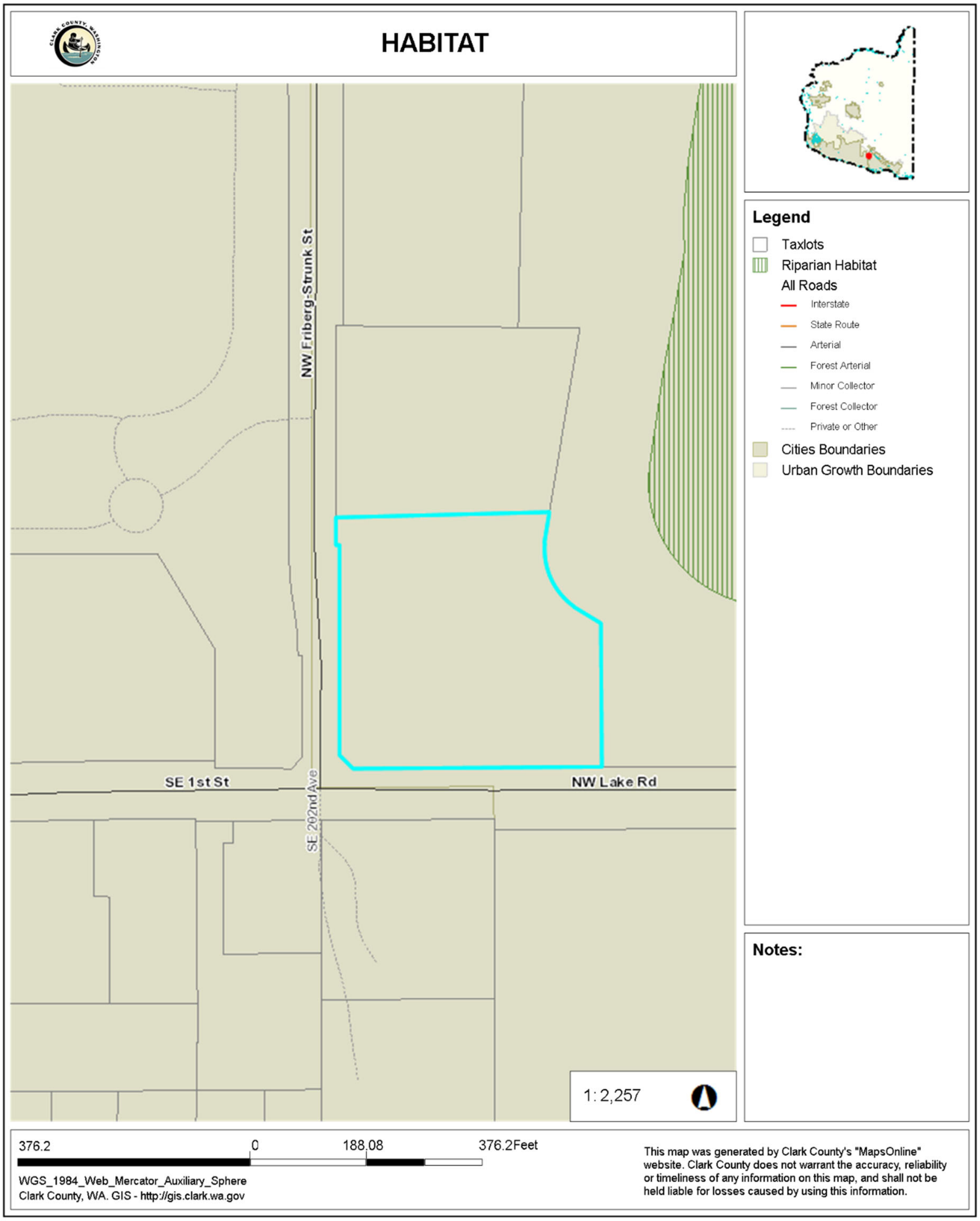
Legend

- Taxlots
- All Roads
  - Interstate
  - State Route
  - Arterial
  - Forest Arterial
  - Minor Collector
  - Forest Collector
  - Private or Other
- Cities Boundaries
- Urban Growth Boundaries

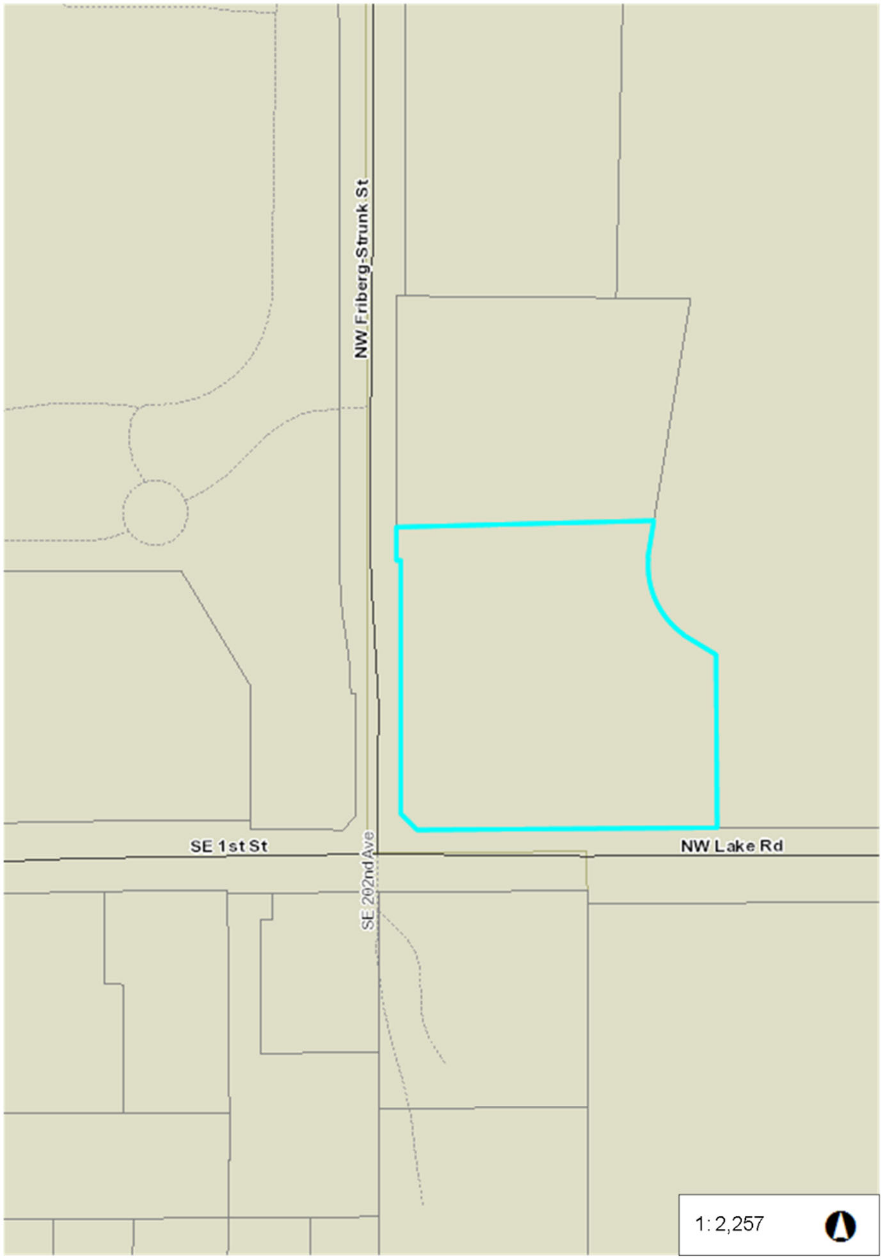
Notes:

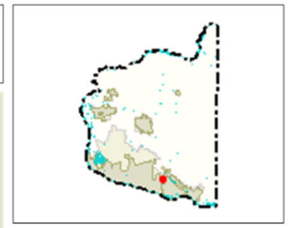
376.2 0 188.08 376.2Feet

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


HABITAT
NW Friberg-Strunk St
SE 1st St
NW Lake Rd
SE 202nd Ave
1: 2,257
Legend
Taxlots
Riparian Habitat
All Roads
Interstate
State Route
Arterial
Forest Arterial
Minor Collector
Forest Collector
Private or Other
Cities Boundaries
Urban Growth Boundaries
Notes:
376.2
0
188.08
376.2Feet
WGS_1984_Web_Mercator_Auxiliary_Sphere
Clark County, WA. GIS - http://gis.clark.wa.gov
This map was generated by Clark County's "MapsOnline" website. Clark County does not warrant the accuracy, reliability or timeliness of any information on this map, and shall not be held liable for losses caused by using this information.

SEVERE EROSION HAZARD AREAS





Legend

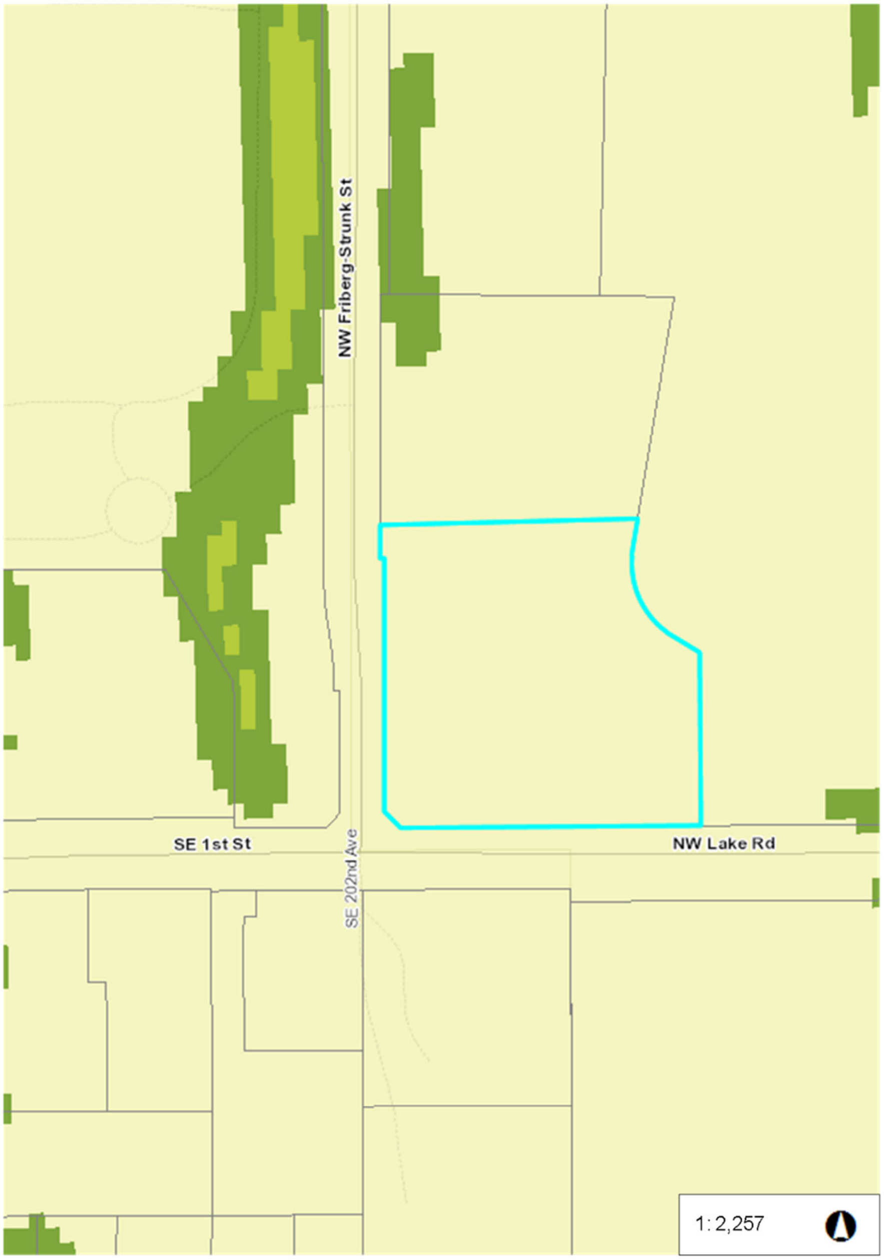
- Taxlots
- Severe Erosion Hazard Areas
- All Roads
  - Interstate
  - State Route
  - Arterial
  - Forest Arterial
  - Minor Collector
  - Forest Collector
  - Private or Other
- Cities Boundaries
- Urban Growth Boundaries

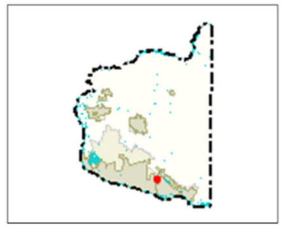
Notes:

376.2 0 188.08 376.2Feet

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

CONTOURS & SLOPES





Legend

- Taxlots
- Percent Slope
  - 0-5 percent
  - >5-10 percent
  - >10-15 percent
  - >15-25 percent
  - >25-40 percent
  - >40-80 percent
  - >80 percent
- All Roads
  - Interstate
  - State Route
  - Arterial
  - Forest Arterial
  - Minor Collector
  - Forest Collector
  - Private or Other
- Cities Boundaries
- Urban Growth Boundaries

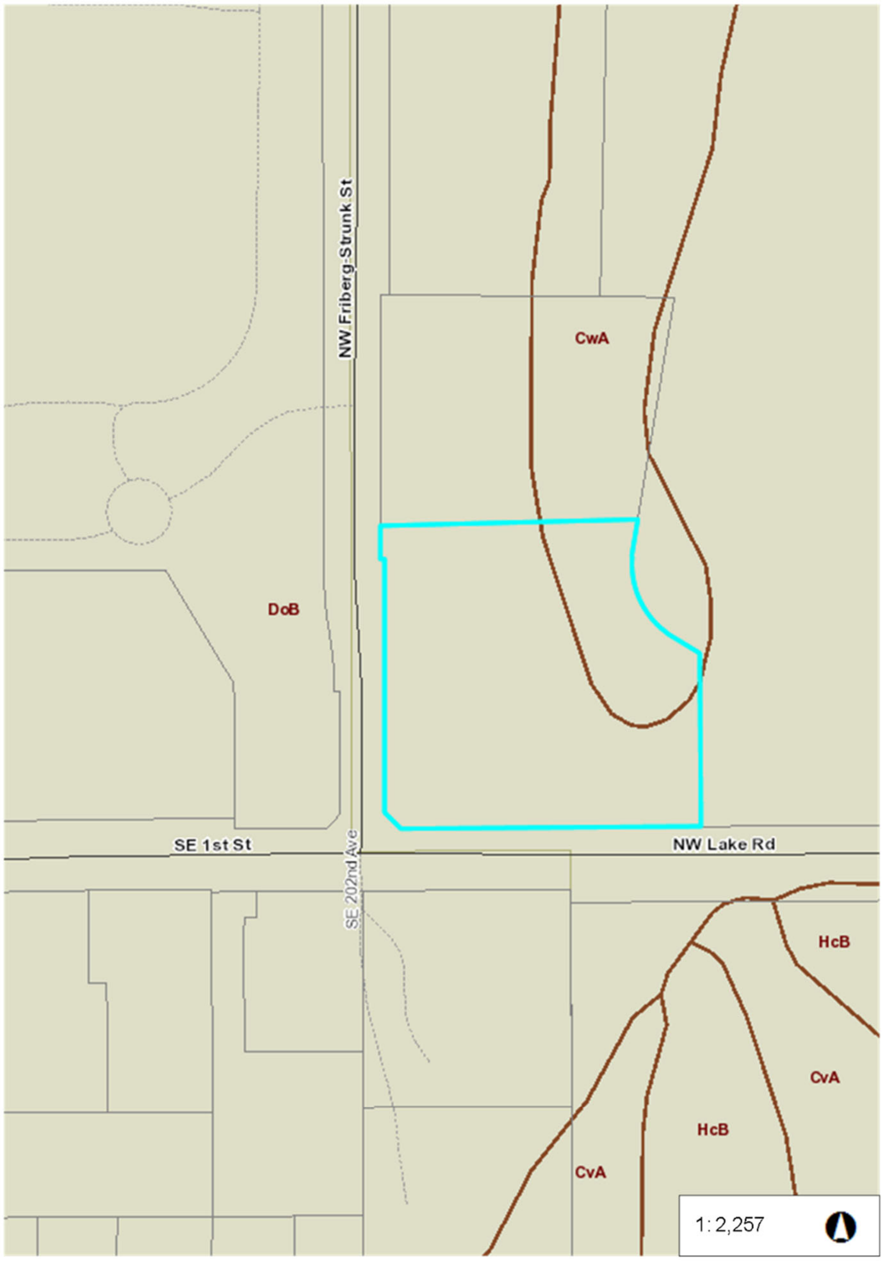
Notes:

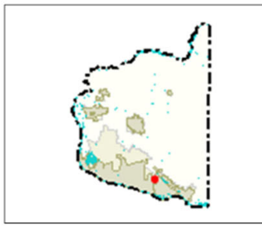
376.2 0 188.08 376.2Feet

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

Exhibit 15 CUP22-01

SOIL





Legend

- Taxlots
- Soil Type

All Roads

- Interstate
- State Route
- Arterial
- Forest Arterial
- Minor Collector
- Forest Collector
- Private or Other

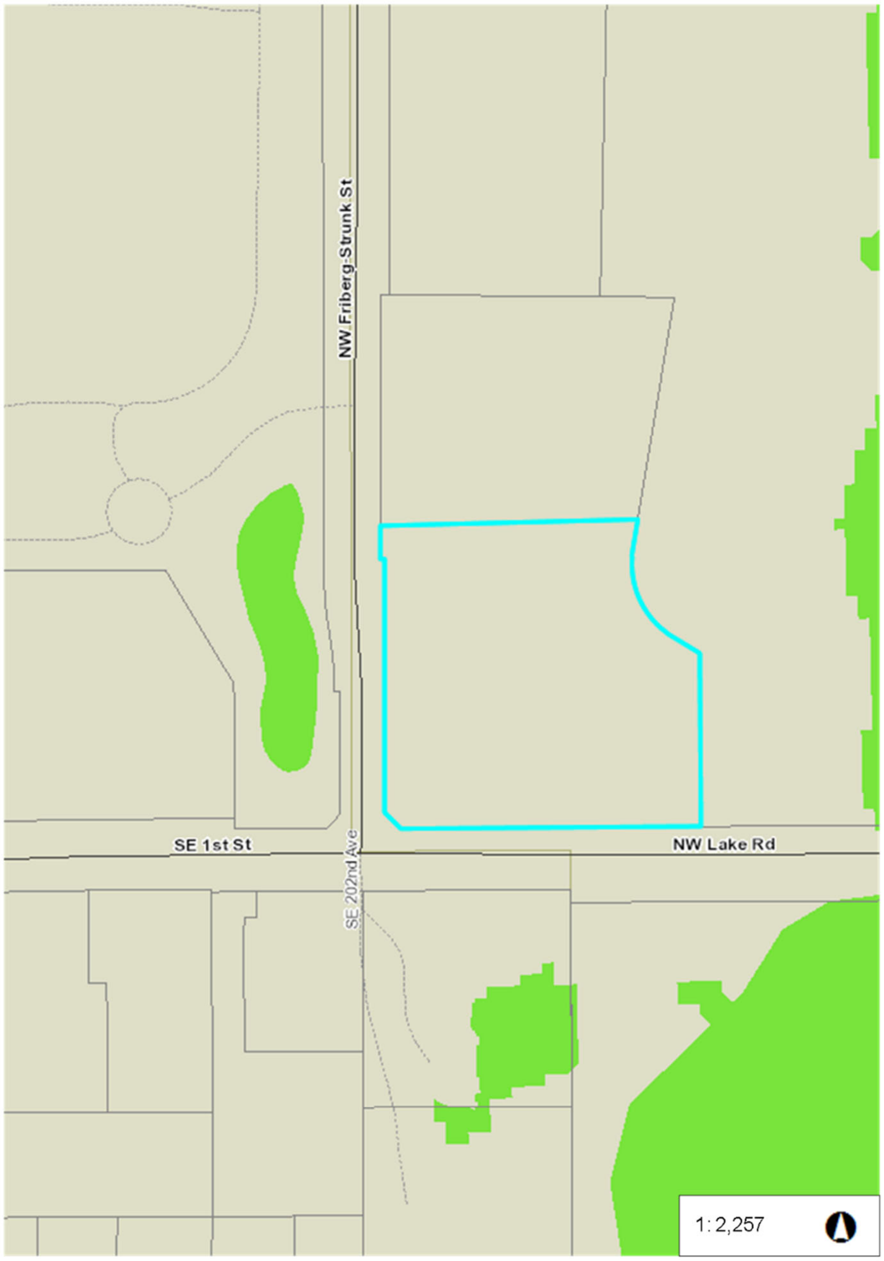
- Cities Boundaries
- Urban Growth Boundaries

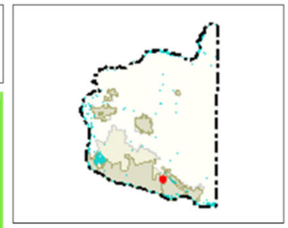
Notes:

376.2 0 188.08 376.2Feet

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

WETLANDS





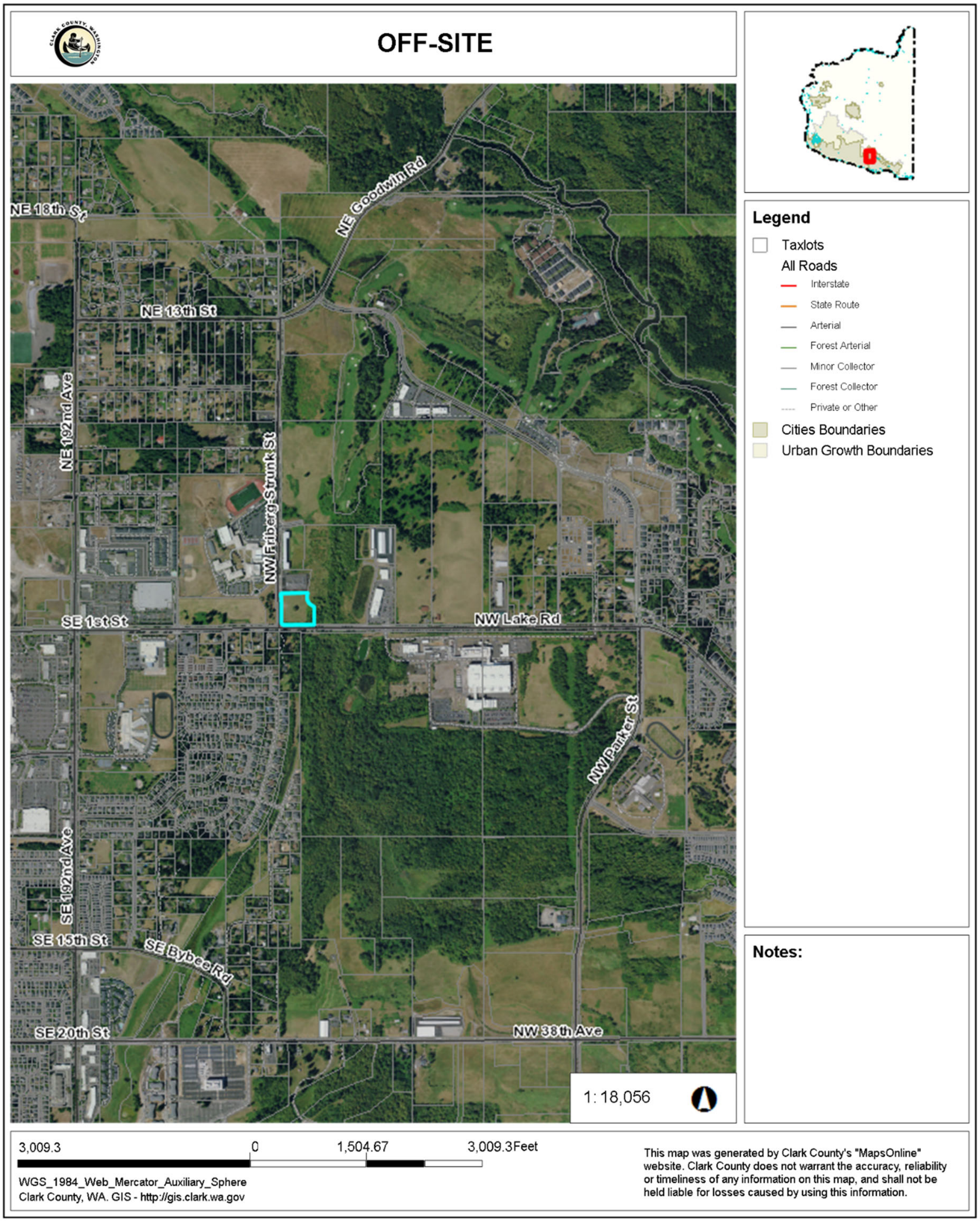
Legend

- Taxlots
- Potential Wetlands Presence
- All Roads
  - Interstate
  - State Route
  - Arterial
  - Forest Arterial
  - Minor Collector
  - Forest Collector
  - Private or Other
- Cities Boundaries
- Urban Growth Boundaries

Notes:

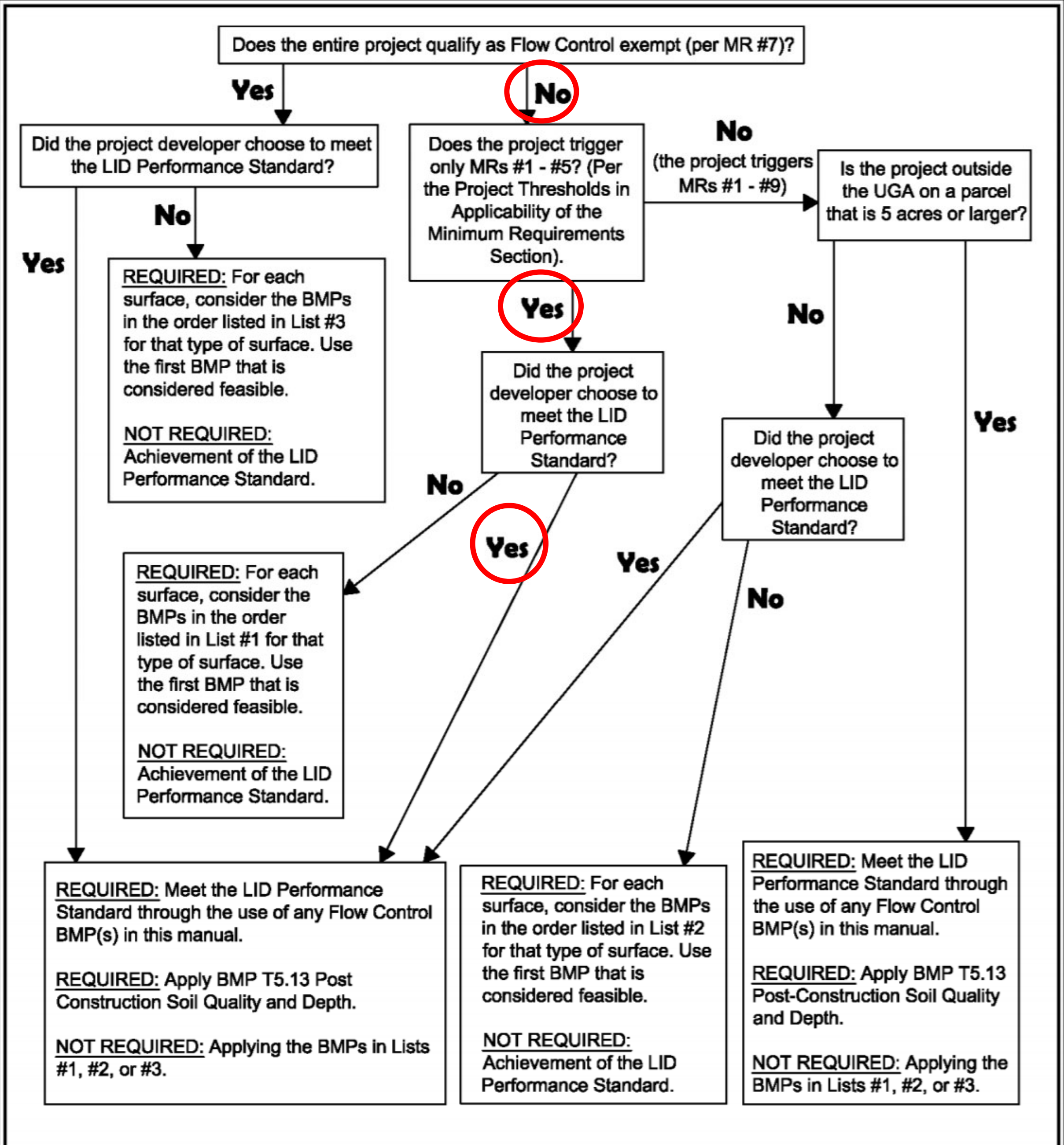
376.2 0 188.08 376.2Feet

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


OFF-SITE
NE 18th St
NE Goodwin Rd
NE 13th St
NE 192nd Ave
NW Friberg-Strunk St
SE 1st St
NW Lake Rd
NW Parker St
SE 192nd Ave
SE 15th St
SE Bybee Rd
SE 20th St
NW 38th Ave
1: 18,056
Legend
Taxlots
All Roads
Interstate
State Route
Arterial
Forest Arterial
Minor Collector
Forest Collector
Private or Other
Cities Boundaries
Urban Growth Boundaries
Notes:
3,009.3
0
1,504.67
3,009.3Feet
WGS_1984_Web_Mercator_Auxiliary_Sphere
Clark County, WA. GIS - http://gis.clark.wa.gov
This map was generated by Clark County's "MapsOnline" website. Clark County does not warrant the accuracy, reliability or timeliness of any information on this map, and shall not be held liable for losses caused by using this information.

Off-site Map

APPENDIX B: FLOW CHARTS

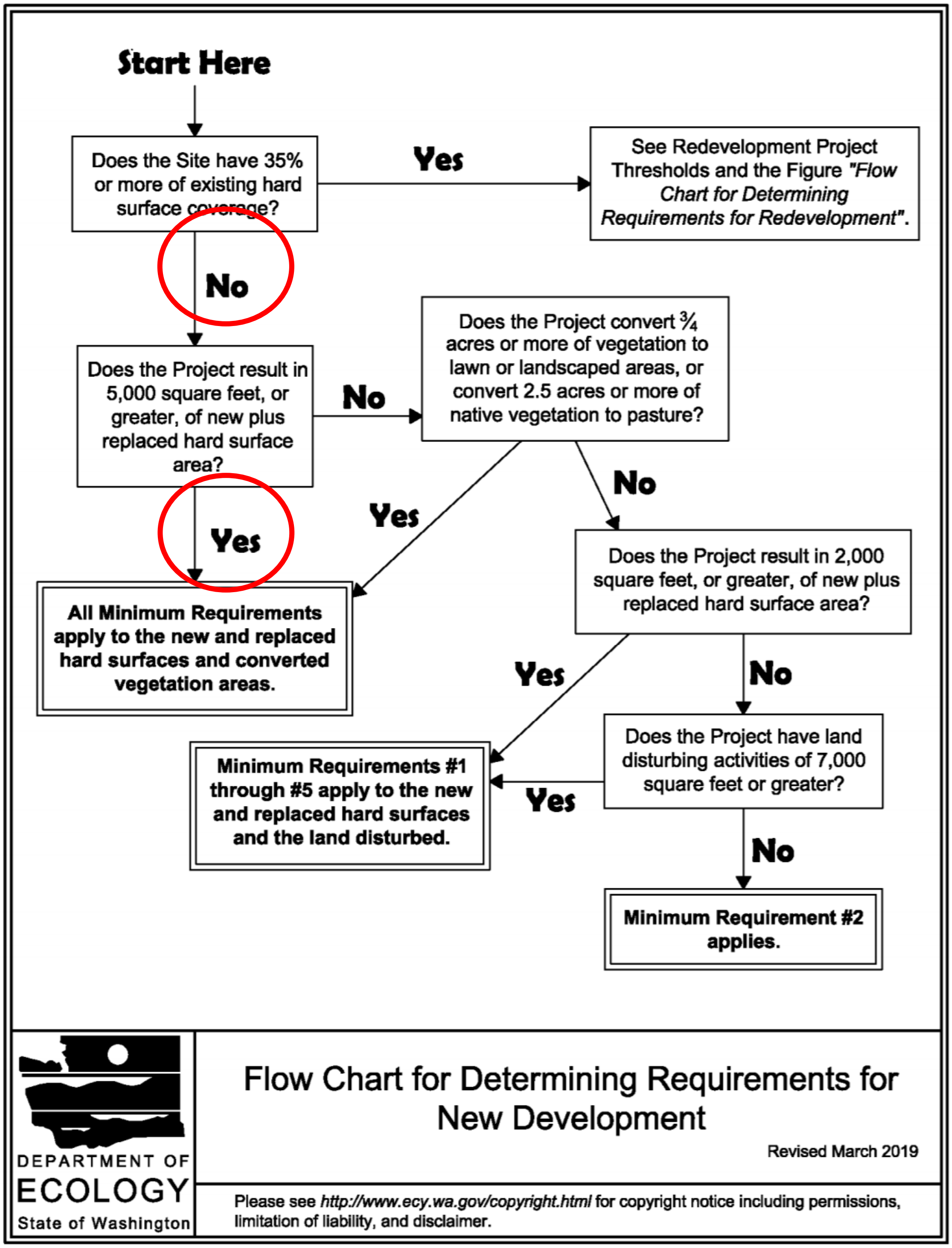


DEPARTMENT OF ECOLOGY
State of Washington

Flow Chart for Determining MR #5 Requirements

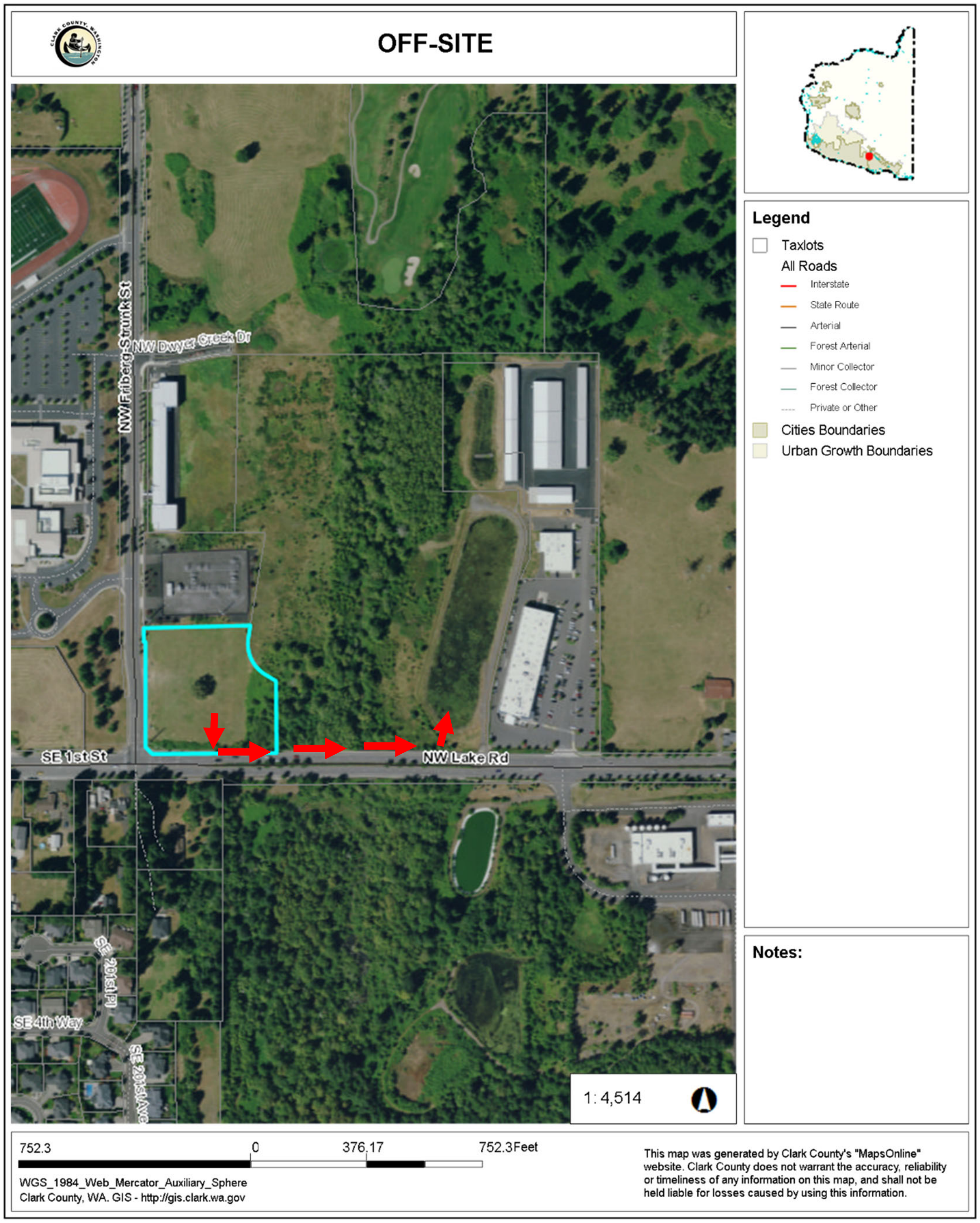
Revised March 2019

Development



Minimum Requirement #8 Flow Chart

APPENDIX C: NATURAL DRAINAGE FLOW PATTERN

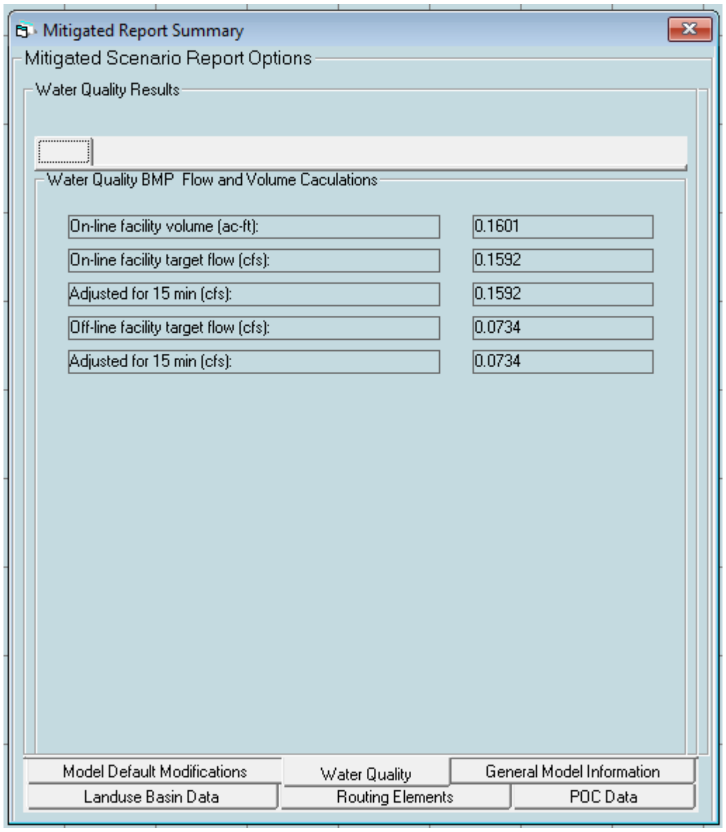


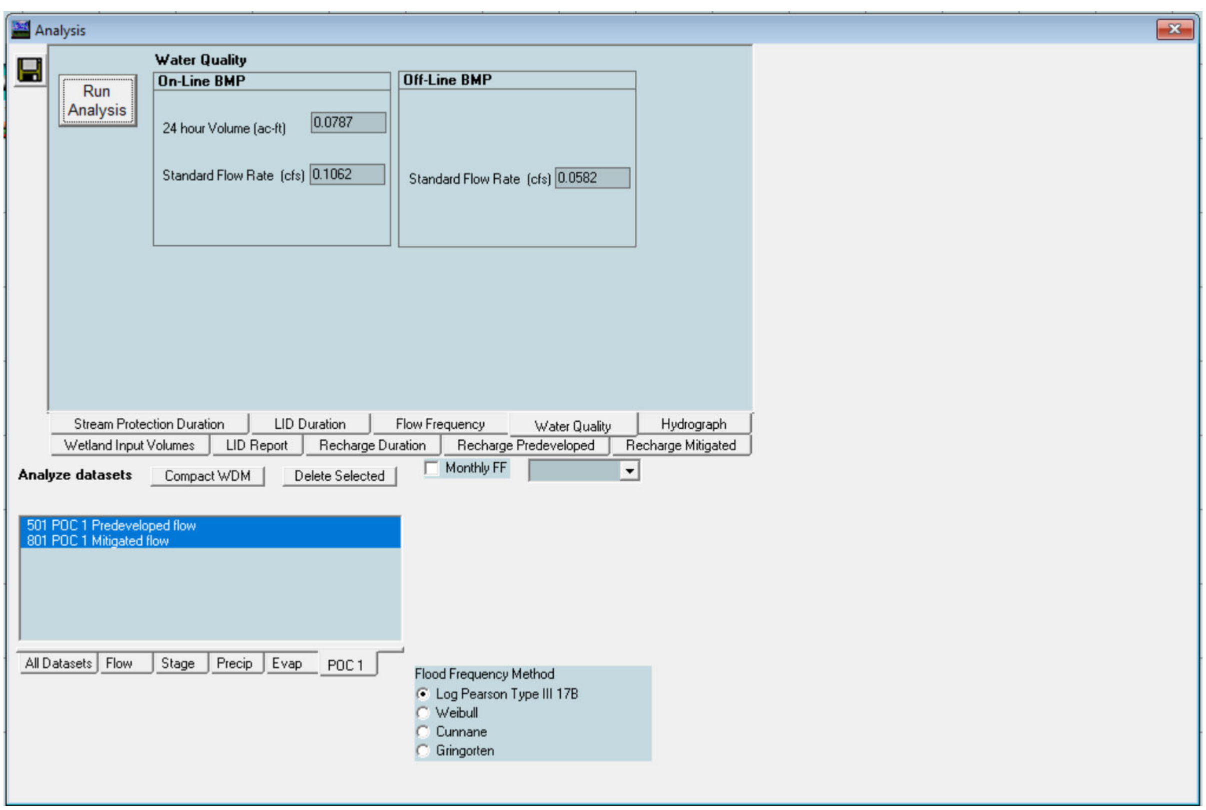
DRAINAGE FLOW PATH

Natural Drainage Flow

APPENDIX D: WWHM SCREEN SHOT

(WATER QUALITY SYSTEM DESIGN)


Mitigated Report Summary
Mitigated Scenario Report Options
Water Quality Results
Water Quality BMP Flow and Volume Caculations
On-line facility volume (ac-ft): 0.1601
On-line facility target flow (cfs): 0.1592
Adjusted for 15 min (cfs): 0.1592
Off-line facility target flow (cfs): 0.0734
Adjusted for 15 min (cfs): 0.0734
Model Default Modifications
Water Quality
General Model Information
Landuse Basin Data
Routing Elements
POC Data


Analysis
Run Analysis
Water Quality
On-Line BMP
24 hour Volume (ac-ft) 0.0787
Standard Flow Rate (cfs) 0.1062
Off-Line BMP
Standard Flow Rate (cfs) 0.0582
Stream Protection Duration
LID Duration
Flow Frequency
Water Quality
Hydrograph
Wetland Input Volumes
LID Report
Recharge Duration
Recharge Predeveloped
Recharge Mitigated
Analyze datasets
Compact WDM
Delete Selected
Monthly FF
501 POC 1 Predeveloped flow
801 POC 1 Mitigated flow
All Datasets
Flow
Stage
Precip
Evap
POC 1
Flood Frequency Method
Log Pearson Type III 17B
Weibull
Cunnane
Gringorten

APPENDIX E: DESIGN FOR PERKFILTER

Design for PerkFilters

Kristar/Oldcastle Precast, Inc. FloGard Perk Filter™ (using ZPC Filter Media)

Ecology's Decision:

Based on Kristar/Oldcastle's application submissions, including the Draft Technical Evaluation Report, dated April 2010, Ecology hereby issues the following use level designations:

1. **General use level designation (GULD) for the Perk Filter™ for basic treatment:**
   - **Using a zeolite-perlite-carbon (ZPC) filter media as specified by Kristar/Oldcastle.**
   - **Sized at hydraulic loading rate of no more than 1.5 gpm/ft² of media surface area, per Table 1.**

Table 1. Design Flowrate per Cartridge

Effective Cartridge Height (inches)	12	18
Cartridge Flowrate (gpm/cartridge)	6.8	10.2

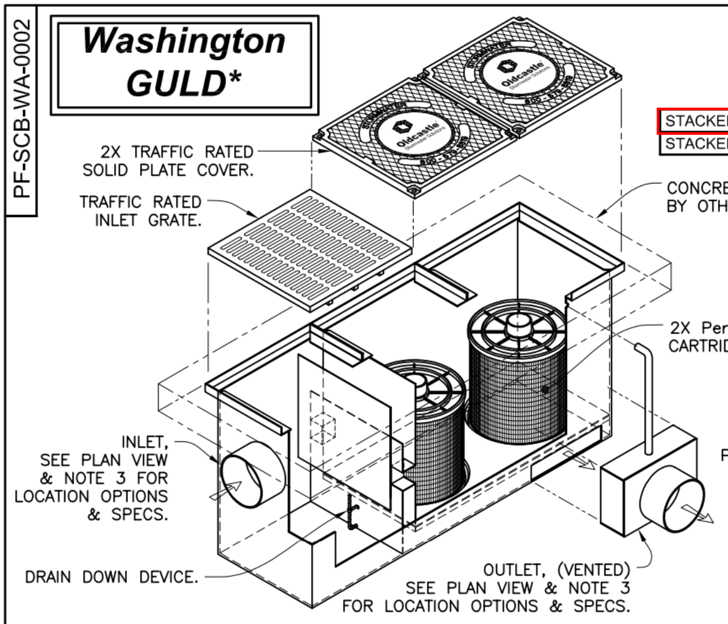
2. **General use level designation (GULD) for the Perk Filter™ for phosphorus treatment:**
   - **Using a zeolite-perlite-carbon (ZPC) filter media as specified by Kristar/Oldcastle.**
   - **Sized at hydraulic loading rate of no more than 1.5 gpm/ft² of media surface area, per Table 1.**

3. **Ecology approves Perk Filter™ units for treatment at the hydraulic loading rates shown in Table 1, and sized based on the water quality design flow rate for an off-line system. The internal weir in the inlet chamber functions as a bypass to route flow in excess of the water quality design flow rate around the treatment chamber. Calculate the water quality design flow rate using the following procedures:**

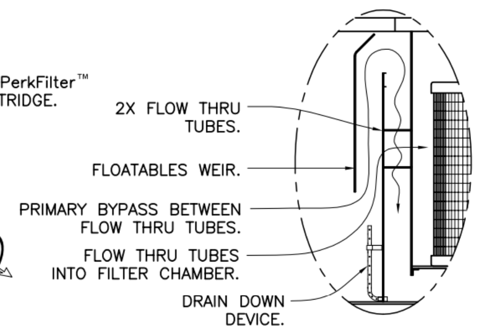
PF-SCB-WA-0002

*Washington GULD**

	CARTRIDGE SIZE	TREATMENT FLOW RATE GPM / CFS	TOTAL FLOW CAPACITY CFS	MINIMUM DEPTH (FEET)
	12.00"	13.6 / 0.030	1.3	1.80
	18.00"	20.4 / 0.045	1.3	2.34
STACKED	12.00" + 12.00"	27.2 / 0.061	1.3	3.08
STACKED	18.00" + 12.00"	34 / 0.076	1.3	3.67



ISOMETRIC VIEW

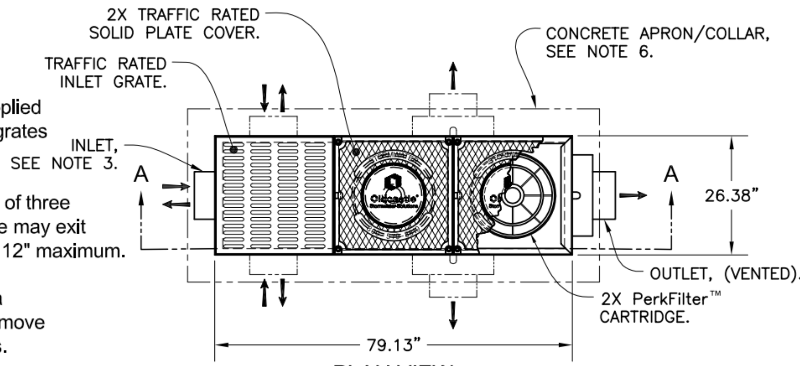


DETAIL A

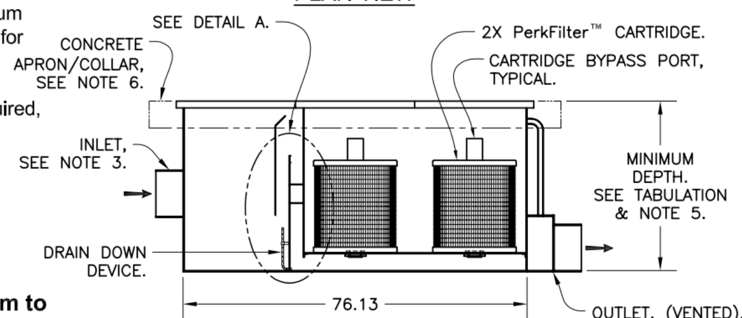
INLET / BYPASS ASSEMBLY & DRAIN DOWN SCALE: 1.5X

Notes:

1. All steel utilized in fabrication and shall be 1/4" plate per ASTM A36.
2. PerkFilter™ Catch basin shall be supplied with traffic rated (H20) bicycle-proof grates and solid plate cover.
3. Inlet pipe(s) may enter device on any of three sides of the inlet chamber. Outlet pipe may exit on any of all four sides. All pipe is Ø 12" maximum.
4. Inlet chamber shall be supplied with a DRAIN DOWN device designed to remove standing water between storm events.
5. For depths less than the specified minimum contact Oldcastle® Stormwater Solutions for engineering assistance.
6. Field poured Concrete Apron / Collar required, by others. Refer to PF-SCB-1000 for recommended configuration.
7. PerkFilter™ cartridge shall be maintained in accordance with manufacturer recommendations.



PLAN VIEW



*** Treatment Flow Rates shown conform to Washington State GULD Specifications.**

SECTION A-A

Media Filtration

PerkFilter™

Steel Catch Basin

Double Cartridge

(End Grate Configuration)

Oldcastle® Stormwater Solutions

7921 Southpark Plaza, Suite 200 | Littleton, CO | 80120 | Ph: 800.579.8819 | oldcastlestormwater.com

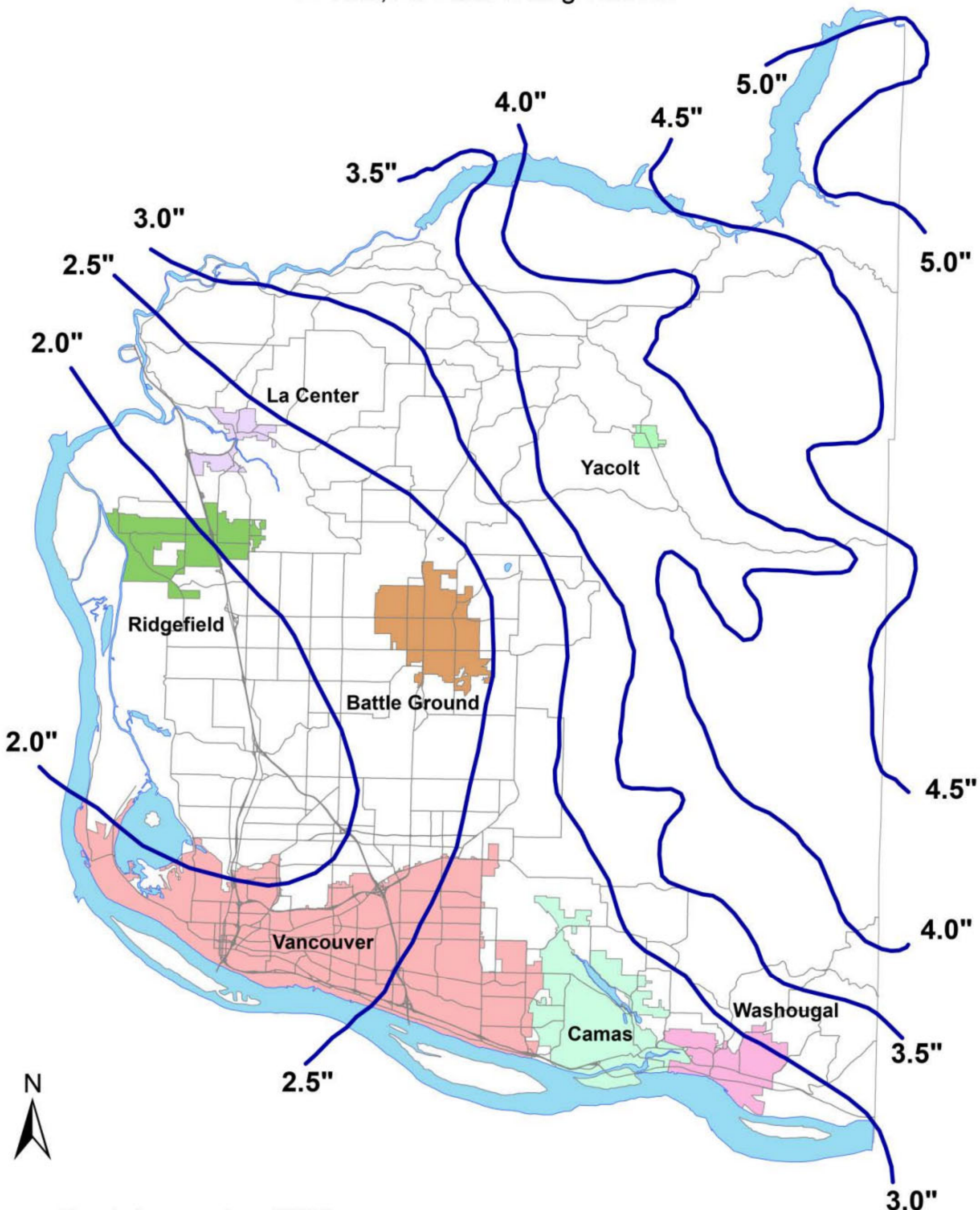
DRAWING NO.	REV	ECO	DATE	
PF-SCB-WA-0002	NR	ECO-0143 NEW	JPR 12/2/16	SHEET 1 OF 1

Since the stacked 12.00” cartridges have a treatment flow rate of 0.061 cfs, which is the first flowrate on the table greater than 0.0582 cfs, the stacked 12.00” + 12.00” cartridges will be used for each filter catch basin.

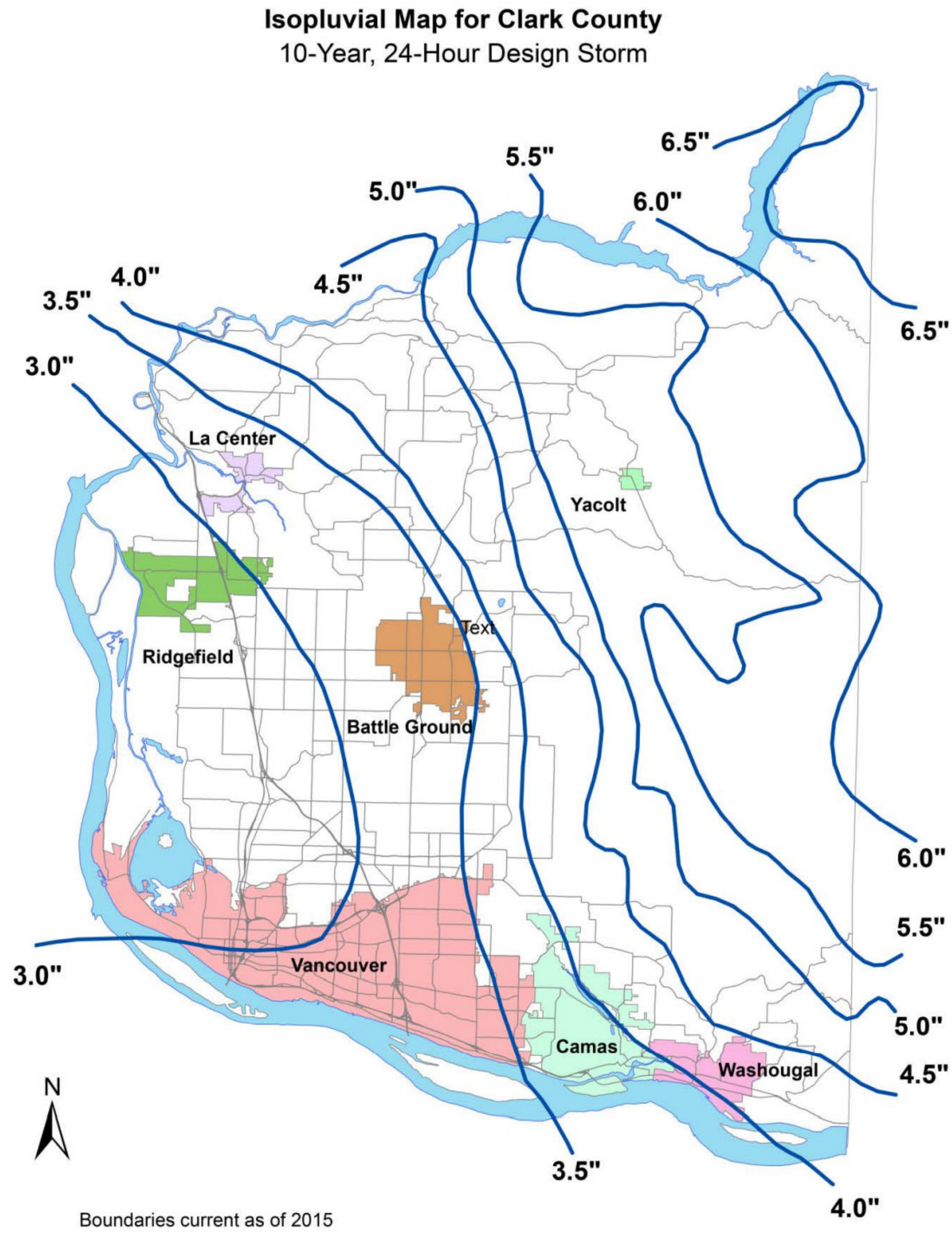
APPENDIX F: WWHM

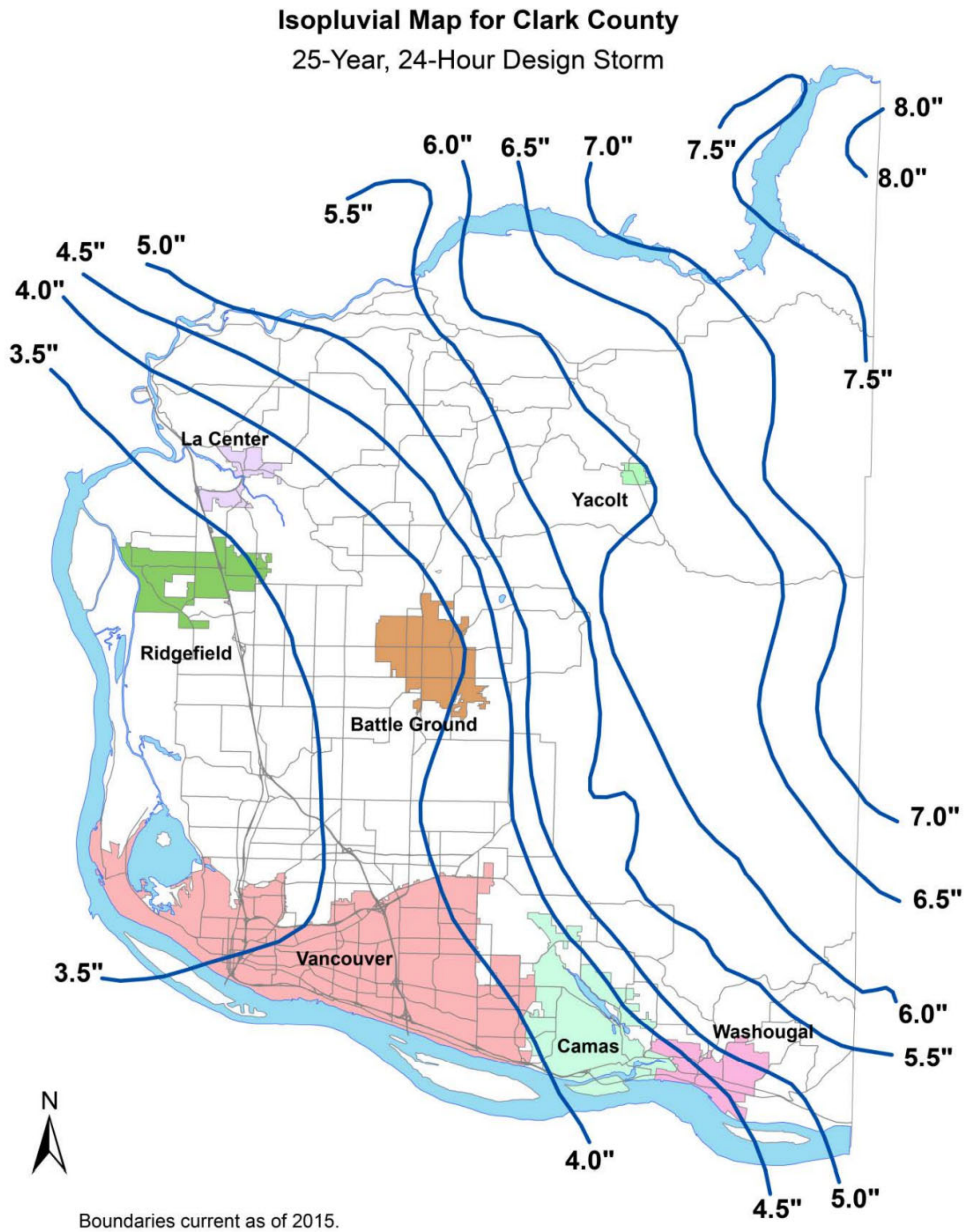
APPENDIX H: ISO MAPS

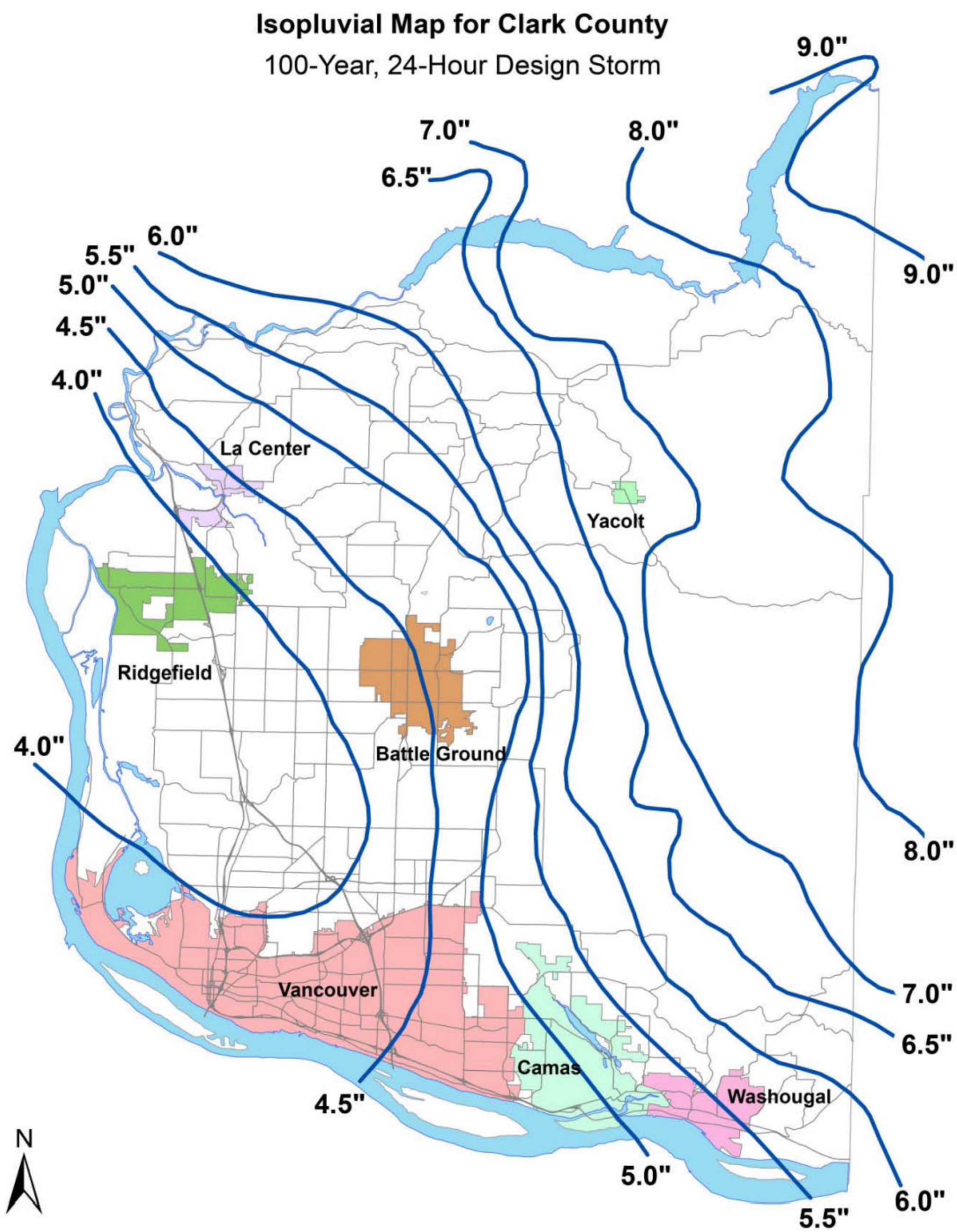
Isopluvial Map for Clark County
2-Year, 24-Hour Design Storm



Boundaries current as of 2015


Isopluvial Map for Clark County
10-Year, 24-Hour Design Storm
6.5"
5.5"
5.0"
6.0"
4.0"
4.5"
3.5"
6.5"
3.0"
La Center
Yacolt
Text
Ridgefield
Battle Ground
6.0"
5.5"
3.0"
Vancouver
5.0"
Camas
4.5"
Washougal
N
3.5"
4.0"
Boundaries current as of 2015


Isopluvial Map for Clark County
25-Year, 24-Hour Design Storm
3.5"
4.0"
4.5"
5.0"
5.5"
6.0"
6.5"
7.0"
7.5"
8.0"
8.0"
7.5"
7.0"
6.5"
6.0"
5.5"
5.0"
4.5"
4.0"
3.5"
La Center
Yacolt
Ridgefield
Battle Ground
Vancouver
Camas
Washougal
N
Boundaries current as of 2015.

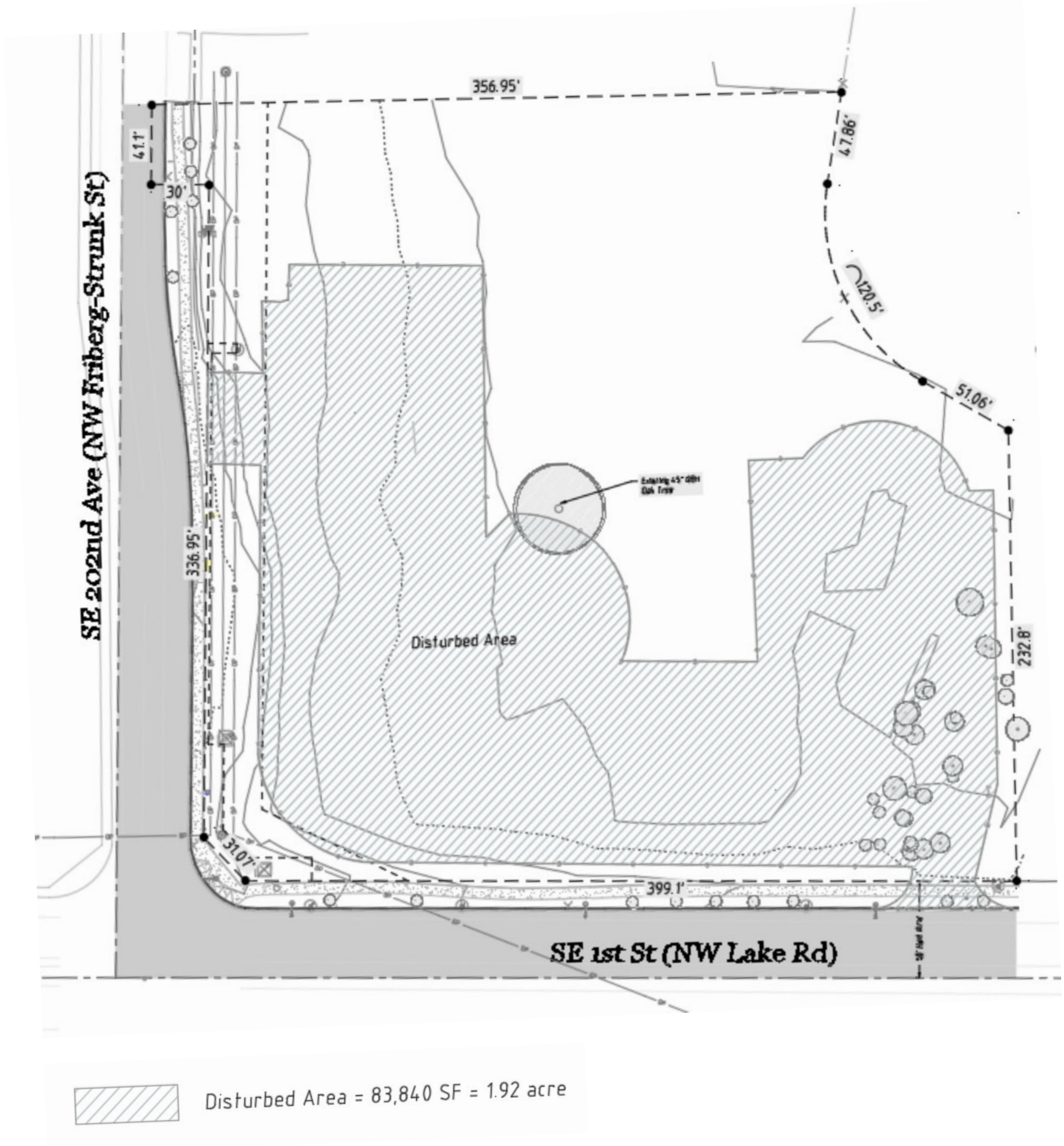


Boundaries current as of 2015.

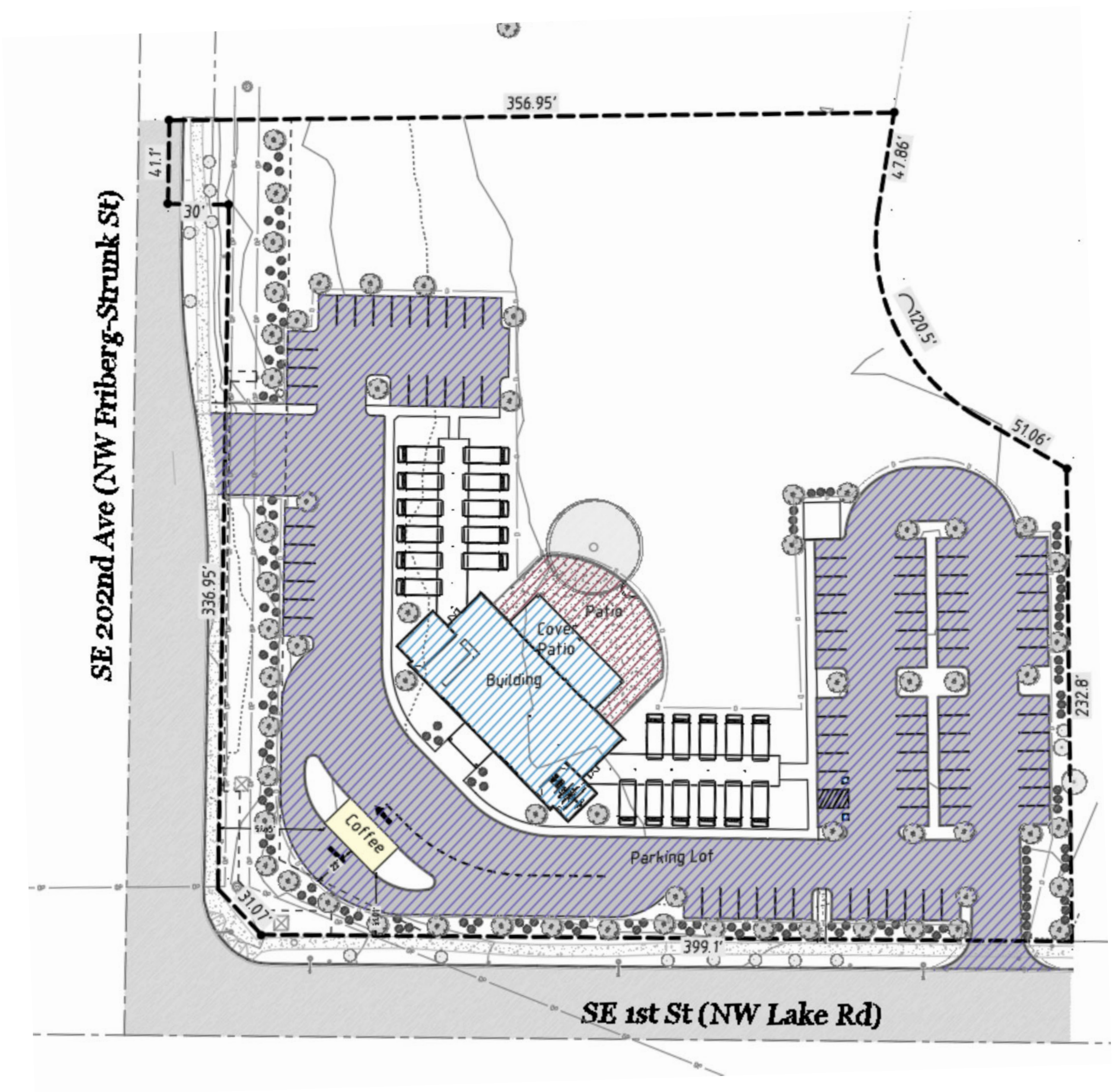
APPENDIX I: BASIN MAPS

Flow Control

Pre-developed



Mitigated



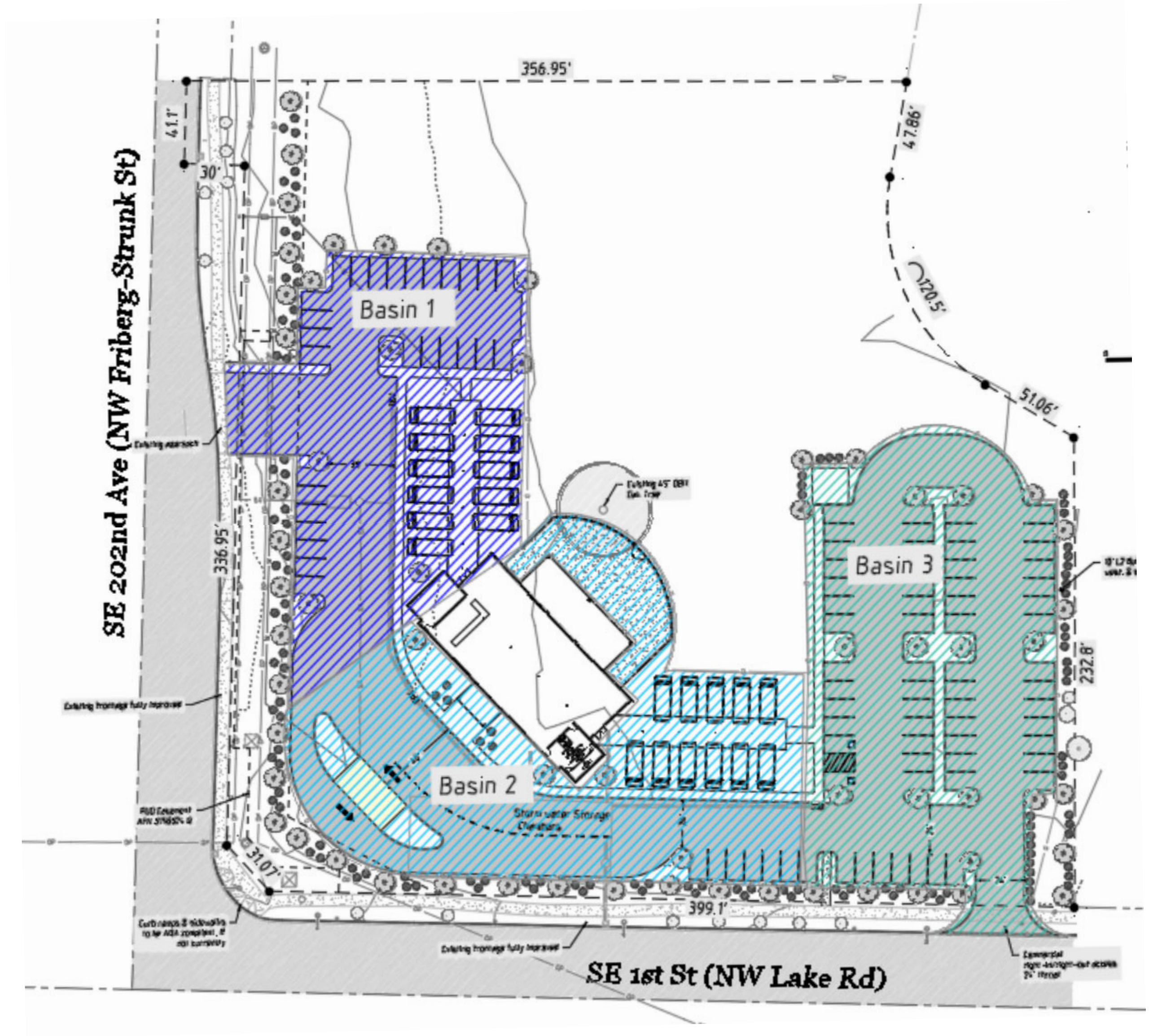
Building + Cover Patio (Roof 1) = 6,690 SF = 0.154 acre

Coffee Kiosk (Roof 2) = 600 SF = 0.014 acre

Patio = 2,875 SF = 0.066 acre

Parking Lot = 47,870 SF = 1.1 acre

Water Quality



WWHM2012

PROJECT REPORT

General Model Information

Project Name:	Oak Tree Station Project Flow Control
Site Name:	176162000
Site Address:	
City:	Camas
Report Date:	1/31/2022
Gage:	Lacamas
Data Start:	1948/10/01
Data End:	2008/09/30
Timestep:	15 Minute
Precip Scale:	1.300
Version Date:	2021/08/18
Version:	4.2.18

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
SG4, Forest, Flat	1.92
Pervious Total	1.92
Impervious Land Use	acre
Impervious Total	0
Basin Total	1.92

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Basin 1

Bypass:	Yes
GroundWater:	No
Pervious Land Use	acre
SG4, Field, Flat	0.586
Pervious Total	0.586
Impervious Land Use	acre
Impervious Total	0
Basin Total	0.586

Element Flows To:

Surface	Interflow	Groundwater

Roof 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
ROOF TOPS FLAT	0.154
Impervious Total	0.154
Basin Total	0.154

Element Flows To:

Surface	Interflow	Groundwater
Vault 1	Vault 1	

Roof 2

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
ROOF TOPS FLAT	0.014
Impervious Total	0.014
Basin Total	0.014

Element Flows To:

Surface	Interflow	Groundwater
Vault 1	Vault 1	

Patio

Bypass:	No	
GroundWater:	No	
Pervious Land Use	acre	
Pervious Total	0	
Impervious Land Use	acre	
DRIVEWAYS FLAT	0.014	
Impervious Total	0.014	
Basin Total	0.014	

Element Flows To:

Surface	Interflow	Groundwater
Vault 1	Vault 1	

Parking Lot

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
PARKING FLAT	1.1
Impervious Total	1.1
Basin Total	1.1

Element Flows To:

Surface	Interflow	Groundwater
Vault 1	Vault 1	

Routing Elements
Predeveloped Routing

Mitigated Routing

Vault 1

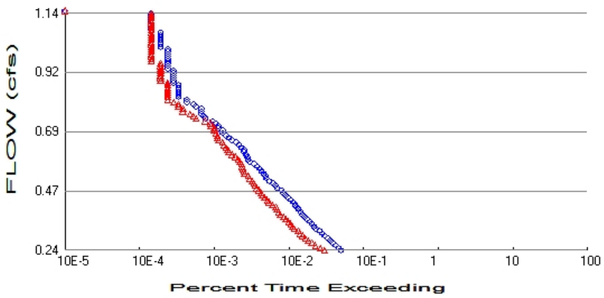
Width:	40 ft.	
Length:	100 ft.	
Depth:	3.75 ft.	
Discharge Structure		
Riser Height:	3.6 ft.	
Riser Diameter:	10 in.	
Orifice 1 Diameter:	0.75 in.	Elevation:0 ft.
Orifice 2 Diameter:	3.5 in.	Elevation:3.2 ft.
Element Flows To:		
Outlet 1	Outlet 2	

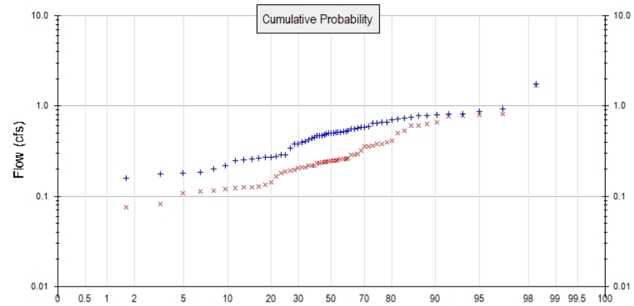
Vault Hydraulic Table

Stage(feet)	Area(ac.)	Volume(ac-ft.)	Discharge(cfs)	Infilt(cfs)
0.0000	0.091	0.000	0.000	0.000
0.0417	0.091	0.003	0.003	0.000
0.0833	0.091	0.007	0.004	0.000
0.1250	0.091	0.011	0.005	0.000
0.1667	0.091	0.015	0.006	0.000
0.2083	0.091	0.019	0.007	0.000
0.2500	0.091	0.023	0.007	0.000
0.2917	0.091	0.026	0.008	0.000
0.3333	0.091	0.030	0.008	0.000
0.3750	0.091	0.034	0.009	0.000
0.4167	0.091	0.038	0.009	0.000
0.4583	0.091	0.042	0.010	0.000
0.5000	0.091	0.045	0.010	0.000
0.5417	0.091	0.049	0.011	0.000
0.5833	0.091	0.053	0.011	0.000
0.6250	0.091	0.057	0.012	0.000
0.6667	0.091	0.061	0.012	0.000
0.7083	0.091	0.065	0.012	0.000
0.7500	0.091	0.068	0.013	0.000
0.7917	0.091	0.072	0.013	0.000
0.8333	0.091	0.076	0.013	0.000
0.8750	0.091	0.080	0.014	0.000
0.9167	0.091	0.084	0.014	0.000
0.9583	0.091	0.088	0.014	0.000
1.0000	0.091	0.091	0.015	0.000
1.0417	0.091	0.095	0.015	0.000
1.0833	0.091	0.099	0.015	0.000
1.1250	0.091	0.103	0.016	0.000
1.1667	0.091	0.107	0.016	0.000
1.2083	0.091	0.111	0.016	0.000
1.2500	0.091	0.114	0.017	0.000
1.2917	0.091	0.118	0.017	0.000
1.3333	0.091	0.122	0.017	0.000
1.3750	0.091	0.126	0.017	0.000
1.4167	0.091	0.130	0.018	0.000
1.4583	0.091	0.133	0.018	0.000
1.5000	0.091	0.137	0.018	0.000
1.5417	0.091	0.141	0.019	0.000
1.5833	0.091	0.145	0.019	0.000

1.6250	0.091	0.149	0.019	0.000
1.6667	0.091	0.153	0.019	0.000
1.7083	0.091	0.156	0.020	0.000
1.7500	0.091	0.160	0.020	0.000
1.7917	0.091	0.164	0.020	0.000
1.8333	0.091	0.168	0.020	0.000
1.8750	0.091	0.172	0.020	0.000
1.9167	0.091	0.176	0.021	0.000
1.9583	0.091	0.179	0.021	0.000
2.0000	0.091	0.183	0.021	0.000
2.0417	0.091	0.187	0.021	0.000
2.0833	0.091	0.191	0.022	0.000
2.1250	0.091	0.195	0.022	0.000
2.1667	0.091	0.199	0.022	0.000
2.2083	0.091	0.202	0.022	0.000
2.2500	0.091	0.206	0.022	0.000
2.2917	0.091	0.210	0.023	0.000
2.3333	0.091	0.214	0.023	0.000
2.3750	0.091	0.218	0.023	0.000
2.4167	0.091	0.221	0.023	0.000
2.4583	0.091	0.225	0.023	0.000
2.5000	0.091	0.229	0.024	0.000
2.5417	0.091	0.233	0.024	0.000
2.5833	0.091	0.237	0.024	0.000
2.6250	0.091	0.241	0.024	0.000
2.6667	0.091	0.244	0.024	0.000
2.7083	0.091	0.248	0.025	0.000
2.7500	0.091	0.252	0.025	0.000
2.7917	0.091	0.256	0.025	0.000
2.8333	0.091	0.260	0.025	0.000
2.8750	0.091	0.264	0.025	0.000
2.9167	0.091	0.267	0.026	0.000
2.9583	0.091	0.271	0.026	0.000
3.0000	0.091	0.275	0.026	0.000
3.0417	0.091	0.279	0.026	0.000
3.0833	0.091	0.283	0.026	0.000
3.1250	0.091	0.287	0.027	0.000
3.1667	0.091	0.290	0.027	0.000
3.2083	0.091	0.294	0.057	0.000
3.2500	0.091	0.298	0.101	0.000
3.2917	0.091	0.302	0.128	0.000
3.3333	0.091	0.306	0.149	0.000
3.3750	0.091	0.309	0.167	0.000
3.4167	0.091	0.313	0.183	0.000
3.4583	0.091	0.317	0.197	0.000
3.5000	0.091	0.321	0.210	0.000
3.5417	0.091	0.325	0.223	0.000
3.5833	0.091	0.329	0.234	0.000
3.6250	0.091	0.332	0.280	0.000
3.6667	0.091	0.336	0.408	0.000
3.7083	0.091	0.340	0.577	0.000
3.7500	0.091	0.344	0.773	0.000
3.7917	0.091	0.348	0.979	0.000
3.8333	0.000	0.000	1.179	0.000

Analysis Results
POC 1





+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 1.92
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0.586
Total Impervious Area: 1.282

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	0.481706
5 year	0.754113
10 year	0.902752
25 year	1.054393
50 year	1.144674
100 year	1.218985

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	0.252788
5 year	0.446568
10 year	0.605093
25 year	0.840633
50 year	1.042309
100 year	1.267008

Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	0.376	0.203
1950	0.493	0.243
1951	0.651	0.238
1952	0.382	0.248
1953	0.515	0.378
1954	0.718	0.396
1955	0.400	0.164
1956	0.784	0.799
1957	0.596	0.250
1958	0.442	0.232

1959	0.257	0.113
1960	0.254	0.126
1961	0.706	0.626
1962	0.474	0.209
1963	0.526	0.220
1964	0.502	0.358
1965	0.450	0.416
1966	0.576	0.376
1967	0.502	0.193
1968	0.656	0.248
1969	0.534	0.261
1970	1.747	1.670
1971	0.276	0.124
1972	0.470	0.180
1973	0.468	0.353
1974	0.737	0.812
1975	0.401	0.186
1976	0.577	0.217
1977	0.016	0.041
1978	0.823	0.504
1979	0.557	0.236
1980	0.340	0.289
1981	0.782	0.609
1982	0.526	0.360
1983	0.875	0.320
1984	0.286	0.116
1985	0.220	0.135
1986	0.272	0.125
1987	0.475	0.260
1988	0.183	0.219
1989	0.199	0.119
1990	0.181	0.143
1991	0.512	0.198
1992	0.564	0.207
1993	0.662	0.257
1994	0.506	0.230
1995	0.420	0.655
1996	0.804	0.764
1997	0.931	0.780
1998	0.752	0.288
1999	0.562	0.529
2000	0.272	0.107
2001	0.157	0.075
2002	0.821	0.297
2003	0.646	0.243
2004	0.177	0.083
2005	0.265	0.128
2006	0.495	0.267
2007	0.245	0.611
2008	0.288	0.254

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	1.7467	1.6697
2	0.9308	0.8122
3	0.8754	0.7991
4	0.8225	0.7798

5	0.8213	0.7642
6	0.8037	0.6549
7	0.7839	0.6260
8	0.7819	0.6112
9	0.7525	0.6093
10	0.7367	0.5286
11	0.7180	0.5045
12	0.7060	0.4159
13	0.6625	0.3955
14	0.6558	0.3778
15	0.6515	0.3762
16	0.6456	0.3604
17	0.5962	0.3577
18	0.5770	0.3531
19	0.5756	0.3198
20	0.5636	0.2971
21	0.5623	0.2888
22	0.5567	0.2878
23	0.5344	0.2666
24	0.5264	0.2606
25	0.5262	0.2604
26	0.5152	0.2573
27	0.5123	0.2540
28	0.5065	0.2500
29	0.5024	0.2483
30	0.5016	0.2477
31	0.4955	0.2433
32	0.4932	0.2428
33	0.4746	0.2382
34	0.4737	0.2363
35	0.4697	0.2321
36	0.4678	0.2305
37	0.4504	0.2195
38	0.4423	0.2189
39	0.4202	0.2174
40	0.4014	0.2086
41	0.3998	0.2068
42	0.3819	0.2031
43	0.3761	0.1977
44	0.3404	0.1933
45	0.2879	0.1859
46	0.2856	0.1804
47	0.2762	0.1640
48	0.2723	0.1425
49	0.2720	0.1347
50	0.2649	0.1279
51	0.2571	0.1259
52	0.2540	0.1253
53	0.2453	0.1237
54	0.2202	0.1190
55	0.1991	0.1162
56	0.1833	0.1131
57	0.1806	0.1073
58	0.1772	0.0827
59	0.1569	0.0753
60	0.0157	0.0413

Duration Flows
The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.2409	1046	626	59	Pass
0.2500	968	556	57	Pass
0.2591	894	476	53	Pass
0.2682	812	425	52	Pass
0.2774	747	377	50	Pass
0.2865	696	350	50	Pass
0.2956	636	320	50	Pass
0.3048	586	293	50	Pass
0.3139	543	281	51	Pass
0.3230	484	247	51	Pass
0.3321	456	233	51	Pass
0.3413	425	223	52	Pass
0.3504	405	208	51	Pass
0.3595	367	189	51	Pass
0.3687	338	178	52	Pass
0.3778	317	160	50	Pass
0.3869	301	153	50	Pass
0.3961	280	146	52	Pass
0.4052	265	141	53	Pass
0.4143	253	125	49	Pass
0.4234	240	114	47	Pass
0.4326	223	108	48	Pass
0.4417	201	98	48	Pass
0.4508	187	92	49	Pass
0.4600	176	89	50	Pass
0.4691	167	84	50	Pass
0.4782	145	78	53	Pass
0.4873	139	74	53	Pass
0.4965	128	71	55	Pass
0.5056	114	67	58	Pass
0.5147	104	62	59	Pass
0.5239	101	60	59	Pass
0.5330	96	53	55	Pass
0.5421	90	52	57	Pass
0.5513	85	51	60	Pass
0.5604	80	50	62	Pass
0.5695	72	49	68	Pass
0.5786	62	46	74	Pass
0.5878	59	44	74	Pass
0.5969	58	40	68	Pass
0.6060	56	36	64	Pass
0.6152	53	33	62	Pass
0.6243	52	32	61	Pass
0.6334	48	30	62	Pass
0.6426	45	28	62	Pass
0.6517	41	27	65	Pass
0.6608	36	24	66	Pass
0.6699	33	23	69	Pass
0.6791	30	23	76	Pass
0.6882	27	22	81	Pass
0.6973	27	21	77	Pass
0.7065	24	21	87	Pass
0.7156	24	19	79	Pass

0.7247	22	19	86	Pass
0.7338	20	16	80	Pass
0.7430	16	12	75	Pass
0.7521	15	11	73	Pass
0.7612	14	10	71	Pass
0.7704	14	8	57	Pass
0.7795	14	8	57	Pass
0.7886	12	7	58	Pass
0.7978	11	7	63	Pass
0.8069	9	6	66	Pass
0.8160	9	5	55	Pass
0.8251	7	5	71	Pass
0.8343	7	5	71	Pass
0.8434	7	5	71	Pass
0.8525	7	5	71	Pass
0.8617	7	5	71	Pass
0.8708	7	5	71	Pass
0.8799	6	5	83	Pass
0.8890	6	4	66	Pass
0.8982	6	4	66	Pass
0.9073	6	4	66	Pass
0.9164	6	4	66	Pass
0.9256	6	4	66	Pass
0.9347	5	4	80	Pass
0.9438	5	4	80	Pass
0.9530	5	4	80	Pass
0.9621	5	3	60	Pass
0.9712	5	3	60	Pass
0.9803	5	3	60	Pass
0.9895	5	3	60	Pass
0.9986	5	3	60	Pass
1.0077	5	3	60	Pass
1.0169	4	3	75	Pass
1.0260	4	3	75	Pass
1.0351	4	3	75	Pass
1.0442	4	3	75	Pass
1.0534	4	3	75	Pass
1.0625	4	3	75	Pass
1.0716	4	3	75	Pass
1.0808	3	3	100	Pass
1.0899	3	3	100	Pass
1.0990	3	3	100	Pass
1.1082	3	3	100	Pass
1.1173	3	3	100	Pass
1.1264	3	3	100	Pass
1.1355	3	3	100	Pass
1.1447	3	3	100	Pass

Water Quality

Water Quality BMP Flow and Volume for POC #1

On-line facility volume:	0 acre-feet
On-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.
Off-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.

LID Report

LID Technique	Used for Treatment ?	Total Volume Needs Treatment (ac-ft)	Volume Through Facility (ac-ft)	Infiltration Volume (ac-ft)	Cumulative Volume Infiltration Credit	Percent Volume Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Vault 1 POC	☐	230.76			☐	0.00			
Total Volume Infiltrated		230.76	0.00	0.00		0.00	0.00	0%	No Treat. Credit
Compliance with LID Standard 8% of 2-yr to 50% of 2-yr									Duration Analysis Result = Passed

Model Default Modifications

Total of 0 changes have been made.

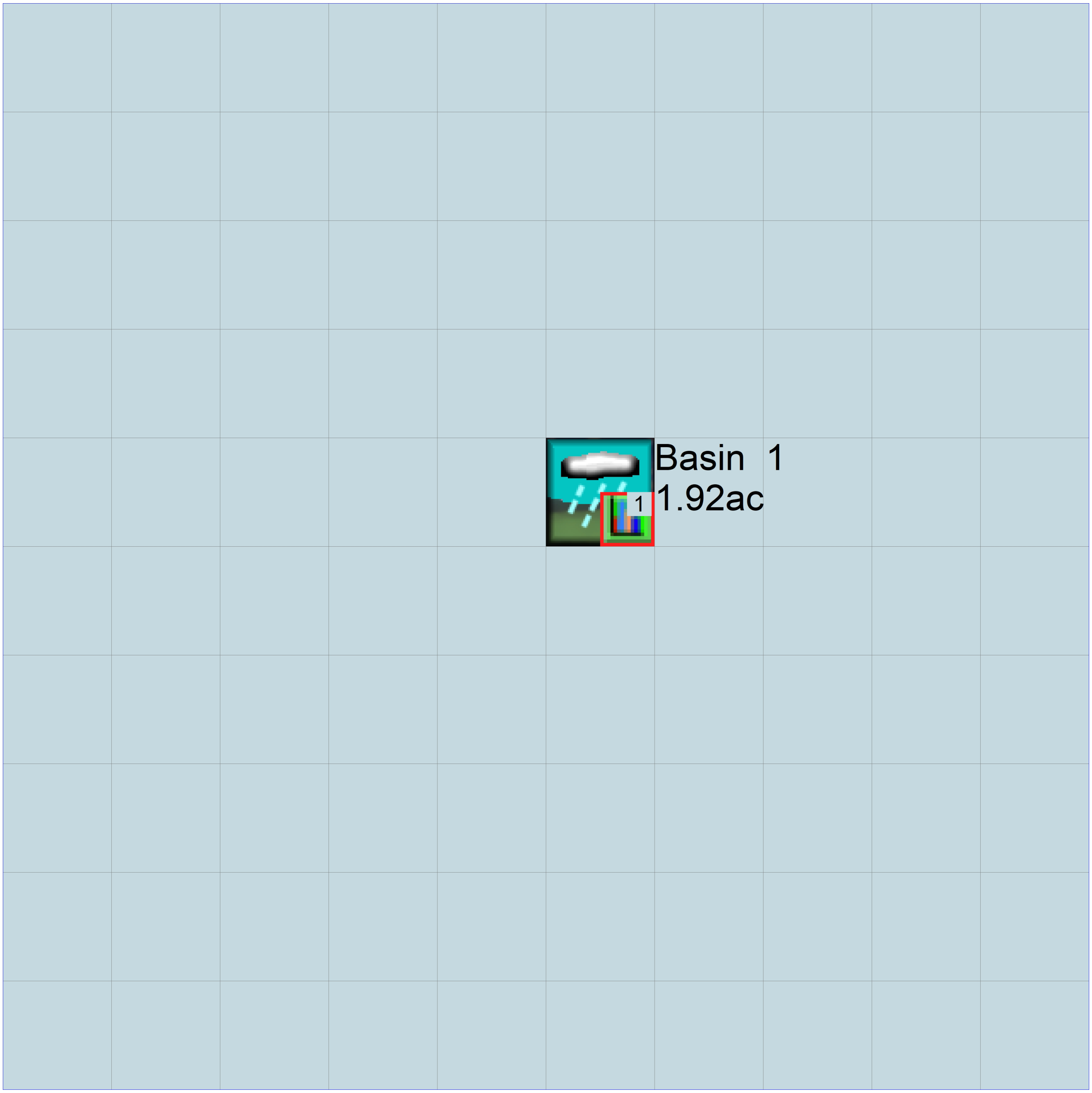
PERLND Changes

No PERLND changes have been made.

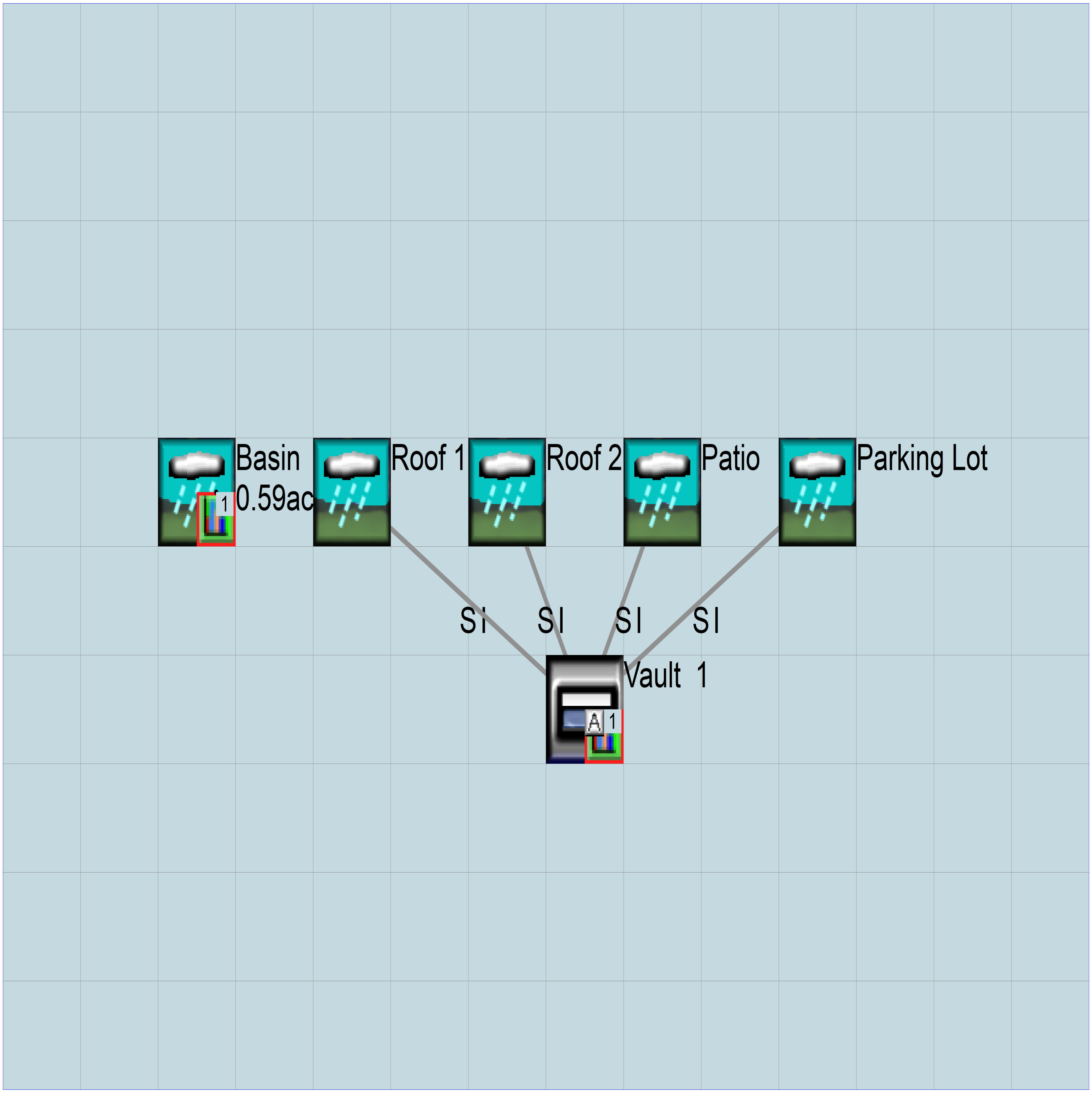
IMPLND Changes

No IMPLND changes have been made.

Appendix
Predeveloped Schematic



Mitigated Schematic



Predeveloped UCI File

Mitigated UCI File

Predeveloped HSPF Message File

Mitigated HSPF Message File

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GEOTECHNICAL ENGINEERING STUDY

For

Oak Tree Station

City of Camas, Washington

Prepared for:

Oak Tree Station

3239 NW Hood Ct

Camas WA 98607

Prepared by:

Engineering Northwest PLLC
6168 NE HWY 99
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PH: 360-931-3122
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January 13, 2022

Contents

1.0 Introduction

This report presents the results of the geotechnical engineering study completed by Engineering Northwest PLLC for the proposed Oak Tree Station project in the City of Camas, Washington. The general location of the site is shown on the Vicinity Map, figure 1. The site includes one parcel, which total approximately 3.95 acres. At the time our study was performed, the site and our exploratory locations were approximate as shown on the site plan map, figure 1.

The purpose of this study was to explore subsurface conditions at the site, and based on the conditions encountered, provide geotechnical recommendations for the proposed construction. This report is subject to the limitations expressed in Section 6.0, Conclusion and Limitations.

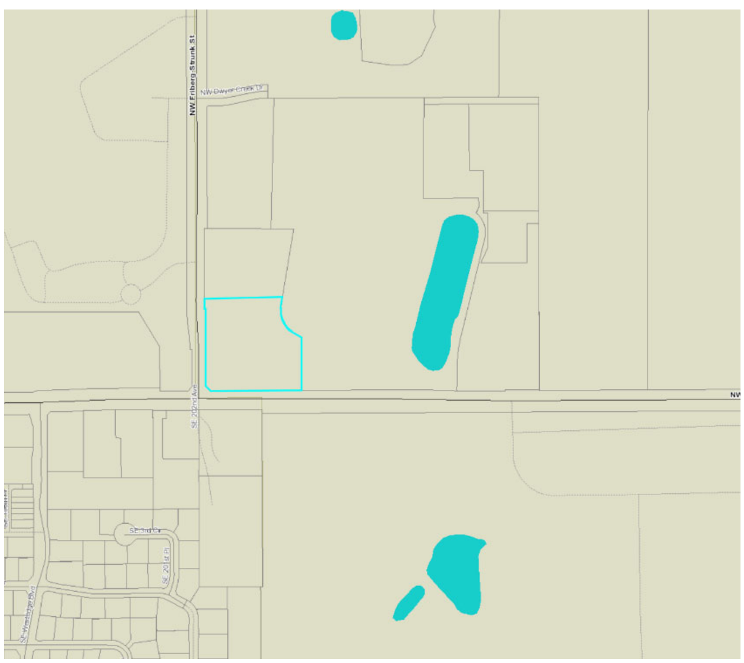


FIGURE 1

1.1 General Site Information

As indicated on Figures 1 the 3.95-acre subject site is located north side of NW Lake Road in the City of Camas, Washington. The approximate latitude and longitude are 45°39'19.72"N and 122°27'56.85"W and the legal description is a portion of the SE ¼ Section 29 Township 2N, Range 3E, Willamette Meridian. The regulatory jurisdictional agency is the City of Camas, Washington.

1.2 Proposed Development

The proposed Oaktree Station project is located on north side of NW lake Road. The project proposes 5,000 square-feet indoor and outdoor eating area and a small coffee shop. The project includes the future construction of utility, private parking, a storm water facility and other related infrastructural improvements.

2.0 Regional Geology and Soil Conditions

According to the Geologic Map of the Vancouver Quadrangle, Washington and Oregon (Washington Division of Geology and Earth Resources, Open-File Report 87-10, Revised November 1987), near-surface geology is:

Map Unit: Qf
Name: Pleistocene outburst-flood deposits
Full Name: Pleistocene outburst-flood deposits
Age: Quaternary (Pleistocene)
Description: Pleistocene gravel and sandy gravel deposits with interbedded silt lenses; deposited as benches along the main stem of the Snake River as a result of rapid draining of glacial Lake Bonneville; also widespread silt, sand, gravel, and boulder deposits deposited during multiple catastrophic drainings of glacial Lake Missoula; includes glaciolacustrine deposits.

The Soil Survey of Clark County, Washington (United States Department of Agriculture, Soil Conservation Service [USDA SCS], November 1972) identifies three separate surface soils with the subject property. Although actual on-site soils may vary from the broad USDA descriptions, soil types and associated descriptions are presented below.

- Dollar (DoB): The Olympic series consists of deep, moderately well drained, nearly level to gently sloping soils. Hydrologic soil group "C"

3.0 Regional Seismology

Recent research and subsurface mapping investigations within the Pacific Northwest appear to suggest the historic potential risk for a large earthquake event with strong localized ground movement may be underestimated. Past earthquakes in the Pacific Northwest appear to have caused landslides and ground subsidence, in addition to severe flooding near coastal areas. Earthquakes may also induce soil liquefaction, which occurs when elevated horizontal ground acceleration and velocity cause soil particles to interact as a fluid as opposed to a solid. Liquefaction of soil can result in lateral spreading and temporary loss of bearing capacity and shear strength.

There are at least four major known fault zones in the vicinity of the site that may be capable of generating potentially destructive horizontal accelerations. These fault zones are described briefly in the following text.

Portland Hills Fault Zone

The Portland Hills Fault Zone consists of server northwest-trending faults located along the eastern boundary of the Portland Hills. The fault zone is approximately 12 miles in length and is located approximately 8.5 southwest of the site. According to Seismic Design Mapping, State of Oregon (Geomatrix Consultants, 1995), there is no definitive consensus among geologists as to the zone fault type. Several alternate interpretations have been suggested, including various strike-slip and dipping thrust fault theories. Evidence exists to suggest that fault movement has impacted shallow Holocene deposits and deeper Pleistocene sediments. Seismologists recorded a M3.9 earthquake thought to be associated with the fault zone near Kelly Point Park in April 2003, and a M3.5 earthquake possibly associated with the fault zone occurred approximately 1.3 miles east of the fault in 1991. Therefore, the Portland Hills Fault Zone is generally thought to be potentially active and capable of producing possible damaging earthquakes.

Gales Creek-Newberg-Mt. Angel Fault Zone

Located approximately 30 miles south of the site, the 50-mile long Gales Creek-Newberg-Mt. Angel Structural Zone consists of a series of discontinuous northwest-trending faults. Possible late-Quaternary geomorphic surface deformation may exist along the structural zone (Geomatrix Consultants, 1995). Although no definitive evidence of impacts to Holocene sediments has reportedly been observed, a M5.6

earthquake occurred in March 1993 near Scotts Mills, approximately four mile south of the mapped extent of the Mt. Angel fault. It is unclear if the earthquake occurred along the fault zone or a parallel structure. Therefore, the Gales Creek-Newberg-Mt Angel Structural Zone is considered potentially active.

Lacamas Creek-Sandy River Fault Zone

The northwest-trending Lacamas Creek Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 3.5 southeast of the site. According to Geologty and Groundwater Conditions of Clark County Washington (USGS Water Supply Paper 1600, Mundorff, 1964) and the Geologic Map of the Lake Owego Quandrangle (Oregon DOGAMI Series GMS-59, 1989), the Lacamas Creek fault zone consists of shear contact between the Troutdale Formation and underlying Oligocene andesite-basalt bedrock. Secondary shear contact associated with the fault zone may have produced a series of prominent northwest-southeast geomorphic lineaments in proximity to the site. Recorded mild seismic activity during the recent past indicates this area may be potentially seismogenic.

Cascadia Subduction Zone

The Cascadia Subduction Zone has recently been recognized as a potential source of strong earthquake activity in the Portland/Vancouver Basin. This phenomenon is the result of the earth's large tectonic plate movement. Geologic evidence indicates that volcanic ocean floor activity along the Juan de Fuca ridge in the Pacific Ocean causes the Juan de Fuca Plate to perpetually move east and subduct under the North American Continental Plate. The subduction zone results in historic volcanic and potential earthquake activity in proximity to the plate interface, believed to lie approximately 20 to 60 miles west of the general location of the Oregon and Washington coast (Geomatrix Consultants, 1995).

4.0 Geotechnical Field Investigation

A geotechnical field investigation consisting of visual reconnaissance and one test pit explorations (TP-1) was conducted at the site. Test pit exploration was performed with a excavator. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications. Disturbed soil samples were collected from relevant soil horizons and analyzed. Test pit locations are indicated on Figure 2.

4.1 Subsurface Exploration and Investigation

We explored subsurface conditions for one test pits (TP-1) were excavated at the site to a maximum depth of 6 feet on January 13, 2022. The approximate locations of the test pit are shown in Figure 2.

Select soil samples from the test pit were tested to determine the natural moisture content, dry density, organic content.

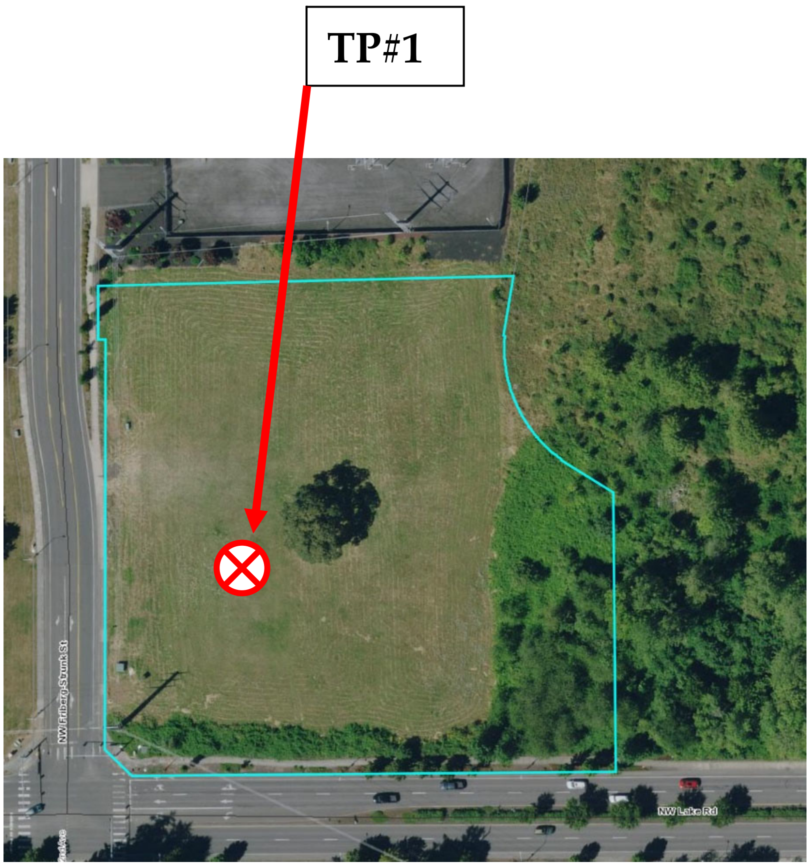


Figure 2

4.2 Surface Investigation and Site Description

The surface topography of the subject property typical slopes from east to west. The majority of the site is covered with field grass one large tree and serval small trees to the east property line.

In general, the surface soil over the majority of the project site varies from dark brown to reddish-brown which is typical Dollar soil type. The near- surface soil conditions has relatively high organic content extended to an approximate depth of 14 inches.

Laboratory testing of selected samples resulted in soil moisture content varying between 8 and 12 percent.

4.2.1 Soil Type Description

The field sample soil results are listed below.

Soil	USDA Texture	Unified	AASHO	Hydrologic Group
DoB	Loam	ML	A-4	C

5.0 Design Recommendations

The geotechnical site investigation suggests that proposes development is generally compatible with surface and subsurface soils, provided the recommendations presented in this report are used and incorporated into the design and construction process.

5.1 Site Preparation and Grading

Trees, grasses, and other grubbing items should be removed from all building, slab, structural fill, and pavement and sidewalk areas. Root balls should be grubbed out to the depth of the roots, which could exceed 2.0 to 3.0 feet depth. Depending on the methods used to remove the root balls, considerable disturbance and lessening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

The existing topsoil/root zone should be stripped and removed from all proposed building, slab, structural fill, pavement and sidewalk areas, and for a 5-foot margins around such areas. Based on our explorations, the average depth of stripping in will be approximately 10 inches. Greater stripping depths will be required to remove localized zones of loose or organic soil.

After grubbing, stripping, and required site cutting have been completed, we recommend proof rolling the subgrade with a fully-loaded dump truck or similar size, rubber tire construction equipment to identify areas of excessive yielding. The proof rolling should be observed by geotechnical engineering who will evaluate the subgrade. If areas of excessive yielding are identified, the material should be excavated and replaced with compacted materials recommended for structural fill. Areas that appear to be too wet and soft to support proof rolling equipment should be prepared in accordance with the recommendations for wet weather construction presented in the following section of this report

The test pit excavations were backfilled using the relatively minimal compaction effort of the hoe bucket; therefore, soft spots can be expected at these locations. We recommend that these relatively uncompacted soils be removed from the test pits to a depth of 3.0 feet below finished subgrade. The resulting excavation should be brought back to grade with structural fill.

5.2 Engineered Structural Fill

The native silt can be used as structural fill provided it is adequately moisture conditioned. Silty soils are generally sensitive to small changes in moisture content and are difficult, if not impossible, to compact adequately during wet weather or when their moisture content is more than a few percentage points above the optimum moisture content. Some moisture conditioning will likely be necessary even during the dry weather construction season. We recommend using clean. Angular imported granular material for structural fill if site soils cannot be properly moisture conditioned. As an alternative, use of the native silt soil as structural fill may be acceptable if it is properly amended with Portland cement or lime.

When used as structural fill, the silt soils should be placed in lifts with a maximum uncompact thickness of 6 to 8 inches and compacted to not less than 92 percent of the maximum dry density, as determined by American Society for Testing and Materials (ASTM) D 1557.

Engineered structural fill should be placed in loose lifts not exceeding 12 inches in depth and compacted using standard conventional compaction equipment. A field density at least equal to 92 percent of the maximum dry density, obtained from the modified Proctor moisture-density relationship test (ASTM D1557), is recommended for structural fill placement. For engineered structural fill placed on sloped grades, the area should be benched to provide a horizontal surface for compaction. Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D6938-08. Field compaction testing should be performed for each vertical foot of engineered fill placed. Engineered fill placement should be observed by an experienced geotechnical engineer or designated representative. Engineered structural fill placement occurs during dry weather conditions, clean non-organic achieve recommended compaction specifications. If adequate compaction is not achievable with clean native soils, import structural fill consisting of well-graded granular material with a maximum particle size of 3 inches and no more than 5 percent passing the No. 200 sieve is recommended. Representative samples of proposed engineered structural fill should be submitted for laboratory analysis and approval by the geotechnical engineer prior to placement. Laboratory analyses should include particle-size gradation and modified Proctor moisture-density analysis.

Trench backfill for the utility pipe base and pipe zone should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 8 percent by weight passing the U.S. Standard No. 200 Sieve. The material should be free of roots, organic matter, and other unsuitable materials. Backfill for the pipe base and pipe zone should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as recommended by the pipe manufacturer. Within building and pavement areas, trench backfill placed above the pipe zone should be compacted to at least 92 percent of ASTM D 1557 at depths greater than 2.0 feet below the finished subgrade and as recommended for structural fill within 2 feet of finished subgrade. In all other areas, trench backfill above the pipe zone should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D 1557.

As an alternative to the use of imported granular material, an experienced contractor may be able to amend the on-site with Portland cement to obtain suitable support properties. Depending on the size of the area, it is generally less costly to amend on-site soils than to remove and replace soft soils with granular material. Based on the moisture contents, soil types, and processing speed, cement amendment would be more suitable at this site than lime amendment.

The amount of cement used to amend the soils generally varies with moisture content. It is difficult to predict field performance of soils to cement amendment due to variability in soil response and we recommend laboratory testing to confirm expectations. However, for preliminary design purposes, we expect acceptable soil strength will be obtained using an amendment rate of 6 pounds Portland cement tilled to a depth of 12 inches. This translates to approximately 6 percent cement by weight. The amount of cement added to the soil may need to be adjusted based on field observations and performance.

5.2.1 Reuse of Undocumented Fill Material

As discussed in Section 5.1 Site Preparation and Grading, undocumented fill was encountered at the entrance of the site. In general, the fill encountered consisted of sand and gravel. If minor debris is encountered the contractor shall notify the geotechnical engineer or designated representative. Cobbles and boulders larger than 6 inches that cannot be broken into smaller fragments should be removed. Crushing and mixing processes should be observed and approved by an experienced geotechnical engineer.

5.3 Cut and Fill Slopes

Fill placed on grades steeper than 5H:1V should be horizontally benched at least 10 feet into the slope. Fill slopes greater than six feet in height should be vertically keyed into existing subsurface soil. Drainage implementations, including subdrains or perforated drain pipe trenches, may also be required in proximity to cut and fill slopes if seeps, springs, or soft mottled soils are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by the geotechnical engineer during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion. Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required.

Final cut or fill slopes should not exceed 2H:1V or 20 feet in vertical height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. Fill slopes should be constructed by placing fill material in maximum 12-inch level lifts, compacting as described in Section 5.2, Engineered Structural Fill and horizontally benching where appropriate. Fill slopes should be overbuilt, compacted, and trimmed at least two feet horizontally to provide adequate compaction of the outer slope face. Proper cut and fill slope construction is critical to overall project stability and should be observed by and experienced geotechnical engineer.

5.4 Home Foundations

Engineering Northwest recommendation to follow the recommendation and guidelines in the most current IBC.

5.5 Temporary Excavations

The following information is provided solely as a service to our client. Under no circumstances should this information be interpreted to mean that Engineering Northwest PLLC is assuming responsibility for construction site safety or the contractor’s activities; such responsibility is not being implied and should not be inferred.

In no case should excavation slopes be greater than the limits specified in local, state and federal safety regulations. Based on the information obtained from our field exploration and laboratory testing, the site soils expected to be encountered in excavations, firm to stiff silt (ML) and silty sand (SM) would be classified as a Type "B" soil by OSHA guidelines.

Therefore, temporary excavations and cuts greater than four feet in height, should be sloped at an inclination no steeper than 1H:1V (horizontal:vertical) for type "B" soils. If slopes of this inclination, or flatter, cannot be constructed or if excavations greater than ten feet in depth are required, temporary shoring may be necessary.

The shoring would help protect against slope or excavation collapse, and would provide protection to workmen in the excavation. If temporary shoring is required, we will be available to provide shoring design criteria, if requested.

5.6 Lateral Earth Pressure

Lateral earth pressure should be carefully considered for design of retaining walls or below-grade structures. Hydrostatic pressure and additional surcharge loading should also be considered. Retained material may include engineered structural backfill or relatively undisturbed native soil. Structural wall backfill may consist of recompacted fine-textured soils or imported granular material. Backfill should be prepared and compacted to at least 92 percent of maximum dry density as determined by the modified Proctor test (ASTM D1557). If seismic design is required, seismic forces for unrestrained wall may be calculated by superimposing a uniform lateral force of $10H^2$ pounds per lineal foot of wall, where H is the total wall height in feet. The resultant force should be applied at 0.6H from the base of the wall.

A continuous one-foot-thick zone of free-draining, washed, open-graded 1-inch by 2-inch drain rock and a 3-inch perforated gravity drain pipe is assumed behind retaining walls. Geotextile filter fabric should be placed between the drain rock and fine-textured backfill soil. Specifications for drainpipe design are presented in Section 5.10, Drainage. If walls cannot be gravity drained, saturated base conditions and /or applicable hydrostatic pressures should be assumed.

5.7 Seismic Design Considerations

Based upon Site Class Map of Clark County, Washington (Washington State Department of Natural Resources, 2004), shallow site soils may be represented by Site Class C as defined in 2009 IBC Table 161.5.2. This assessment is preliminary, pertains to near-surface soils, and is based upon limited field exploration and research of existing published literature. Additional exploration would be necessary to provide soil site class information at greater depths. Amplification of seismic energy due to depth to competent bedrock, compression and shear wave velocity of bedrock, presence and thickness of loose, unconsolidated alluvial deposits, soil plasticity, grain size, and other factors is possible at the site

According to Clark County Maps Online, the site is mapped as very low to low potential for liquefaction.

Identification of specific seismic response spectra, probabilistic ground motions, and liquefaction analysis for the site are beyond the scope of this investigation. If site structures are designed in accordance with recommendations specified in the 2009 IBC, the potential for peak ground accelerations in excess of adjusted and amplified values should be understood.

5.9 Drainage

At a minimum, site drainage should include surface water collection and conveyance to properly designed Stormwater management structures and facilities. Drainage design in general should conform to the Clark County regulations. Finished site grading should be conducted with positive drainage away from structures. Depressions or shallow areas that may retain ponding water should be avoided. Roof drains and perimeter foundation drains are recommended for proposed structures. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from foundations into the storm water system or approved discharge location.

5.10 Bituminous Asphalt And Portland Cement Concrete

According to the preliminary short plat plan the subject site is not anticipated to include asphalt concrete for the new parking lot. Based upon analytical laboratory test results and field exploration. Engineering Northwest PLLC recommends the general pavement design consist of 12 inches of compacted crushed aggregate base overlain with a minimum of 3.0 inches of asphalt concrete pavement for truck loading and traffic areas.

For dry weather construction, pavement surface sections should bear upon competent subgrade consisting of compacted native soil or engineered structural fill. Wet weather pavement construction is discussed later in Section 5.44, Wet Weather Construction Methods and Techniques. Subgrade conditions should be evaluated and tested by a licensed geotechnical engineer or designated representative prior to placement of crushed aggregate base. Subgrade evaluation should include nuclear gauge density testing and wheel proof-roll observations conducted with a 12-cubic yard, double-axle dump truck or equivalent. Nuclear gauge density testing should be conducted at 250-foot intervals or as determined by the onsite geotechnical engineer. Subgrade soil should be compacted to at least 95 percent of the modified Proctor dry density as determined by ASTM D1557. Areas of observed deflection or rutting during proof-roll evaluation should be excavated to a firm surface and replaced with compacted crushed aggregate.

Crushed aggregate base should be compacted and tested in accordance with the specifications outlined above. Asphalt concrete pavement should be compacted to at least 91 percent of maximum Rice Density. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with Washington Department of Transportation and the Clark County specifications.

Curb and sidewalk aggregate base should be observed and proof-rolled in the presence of an experienced geotechnical engineer or designated representative. Soft areas that deflect or rut should be stabilized prior to pouring concrete. Concrete should be tested during installation in accordance with ASTM C171, C138, C231, C143, C1064, and C31. This includes casting of cylinder specimen at a frequency of four cylinders per 100 cubic yards of poured concrete. Recommended filed and analytical laboratory concrete testing includes slump. Air entrainment.
Temperature, and unit weight.

5.11 Wet Weather Construction Methods and Techniques

Wet weather construction often results in significant shear strength reduction and soft areas that my rut or deflect. Installation of granular working layers may be necessary to provide a firm support base and sustain construction equipment. Granular layers should consist of all-weather gravel, 4-inch by 6-inch gabion, or other similar material (6- inch maximum size with less than 5 percent passing the No. 200 sieve).

Construction equipment traffic across exposed native soil should be minimized. Equipment traffic induces dynamic loading, which may result in weak areas and significant reduction in shear strength for soils above plastic limit. Wet weather construction may generate significant excess quantities of soft wet soil, which should be removed from the site or stockpiled in a designated area.

5.12 Soil Erosion Potential

Based upon review of the Soil Survey of Clark County, Washington and field observations, the erosion hazard for the site soil is considered low to moderate. For flat to shallow- gradient portions of the property the erosion hazard is likely to be low. Erosion potential generally increases in sloped areas. Therefore, disturbance to vegetation in sloped areas should be minimized during construction activities. Soil is also prone to erosion if unprotected and unvegetated during periods of increased precipitation.

Erosion can be minimized by performing construction activities during dry summer months.

Site-specific erosion control measures should be implemented to address the maintenance of exposed areas. This may include silt fence, biofilter bags, straw wattles, or other suitable methods. During construction activities, exposed areas should be well-compacted and protected from erosion with visqueen, surface tactifier, or other means, as appropriate. Temporary slopes or exposed areas may be covered with straw, crushed aggregate, or riprap in localized areas to minimize erosion. Erosion and water runoff during wet weather conditions may be controlled by application of strategically placed channels and small detention depressions with overflow pipes. After grading the surface should be vegetated as soon as possible with erosion-resistant native grasses and forbs. Jute mesh or straw may be applied to enhance vegetation. Once established, vegetation should be properly maintained. It is also recommended that disturbance to existing native vegetation and surrounding organic soil be minimized during construction activities.

5.13 Soil Shrink/Swell Potential

The Soil Survey of Clark County, Washington indicates a very low potential for shrinking and swelling of the native site soils or imported subbase material.

5.14 Utility Installation

Utility installation at the site may require subsurface excavation and trenching. Excavation, trenching and shoring should conform to federal Occupational Safety and Health Administration (OSHA)(29 CFR, PART 1926) and WISHA (WAC, Chapter 296-155) regulations. Site soils may slough when cut vertically and sudden precipitation events or perched ground water may result in accumulation of water within excavation zones and trenches. These areas should be dewatered in accordance with appropriate discharge regulations.

Utilities should be installed in general accordance with manufacturer's recommendations. Utility trench backfill should consist of crushed aggregate or other coarse-textured, free-draining material acceptable to the Clark County and the site geotechnical engineer. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). The remaining backfill should be compacted to at least 90 percent of maximum dry density as determined by the modified Proctor moisture-density test (ASTM D1557). With exception of the pipe zone, backfill should be placed in loose lifts not exceeding 12 inches in thickness.

Compaction of utility trench backfill material should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D6938-08. Field compaction testing should be performed at 250-foot intervals along the utility trench centerline at the surface and midpoint depth of the trench. Compaction frequency and specifications may be modified for non-structural areas in accordance with recommendations of the site geotechnical engineer.

5.15 Groundwater

No groundwater was an encounter in any of the test pits to the maximum exploration depth of 6 feet below the existing ground surface.

It is important to note that groundwater conditions are not static; fluctuation may be expected in the level and seepage flow depending on the season, amount of rainfall, surface water runoff, and other factors. Generally, the groundwater level is higher and seepage rate is greater in the wetter winter months (typically November through May).

5.17 Lab Soil Test Results

Laboratory test were conducted on representative soil samples to verify or modify the field soil classification of the units encountered, and to evaluate the general physical properties as well as the engineering characteristics of the soils encountered. The following provides information about the testing procedures performed on representative soil samples and the general condition of subsurface soil conditions encountered:

- Moisture Content (ASTM-D2216-92) test were performed on representative samples. In the upper layer of poorly graded gravel-sand mixes, the moisture content ranges from seven to ten percent.

- Grain Size Analyses (ASTM-D1140-97 and D422-90) were performed on samples collected from the proposed subbase. These tests indicate that soil consists predominantly of silt loam. Passing the #200 sieve result below, sample taken at 3.5- feet below the existing ground.

Test Pit	Percent Passing #200 sieve
1	68

- The result of laboratory tests performed on specific samples are provided at the appropriate sample depth on the individual test pit logs. However, it is important to note that some variation of subsurface conditions may exist. Our geotechnical recommendations are based on our interpretation of these test results.

5.18 Infiltration Testing

No infiltration test was performed.

5.19 Conclusion

This geotechnical site investigation report was prepared in accordance with accepted standard conventional principles and practices of geotechnical engineering. This investigation pertains only to material tested and observed as of the date of this report, and is based upon proposed site development as described in the text herein.

5.20 Limitations

Our recommendations and conclusions are based on the site materials observed, selective laboratory testing, engineering analyses, the design information provided to Engineering Northwest PLLC and our experience as well as engineering judgment. The conclusions and recommendations are professional opinions derived in a manner consistent with that level of care and skill ordinarily exercised by other members of the profession currently practicing under similar conditions in this area. No warranty is expressed or implied.

The recommendations submitted in this report are based upon the data obtained from the test pits. If soil variations do appear Engineering Northwest PLLC should be requested to reevaluate the recommendations contained in this report and to modify or verify them in writing prior to proceeding with the proposed construction.

LOG OF TEST PIT -1

Surface Elevation: 270
Boring Date: 01/12/2022
Boring Location: 3761706
Dilling Method: Machine

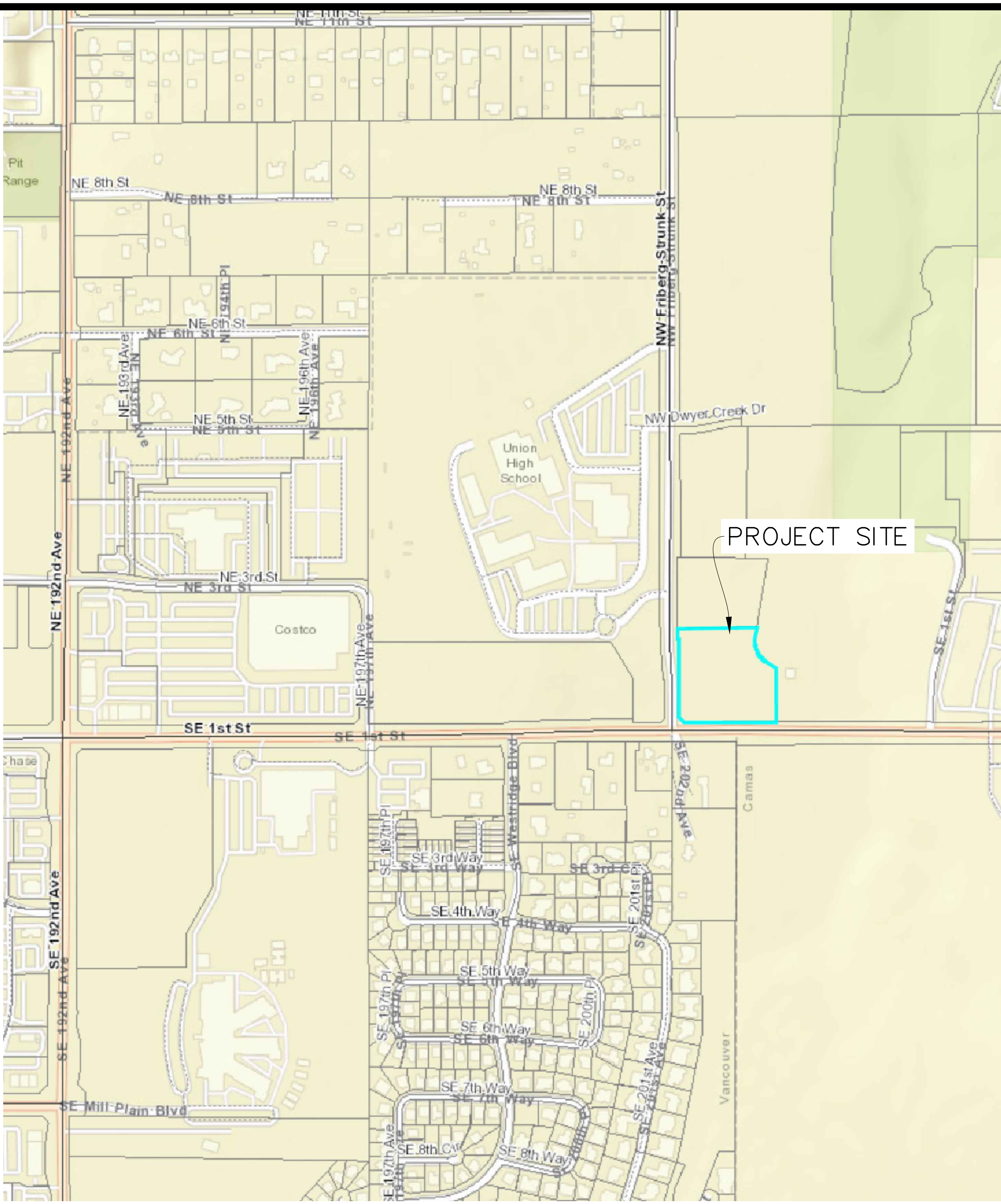
Depth	Remarks	COLOR	MOISTURE	MOISTURE CONTENT (%)	CONSISTENCY	Percent Pass #200		(ASHTO)	SOIL CLASSIFICATION
0					Hard		X		High organic content 4 to 12 inches of grass, roots and
		Dark Brown	Moist	8.5	Medium		■		topsoil Dark brown, low plasticity
5									
		Brown		11	Medium	68	■	A-4	Brown SILT with trace to some sand; moist.
									No ground water @ 6' below existing ground
10									
15									
20									
25									
30								END	Boring completed at and depth of about 6 feet below the ground surface.
35									

LOG OF BORING

ENGINEERING NORTHWEST	Oak Tree Station City of Camas WA	Plate 1

Engineering Northwest
6168 NE HWY 99 St 100
VANCOUVER WA 98685
360-931-3122

SITE VICINITY MAP



Project: Oak tree Station
Parcel #176162-000

FIGURE 1

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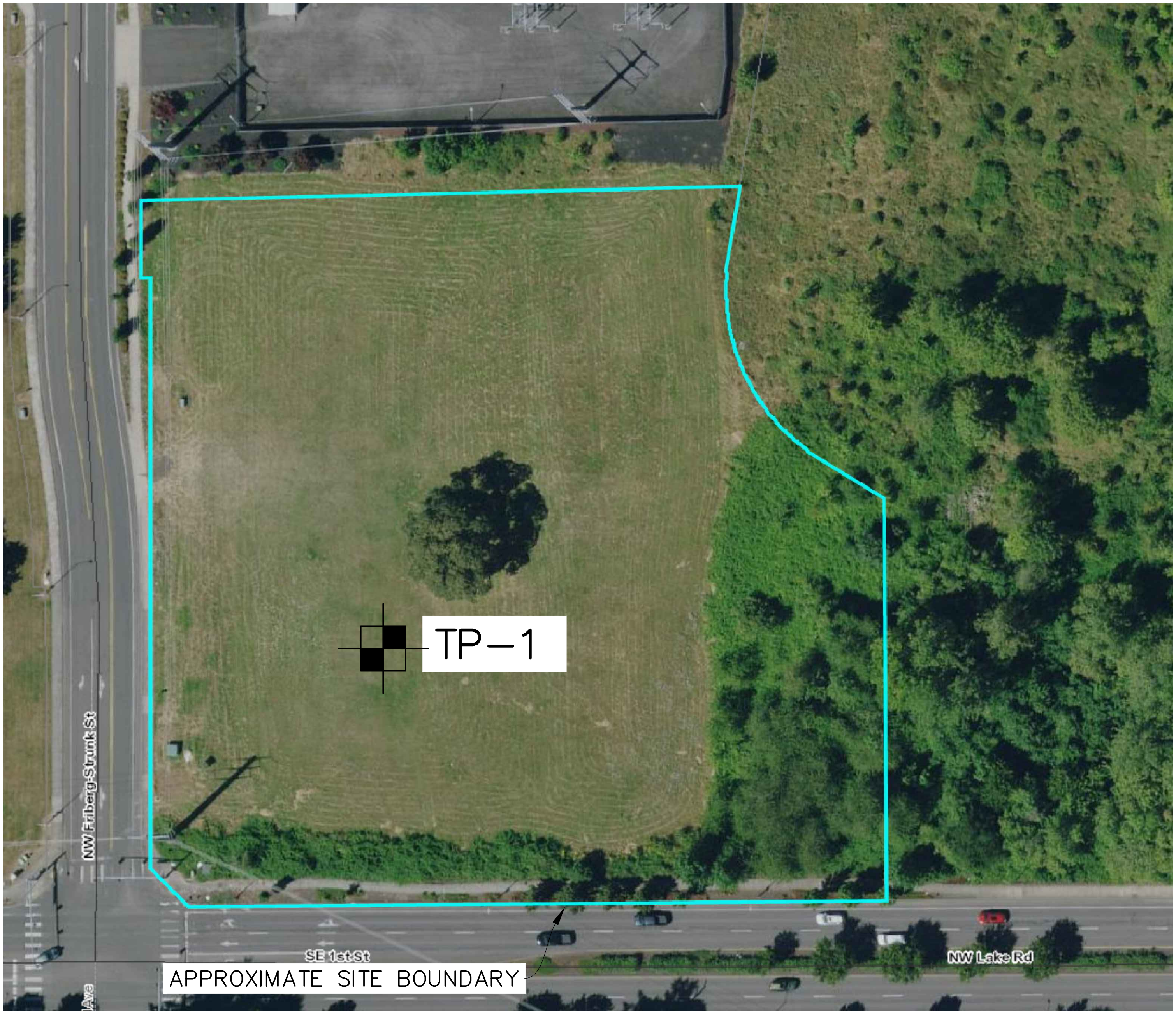
SITE AERIAL AND EXPLORATION LOCATION



Project: Oak tree Station
Parcel #176162-000

FIGURE 2

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SOIL MAP

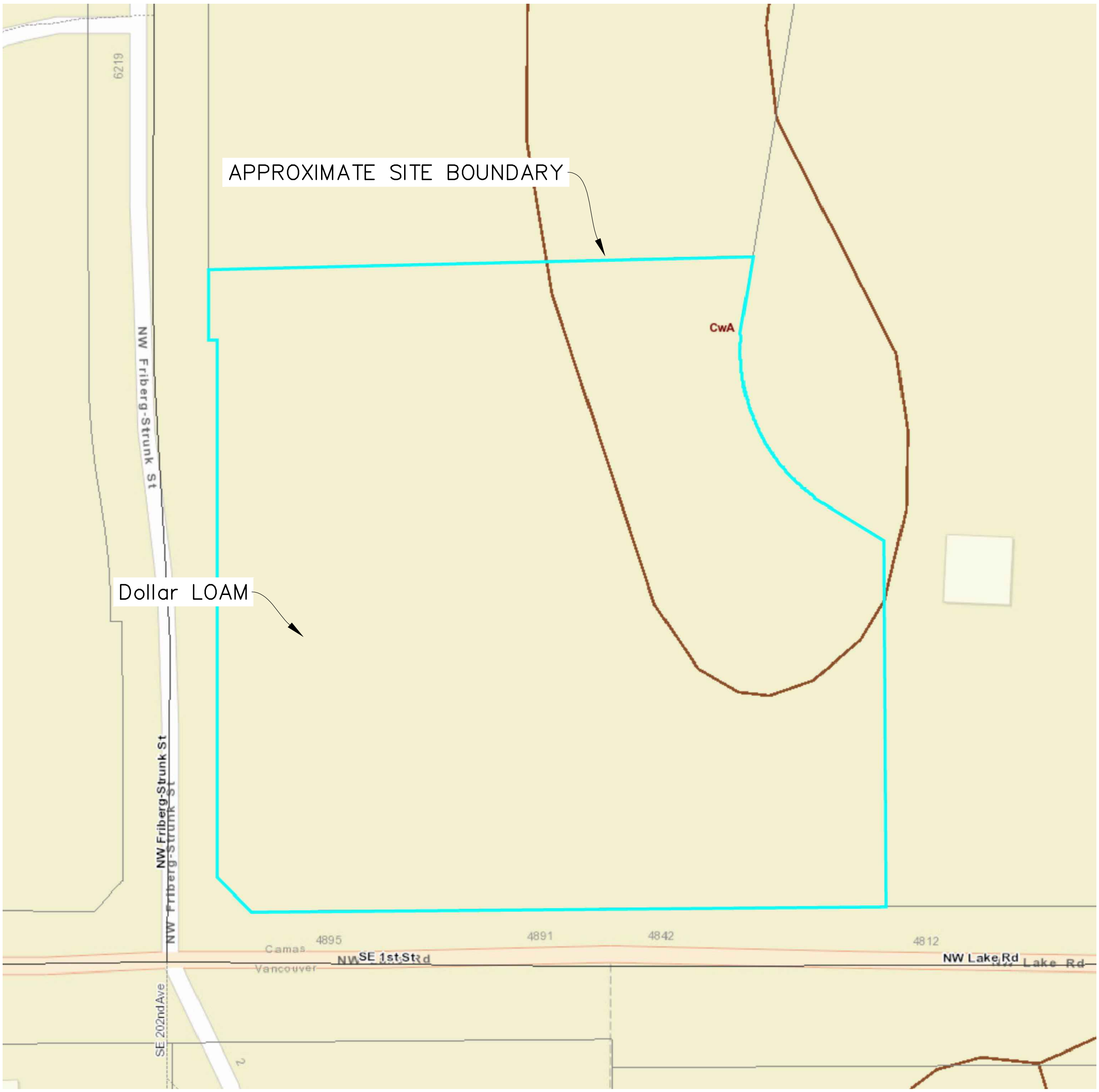

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FIGURE 3

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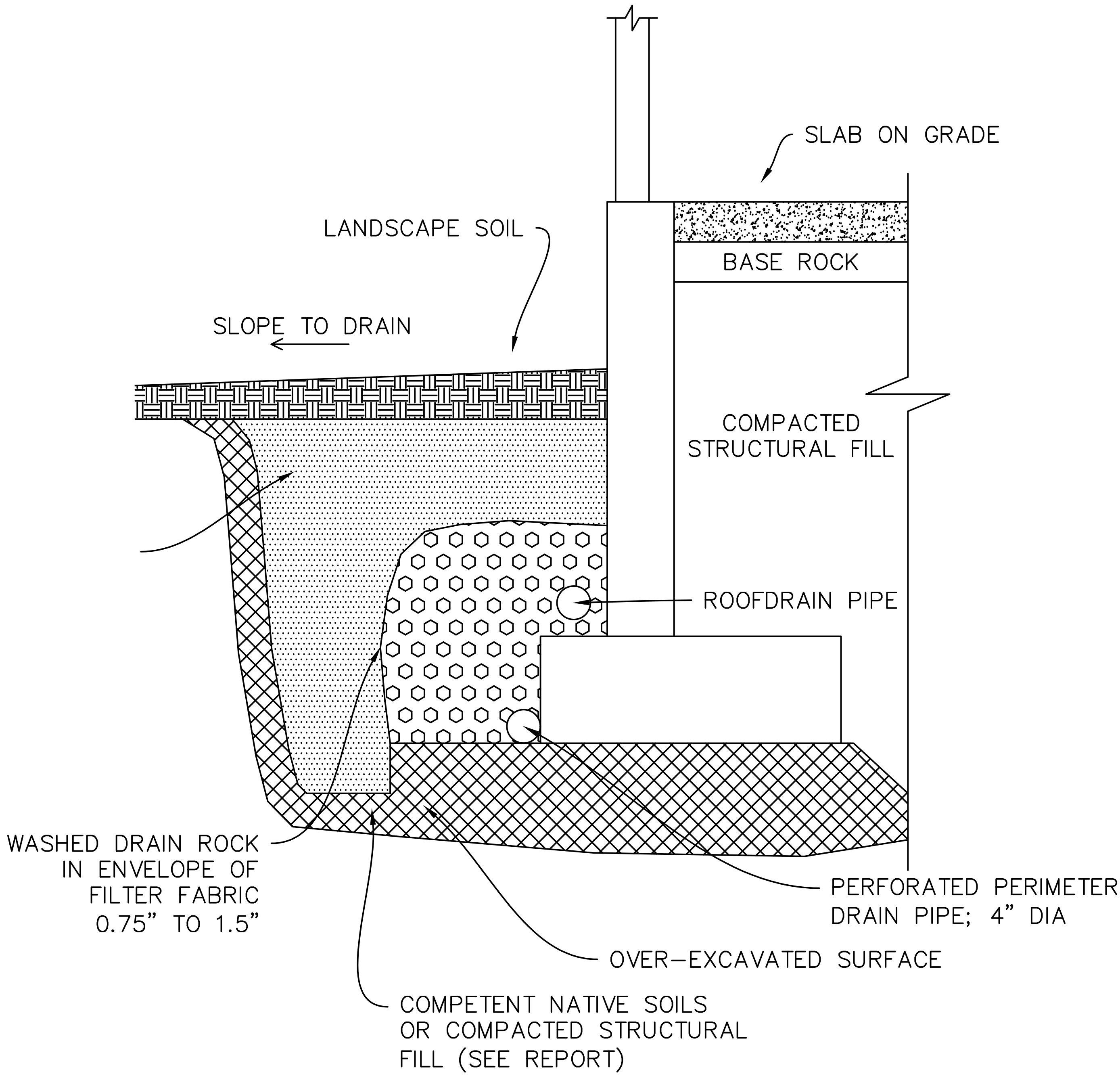
FOOTING DETAIL

Not to Scale



NOTES:

1. PERFORATED OR SLOTTED RIGID PVC PIPE WITH A POSITIVE DRAINAGE GRADIENT
2. FILTER SAND – FINE AGGREGATE FOR PORTLAND CEMENT; SECTION 9=03. 1(2)
3. FILTER FABRIC OPTIONAL IF FILTER SAND USED

Project: Oak tree Station
Parcel #176162-000

FIGURE 4

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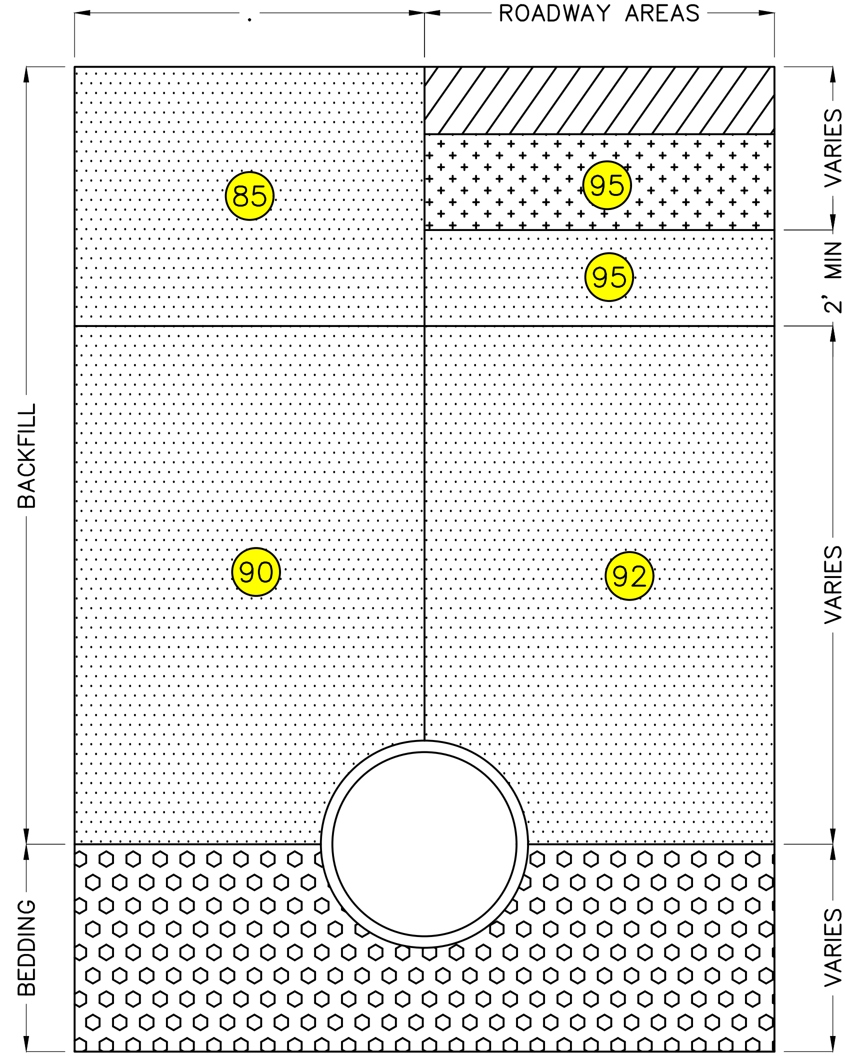
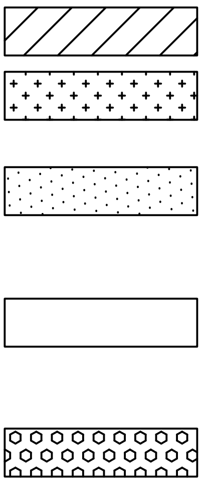
UTILITY TRENCH DETAIL



LEGEND



ASPHALT OR CONCRETE PAVEMENT

ROADWAY BASE MATERIAL OR BASE ROCK

BACKFILL: COMPACTED ON-SITE SOIL OR IMPORTED SELECT FILL MATERIALS AS DESCRIBED IN THE SITE PREPARATION OF THE GENERAL EARTHWORK SECTION OF THE ATTACHED REPORT TEXT

MINIMUM PERCENTAGE OF MAXIMUM LABORATORY DRY DENSITY AS DETERMINED BY ASTM TEST METHOD D1557 (MODIFIER PROCTOR), UNLESS OTHERWISE SPECIFIED IN THE ATTACHED REPORT TEXT

BEDDING MATERIAL: MATERIAL TYPE DEPENDS ON TYPE OF PIPE AND LAYING CONDITIONS. BEDDING SHOULD CONFORM TO THE MANUFACTURER'S RECOMMENDATIONS FOR THE TYPE OF PIPE SELECTED.

Project: Oak tree Station
Parcel #176162-000

FIGURE 5

Engineering Northwest
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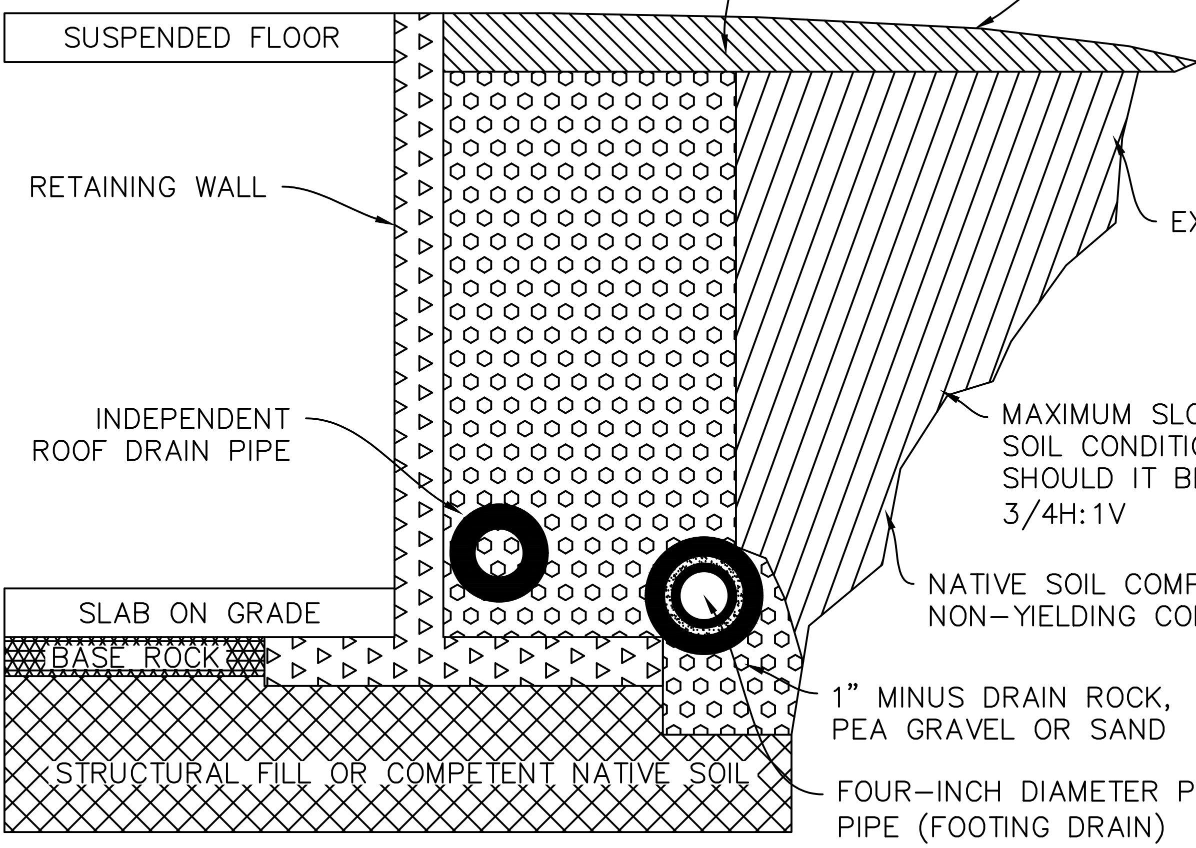
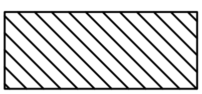
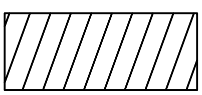
TYPICAL RETAINING WALL BACKFILL & FOOTING DRAINAGE



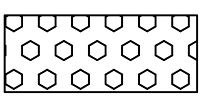
LEGEND



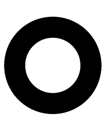
SURFACE SEAL: NATIVE SOIL OR OTHER LOW PERMEABILITY MATERIALS. SLOPE TO DRAIN AWAY FROM STRUCTURE



GRANULAR BACKFILL SHOULD CONSIST OF FREE DRAINING, ORGANIC FREE GRANULAR MATERIAL WITH A MAXIMUM SIZE OF 3"



DRAIN ROCK SHALL CONSIST OF CLEAN ONE INCH MINUS ROUND ROCK OR PEA GRAVEL WITH LITTLE TO NO FINES COLUMN OF DRAIN ROCK SHALL HAVE A MINIMUM WIDTH OF EIGHTEEN INCHES WITH ~TEN INCHES OF NATIVE SOIL COVER.



DRAIN PIPE: SMOOTH WALL PERFORATED OR SLOTTED RIGID PVC PIPE LAID WITH PERFORATIONS OR SLOTS FACING DOWN: TIGHT JOINTED; WITH A POSITIVE GRADIENT. DO NOT USE FLEXIBLE CORRUGATED PLASTIC PIPE. DRAIN LINE SHOULD BE BEDDED ON AND SURROUNDED WITH FREE DRAINING 1" MINUS ROCK, PEA GRAVEL, SAND AS DESIRED. THE DRAIN ROCK MAY BE ENCAPSULATED WITH A GEOTECHNICAL DRAINAGE FABRIC AT THE ENGINEER'S DISCRETION

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FIGURE 6

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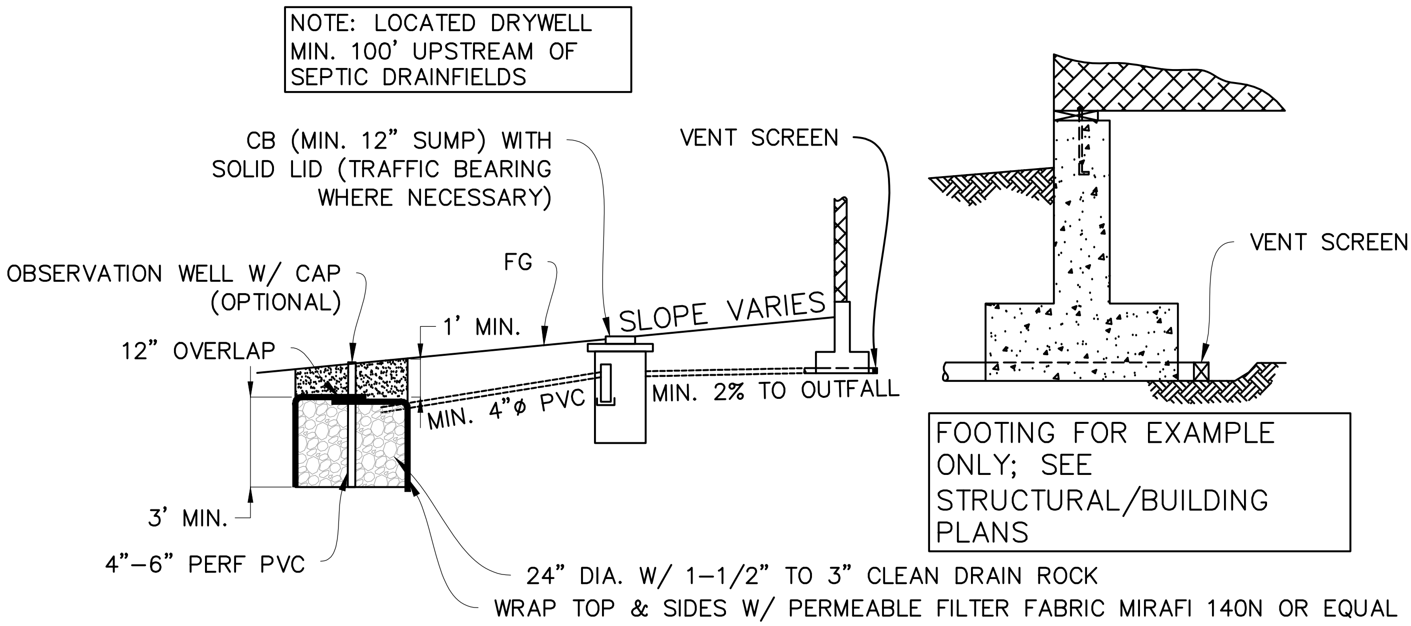
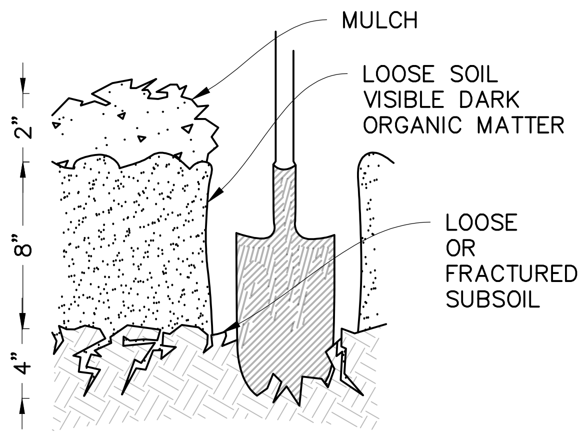
LOW POINT DRAIN SOIL QUALITY AND DEPTH



LOW POINT DRAIN

NTS



BMP T5.13 SOIL QUALITY AND DEPTH

NOT TO SCALE

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FIGURE 7

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SOIL LEGEND

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
Coarse Grained Soils	Gravel and Gravelly Soils	Clean Gravels (little or no fines)		GW gw	Well-Graded Gravels, Gravel-Sand Mixtures Little or no Fines
				GP gp	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines
	More than 50% Coarse Fraction Retained on No 4 Sieve	Gravels with Fines (appreciable amount of fines)		GM gm	Silty Gravels, Gravel-Sand-Silt Mixtures
				GC gc	Clayey Gravels, Gravel-Sand-Clay Mixtures
More Than 50% Material Larger Than No 200 Sieve Size	Sand and Sandy Soils	Clean Sand (little or no fines)		SW sw	Well-Graded Sands, Gravelly Sands Little or no Fines
				SP sp	Poorly-Graded Sands, Gravelly Sands Little or no Fines
	More Than 50% Coarse Fraction Passing No 4 Sieve	Sands with Fines (appreciable amount of fines)		SM sm	Silty Sands, Sand-Silt Mixtures
				SC sc	Clayey Sands, Sand-Clay Mixtures
Fine Grained Soils	Silts and Clays	Liquid Limit Less than 50		ML ml	Inorganic Silts and Very Fine Sands, Rock Flour, Silty-Clayey Fine Sands; Clayey Silts w/ slight Plasticity
				CL cl	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean
				OL ol	Organic Silts and Organic Silty Clays of Low Plasticity
More Than 50% Material Smaller Than No 200 Sieve Size	Silts and Clays	Liquid Limit Greater than 50		MH mh	Inorganic Silts, Micaceous or Diatomaceous Fine Sand or Silty Soils
				CH ch	Inorganic Clays of High Plasticity, Fat Clays
				OH oh	Organic Clays of Medium to High Plasticity, Organic Silts
Highly Organic Soils				PT pt	Peat, Humus, Swamp Soils with High Organic Contents

Topsoil		Humus and Duff Layer
Fill		Highly Variable Constituents

TYPICAL DESCRIPTIONS

Grab Somple	SPT Drive Sampler (ASTM D1586)	Shelby Tube Push Sampler (ASTM D1587)	Dames and Moore Drive Sampler (ASTM D3550

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FIGURE 8

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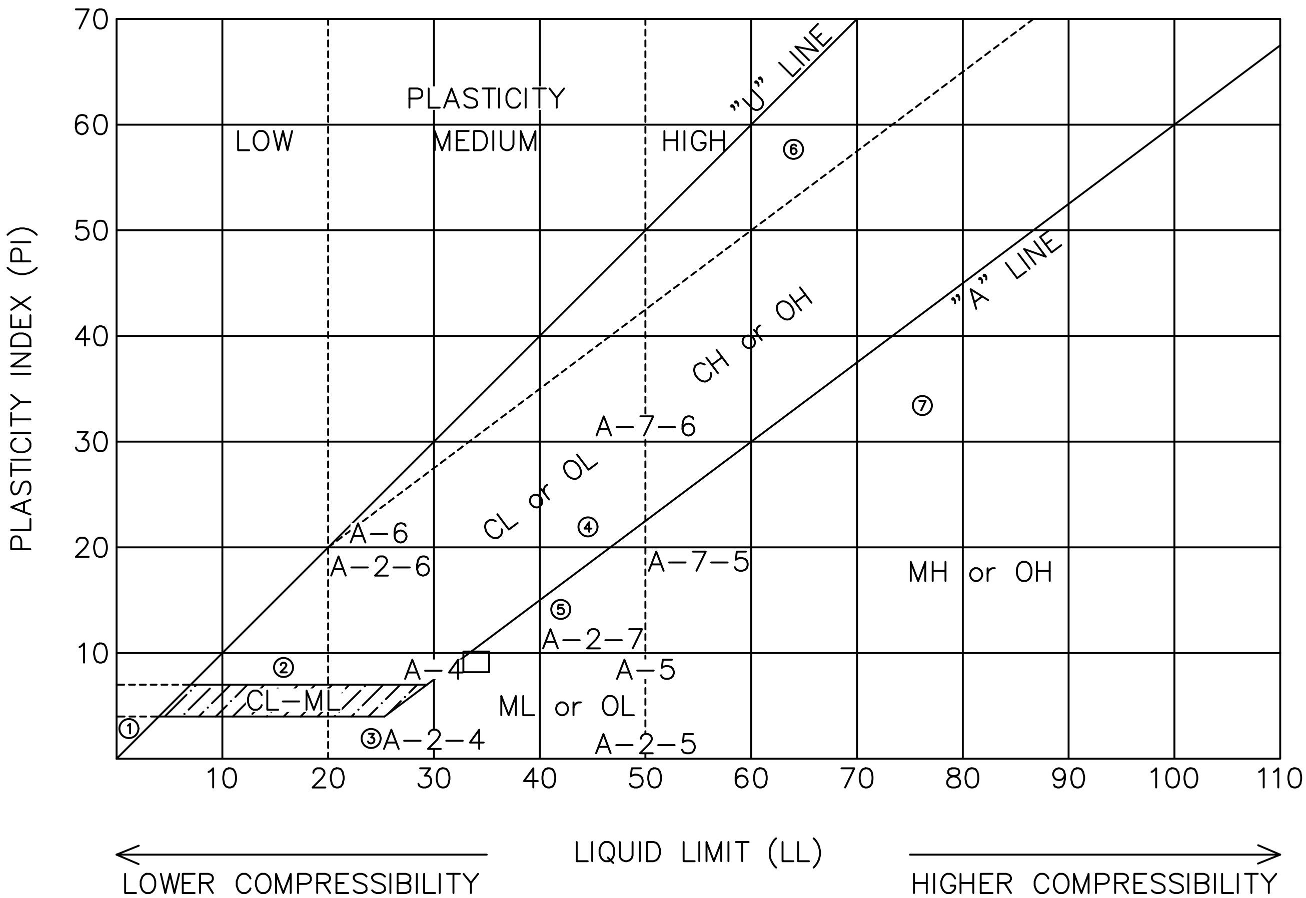
ATTERBERG LIMITS



LEGEND

☐ TP – 4 One feet below existing grade

1. COHESIONLESS SOILS
2. INORGANIC CLAYS, LOW PLASTICITY
3. INORGANIC SILTS, LOW COMPRESSIBILITY
4. INORGANIC CLAYS, MEDIUM PLASTICITY
5. INORGANIC SILTS AND ORGANIC CLAYS, MEDIUM COMPRESSIBILITY
6. INORGANIC CLAYS, HIGH PLASTICITY
7. INORGANIC SILTS AND ORGANIC CLAYS, HIGH COMPRESSIBILITY

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FIGURE 9