Stormwater Site Plan

Residential Treatment Facility North (RTFN)
32-Bed Community Residential Treatment Facility

29901 & 29919 80th Ave NW,
Stanwood, WA 98292

Snohomish County Project File No: 22 102230 CUP

Prepared by BCRA

June 2022

bcradesign.com

bcra

2106 Pacific Avenue, Suite 300
Tacoma, WA 98402

STORMWATER SITE PLAN

June 2022

PROJECT:
Residential Treatment Facility North (RTFN)
29901 & 29919 80th Ave NW,
Stanwood, WA 98292

OWNER:
Tulalip Tribes

OWNER REPRESENTATIVE:
The Wenaha Group
505 S 336th Street, Suite 630
Federal Way, WA 98003

ENGINEER:
BCRA Civil Engineering
2106 Pacific Avenue, Suite 300
Tacoma, WA 98402

PREPARED BY:
Michael Barene, EIT
mbarene@bcradesign.com

REVIEWED BY:
Zachary Crum, PE
zcrum@bcradesign.com

I hereby state that this drainage report for RTFN has been prepared by me or under my supervision and meets the standard of care and expertise which is usual and customary in this community for professional engineers. I understand that Snohomish County does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities prepared by me.

ZACHARY MERLIN CRUM
52111

06/24/2022

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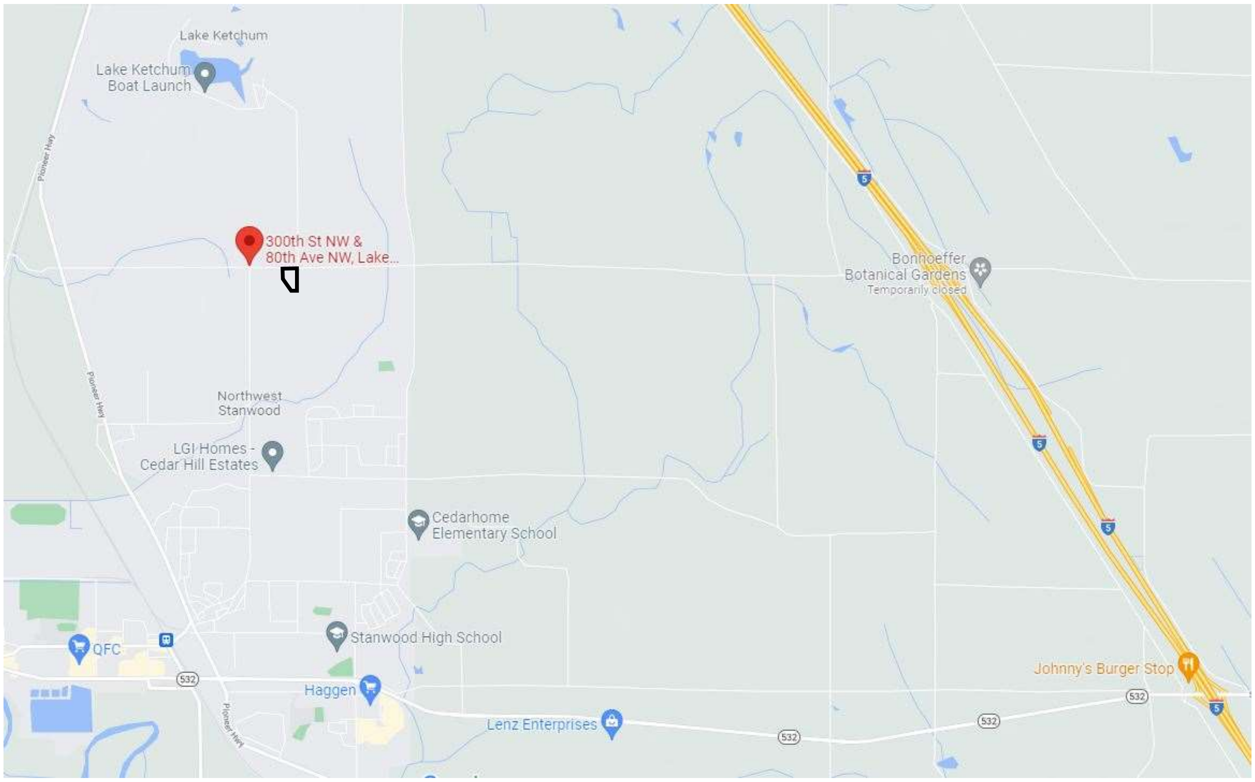
CHAPTER 1 - PROJECT OVERVIEW

This project includes two secure, 16-bed facilities (32 beds total) for in-patient residential behavioral health treatment with related site development including parking, frontage improvements, landscaping, and utilities, including septic. Construction of the buildings will be phased. The first phase will construct the southern building, a building pad for the northern building and install site infrastructure in support of both buildings. Phase two will construct the northern building at a later date. This Stormwater Site Plan assumes the fully built condition.

The buildings will be single story with approximately 15,000 sf each. The facilities will have an average length of stay for patients between 90-180 days are considered a Level II Heath and Social Service Facility per the Snohomish County Municipal Code (SCC) (30.91H.095). The project is located at 29919 80th Ave NW in Snohomish County north of Stanwood, WA on parcel #32041800100100.

The project is proposing a Boundary Line Adjustment (BLA), limiting development to the northeast corner of the site. The existing site consists of two parcels and a total of 30.22 acres. After required right of way dedication, the new development's parcel will be approximately 4.61 acres with the existing developed areas on the western half of the site on the remaining 25.61 acres. This report will only consider the site after the BLA has been completed. See Figure 1 for the project location.

Figure 1: Vicinity Map



The site will be accessed from 300th Street NW and include a drive aisle, parking, and a fire turn around. Sidewalks, trash enclosures, utility yards, and maintenance facilities will be constructed to serve the project functions. The septic system will be contained within an easement to the west of the new parcel. Stormwater will be collected and detained below the parking in an underground detention system and released into the buffer of the on-site wetlands.

CHAPTER 2 – EXISTING CONDITIONS SUMMARY

The existing parcels are currently used as a combined residential treatment facility and equestrian center. The site contains several houses, barns, sheds, and outbuildings. The eastern half and portion of site to be developed is pasture used for hay and horses. The steepest grades on site are approximately eight percent. The existing parcels are generally sloped to the center of the site toward two on-site wetlands and a drainage ditch that conveys runoff from the culvert and drainages along 300th Street NW on the north side of the property down the center of the site to the south property line. Both wetlands are category III wetlands and are located along the western boundary of the new parcel. On-site stormwater discharges to the wetlands via overland sheet flow.

The existing soils encountered generally consist of Vashon lodgment till, Vashon advance outwash and pre-Olympia glaciomarine deposits. Native soils were capped by surficial topsoil and fill. The geotechnical report dated December 9, 2021 was completed by Associated Earth Sciences, Inc. and is included in Appendix A of this report.

CHAPTER 3 - OFFSITE ANALYSIS

A preliminary review of the site was performed using survey data, aerial imagery, GIS data, and local photography. The analysis for upstream and downstream of the site is described below.

Upstream

The upstream analysis and review of local topography reveal limited area upstream of the site. The only off-site run-on is from the south half of 300th Street adjacent to the developed parcel. This area is 3,374 sq. ft. The permanent stormwater control plan has been developed to account for the off-site run-on.

Downstream

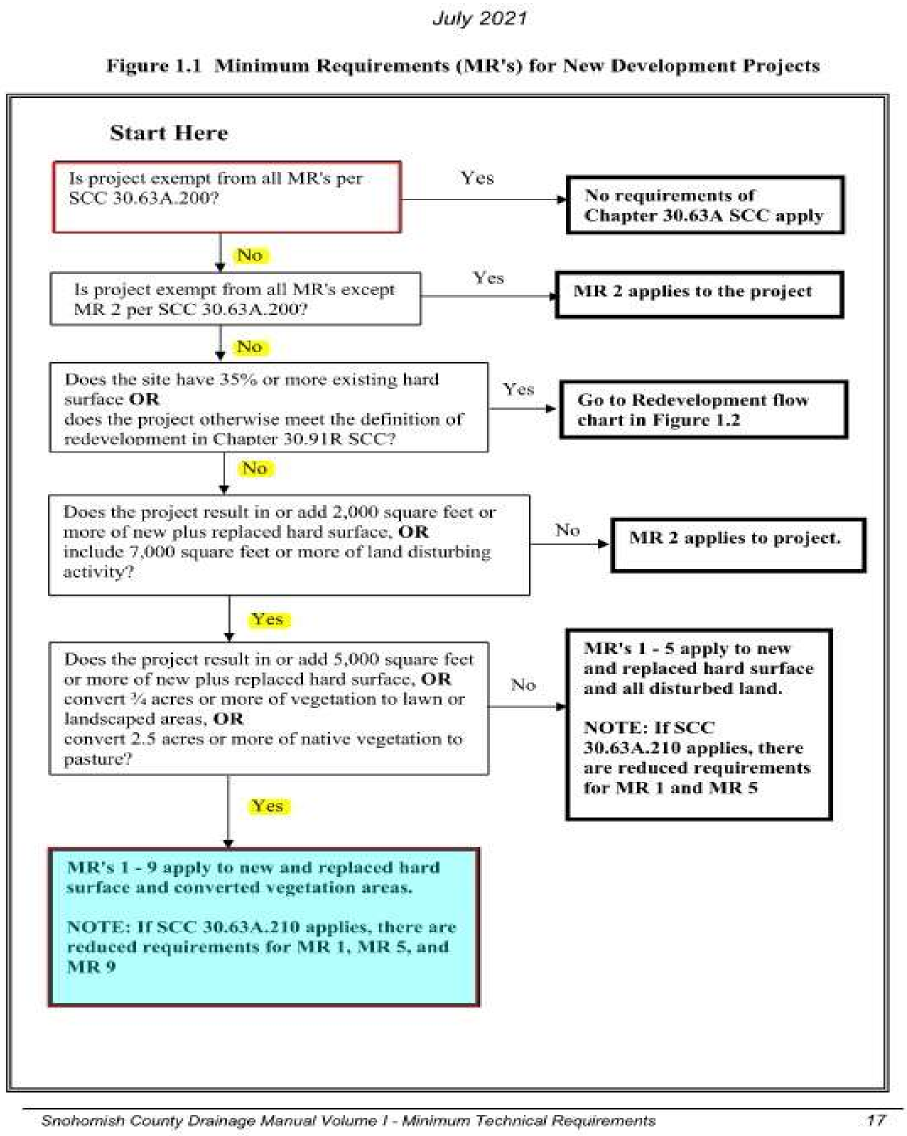
Downstream surface receiving water is the wetland. The wetland is an adequate receiving water. If there is any negative impact it will be because the wetland is receiving less water than in the predeveloped condition. New impervious surfaces will reduce the volume of water going to groundwater sources which feed the wetland. The downstream analysis reveals no potential for flooding to occur. The onsite wetlands ultimately discharge to Douglas Creek within the Skagit Watershed.

CHAPTER 4 – PERMANENT STORMWATER CONTROL PLAN

SECTION 1 – PERFORMANCE GOALS AND STANDARDS

Stormwater facilities for this project are designed to meet the drainage requirements of SCC Chapter 30.63A and Volumes I-VI of the Snohomish County Drainage Manual (SCDM) as described in this report. The project meets the thresholds for a Full Stormwater Site Plan per the Drainage Review Submittal Checklist. The project is a new development project with more than 5,000 square feet of new plus replaced hard surface. Minimum Requirements (MR) 1-9 apply to new and replaced hard surfaces and converted vegetation areas. A discussion of how the project will meet each of the applicable requirements is provided in Chapter 5.

Figure 2: New Development MR Flowchart



July 2021

Figure 1.1 Minimum Requirements (MR's) for New Development Projects

Start Here

Is project exempt from all MR's per SCC 30.63A.200?

Yes → **No requirements of Chapter 30.63A SCC apply**

No

Is project exempt from all MR's except MR 2 per SCC 30.63A.200?

Yes → **MR 2 applies to the project**

No

Does the site have 35% or more existing hard surface **OR** does the project otherwise meet the definition of redevelopment in Chapter 30.91R SCC?

Yes → **Go to Redevelopment flow chart in Figure 1.2**

No

Does the project result in or add 2,000 square feet or more of new plus replaced hard surface, **OR** include 7,000 square feet or more of land disturbing activity?

No → **MR 2 applies to project.**

Yes

Does the project result in or add 5,000 square feet or more of new plus replaced hard surface, **OR** convert ¾ acres or more of vegetation to lawn or landscaped areas, **OR** convert 2.5 acres or more of native vegetation to pasture?

No → **MR's 1 - 5 apply to new and replaced hard surface and all disturbed land.**

NOTE: If SCC 30.63A.210 applies, there are reduced requirements for MR 1 and MR 5

Yes

MR's 1 - 9 apply to new and replaced hard surface and converted vegetation areas.

NOTE: If SCC 30.63A.210 applies, there are reduced requirements for MR 1, MR 5, and MR 9

Snohomish County Drainage Manual Volume I - Minimum Technical Requirements 17

SECTION 2 – EXISTING SITE HYDROLOGY

The project site is comprised of a single threshold discharge area (TDA). In the existing condition, runoff from the site sheet flows from northeast to southwest. There is a roadside ditch along the eastern portion of the 300th Street frontage leading to the north wetland. There is no other stormwater infrastructure in the existing condition. Stormwater runoff drains to the wetlands via overland sheet flow. Existing basin areas for the project site are provided in Table 1 below.

Table 1: Existing Basin Areas

Basin	Description	Impervious (ac)	Pervious (ac)	Total (ac)
Basin A	Onsite area and ROW	0.0946	3.3640	**3.4586**

A geotechnical evaluation was performed for the site. Groundwater was noted at a depth of 17 feet as well as the presence of shallow perched groundwater. Shallow infiltration was not recommended, and deeper infiltration strategies were also determined to have low potential at this site. Further discussion can be found in the geotechnical report in Appendix A.

SECTION 3 – DEVELOPED SITE HYDROLOGY

The developed site is located within one TDA. The project proposes to maintain the natural site hydrology and protect downstream wetlands to the maximum extent feasible. In the developed condition, stormwater management facilities will discharge to the onsite wetland to maintain the existing hydroperiod of the wetland. Runoff from pollution generating surfaces will be conveyed to either a bioretention cell or Modular Wetland Systems for treatment upstream of an ADS Stormtech MC-3500 chamber detention facility. Basin 1 includes all areas that drain to the detention facility including building roofs, ROW, drive aisle and parking. Basin 2 includes all areas that will bypass the detention facility due to the topography of the site and include landscape areas and the maintenance access road on the south side of the south building. Developed basin areas are provided in Table 2 below. The developed basin map is provided in Figure 4.

Table 2: Proposed Basin Areas

Basin	Description	Impervious (ac)	Pervious (ac)	Total (ac)
Basin 1	Onsite to Stormtech	1.7311	0.7025	**2.4336**
Basin 2	Bypass	0.3145	0.7104	**1.0250**
Total	Total basin areas	2.0456	1.4129	**3.4586**

RESIDENTIAL TREATMENT FACILITY NORTH

FIGURE 3: EXISTING CONDITION BASIN MAP

JUNE 2022

RESIDENTIAL TREATMENT FACILITY NORTH

FIGURE 4: DEVELOPED CONDITION BASIN MAP

SECTION 4 - FLOW CONTROL SYSTEM

A stormwater detention system sized to meet the flow control requirement will not release the required flow volumes during the summer months to meet the wetland hydroperiod protection. See Figures 5-7. For this reason, MR8 is prioritized according to Volume I Chapter 2.5.8 Page 29 “Reconciling the Flow Control Performance Standard from MR7 with MR8,” of the SCDM. The detention system was sized to provide some level of flow attenuation but primarily maintain runoff volumes to the wetland. Refer to section 6 for a discussion of the wetland protection system. The results from the flow control modeling are shown in Figure 18: Flow Control Duration Results

SECTION 5 - WATER QUALITY SYSTEM

According to Chapter 4.2 Step 5 of the manual, the project is required to provide Enhanced water quality treatment. Two Modular Wetland Systems (MWS) will be sized per GULD documentation included in Appendix B and coordinated with the manufacturer, BioClean, using design information from the water quality basins. Design flow rates from WWHM are provided below in Table 4. The water quality basin map can be found in Figure 11: Water Quality Basin Map.

The two MWS will provide treatment for the parking, drive aisle, and fire turn-around. The maintenance access road running around the east end of the building and connecting to the western parcel through an access and utility easement is for maintenance and deliveries only. This low volume road will see infrequent use and therefore is not considered to be pollution generating impervious surface and will not require treatment prior to dispersing to recharge the wetland. Stormwater runoff from the 300th Street right-of-way will sheet flow to a road-side ditch that leads to a bioretention cell. Because native soils are not suitable for infiltration, the bioretention cell will include and underdrain. The bioretention cell has been sized to infiltrate the 91% water quality design storm through the engineered bioretention soil media mix. The WWHM water quality flow rate calculations are included in the following figures.

Table 3: Water Quality Basin Areas

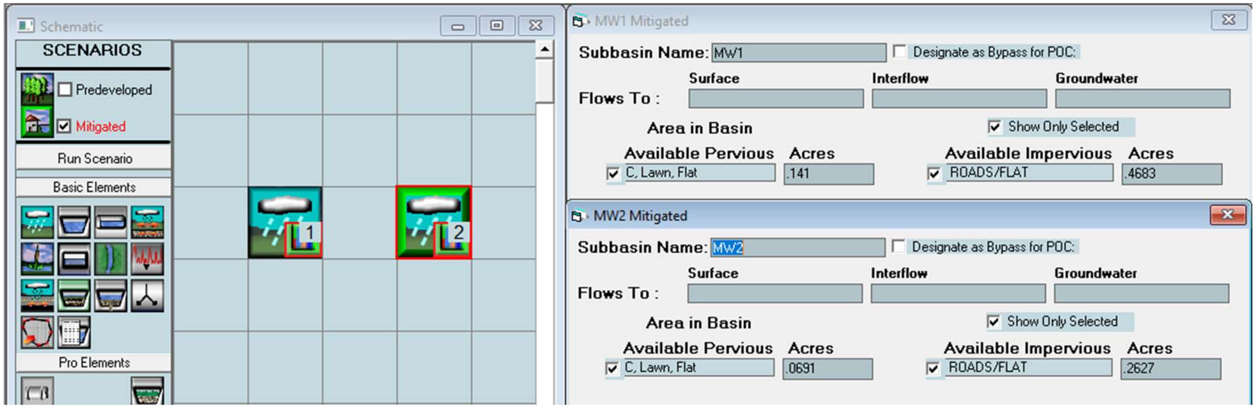
Basin	Description	Impervious (ac)	Pervious (ac)	Total (ac)
MWS#1	Parking area west of building	0.4683	0.1410	**0.6093**
MWS#2	East side and fire turn around	0.2627	0.0691	**0.3318**
Bioretention	Runoff from ROW	0.1905	0.4972	**0.6877**

Table 4: Water Quality Flow Rates

Basin	Off-line WQ flow rate (cfs)	Treatment Facility	Facility Size
West WQ Basin	0.041	MWS #1	**5’x7.33’x4.17’**
East WQ Basin	0.023	MWS #2	**5’x7.33’x4.58’**

Bio WQ Basin	95.77% filtered	Bioretention	**0.068 ac-ft**

Figure 5: Modular Wetland Modeling Schematic



MW1 Mitigated

Subbasin Name: MW1 ☐ Designate as Bypass for POC:

Flows To : Surface Interflow Groundwater

Area in Basin ☑ Show Only Selected

Available Pervious Acres: ☑ C, Lawn, Flat .141

Available Impervious Acres: ☑ ROADS/FLAT .4683

MW2 Mitigated

Subbasin Name: MW2 ☐ Designate as Bypass for POC:

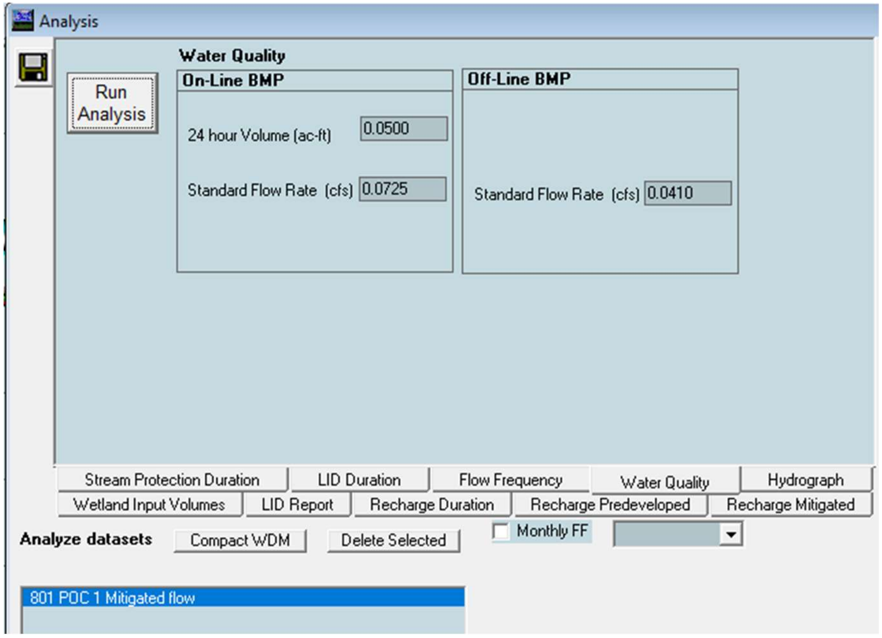
Flows To : Surface Interflow Groundwater

Area in Basin ☑ Show Only Selected

Available Pervious Acres: ☑ C, Lawn, Flat .0691

Available Impervious Acres: ☑ ROADS/FLAT .2627

Figure 6: Modular Wetland #1 Flow rate



Water Quality

On-Line BMP

24 hour Volume (ac-ft) 0.0500

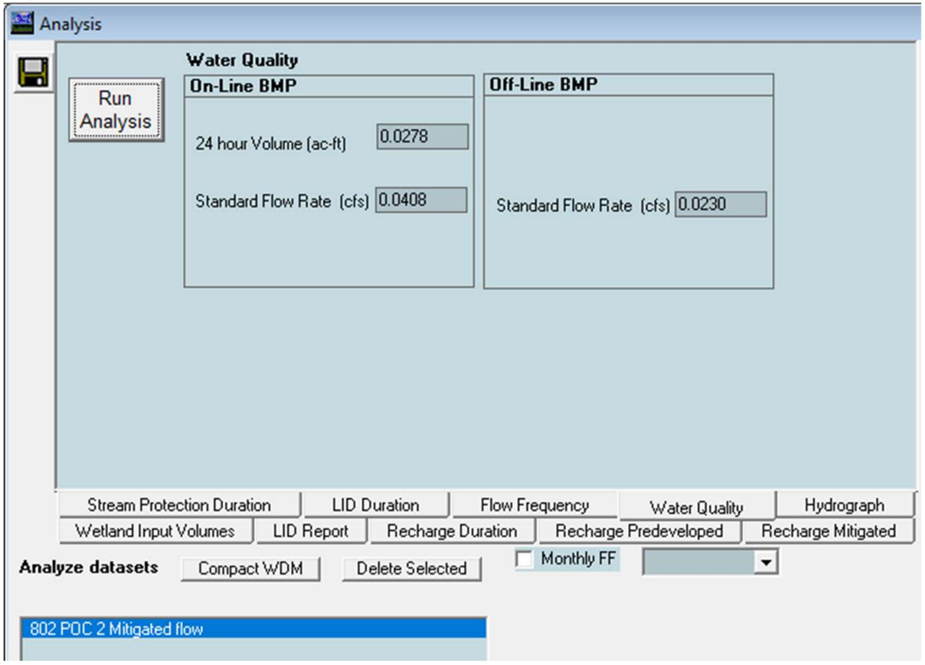
Standard Flow Rate (cfs) 0.0725

Off-Line BMP

Standard Flow Rate (cfs) 0.0410

801 POC 1 Mitigated flow

Figure 7: Modular Wetland #2 Flow rate



Water Quality

On-Line BMP

24 hour Volume (ac-ft) 0.0278

Standard Flow Rate (cfs) 0.0408

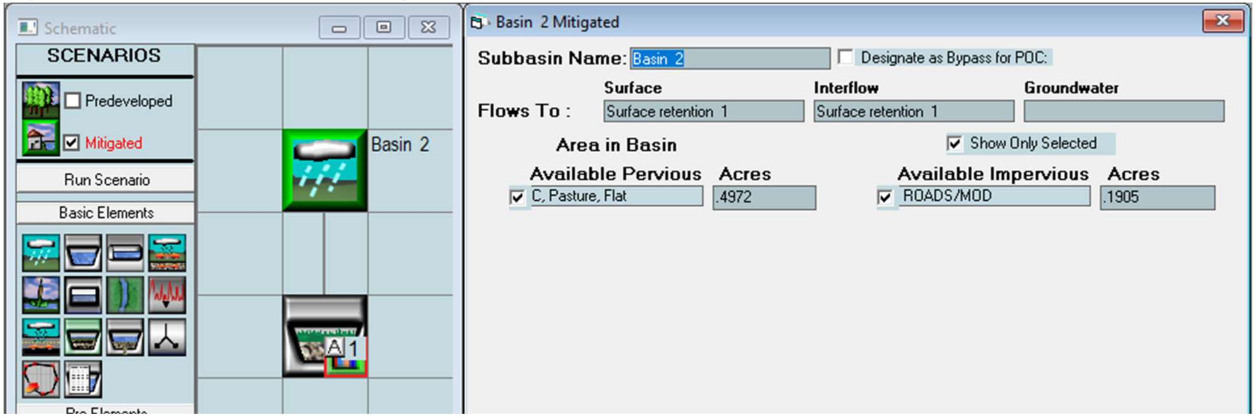
Off-Line BMP

Standard Flow Rate (cfs) 0.0230

☐ Monthly FF

802 POC 2 Mitigated flow

Figure 8: Bioretention Modeling Schematic



☐ Predeveloped

☑ Mitigated

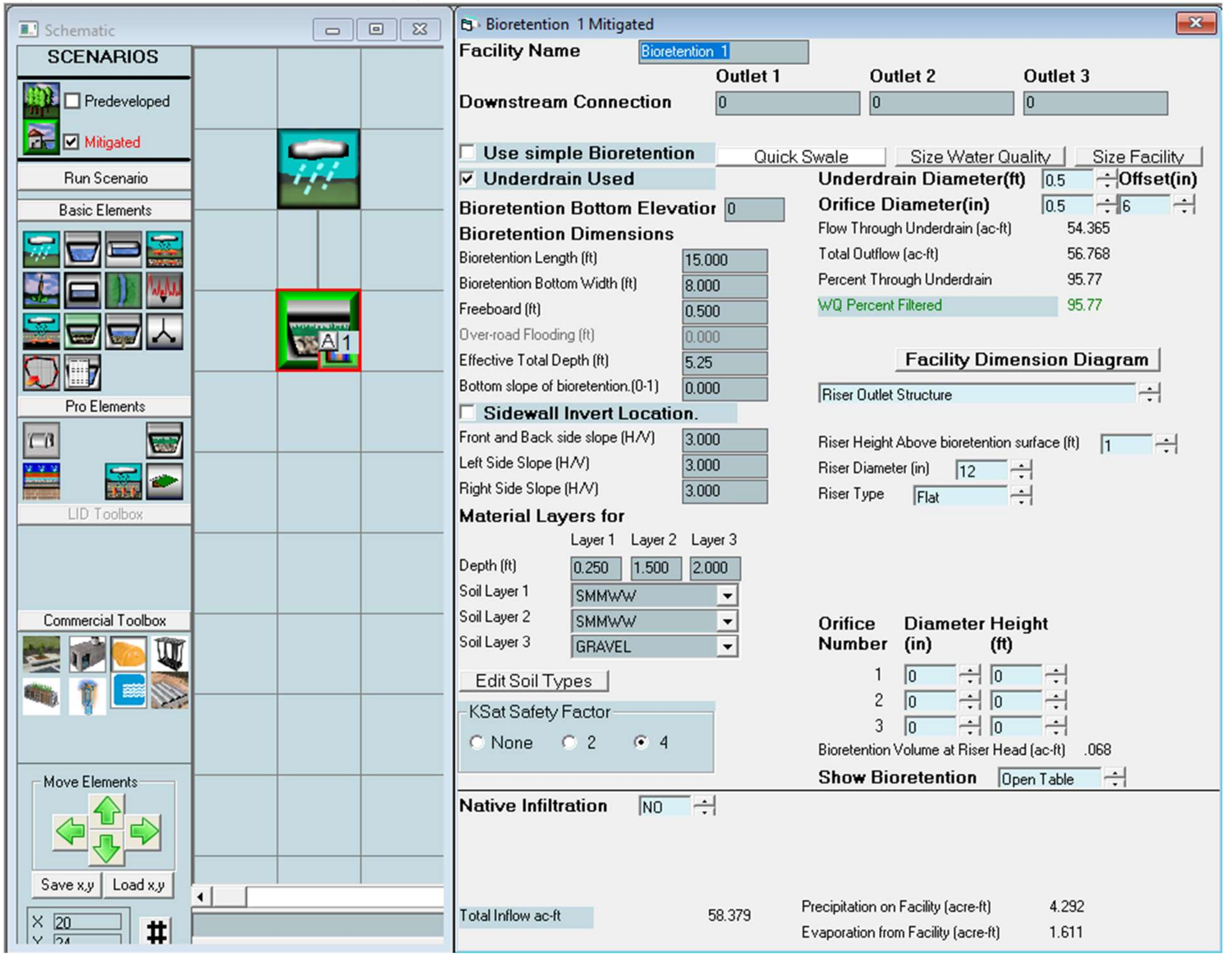
Subbasin Name: Basin 2 ☐ Designate as Bypass for POC:

Flows To : **Surface** Surface retention 1 **Interflow** Surface retention 1 **Groundwater**

Area in Basin ☑ Show Only Selected

Available Pervious	Acres	Available Impervious	Acres
☑ C, Pasture, Flat	.4972	☑ ROADS/MOD	.1905

Figure 9: Bioretention Cell Configuration



Scenarios: ☐ Predeveloped ☑ Mitigated

X 20
Y 24

Facility Name: Bioretention 1

	Outlet 1	Outlet 2	Outlet 3
Downstream Connection	0	0	0

☐ Use simple Bioretention

☑ Underdrain Used

Underdrain Diameter(ft) 0.5 Offset(in)

Orifice Diameter(in) 0.5 6

Bioretention Bottom Elevation 0

Bioretention Dimensions

Bioretention Length (ft)	15.000
Bioretention Bottom Width (ft)	8.000
Freeboard (ft)	0.500
Over-road Flooding (ft)	0.000
Effective Total Depth (ft)	5.25
Bottom slope of bioretention.(0-1)	0.000

Flow Through Underdrain (ac-ft)	54.365
Total Outflow (ac-ft)	56.768
Percent Through Underdrain	95.77
WQ Percent Filtered	95.77

☐ Sidewall Invert Location.

Front and Back side slope (H/V)	3.000
Left Side Slope (H/V)	3.000
Right Side Slope (H/V)	3.000

Riser Outlet Structure

Riser Height Above bioretention surface (ft)	1
Riser Diameter (in)	12
Riser Type	Flat

Material Layers for

	Layer 1	Layer 2	Layer 3
Depth (ft)	0.250	1.500	2.000

Soil Layer 1	SMMWW
Soil Layer 2	SMMWW
Soil Layer 3	GRAVEL

Orifice Number	Diameter (in)	Height (ft)
1	0	0
2	0	0
3	0	0

KSat Safety Factor: ○ None ○ 2 ● 4

Bioretention Volume at Riser Head (ac-ft) .068

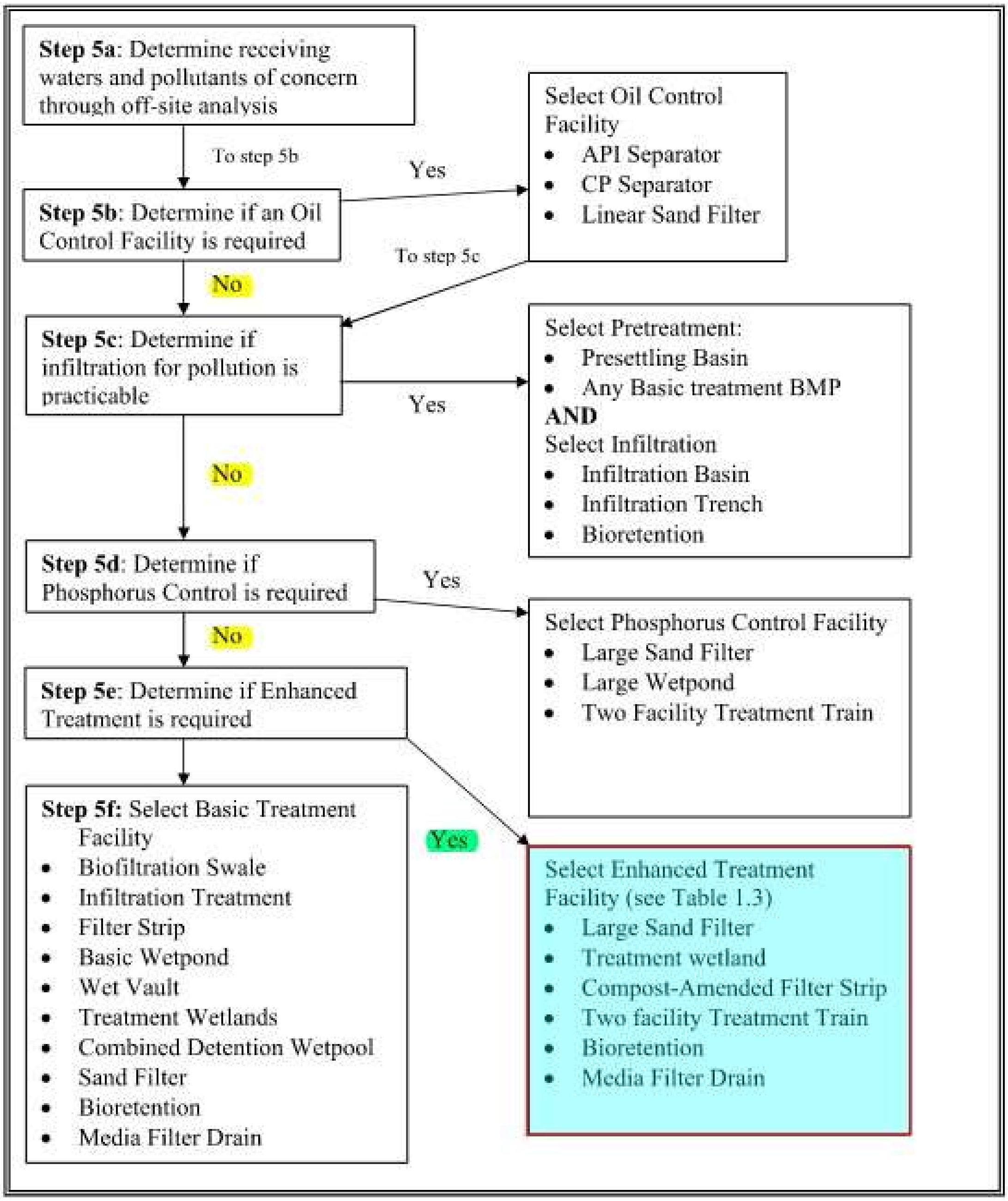
Show Bioretention

Native Infiltration NO

Total Inflow ac-ft	58.379
Precipitation on Facility (acre-ft)	4.292
Evaporation from Facility (acre-ft)	1.611

Figure 10: Treatment Facility Selection Flowchart

Figure 1.4 Treatment Facility Selection Flow Chart



RESIDENTIAL TREATMENT FACILITY NORTH

FIGURE 14: WATER QUALITY BASIN MAP

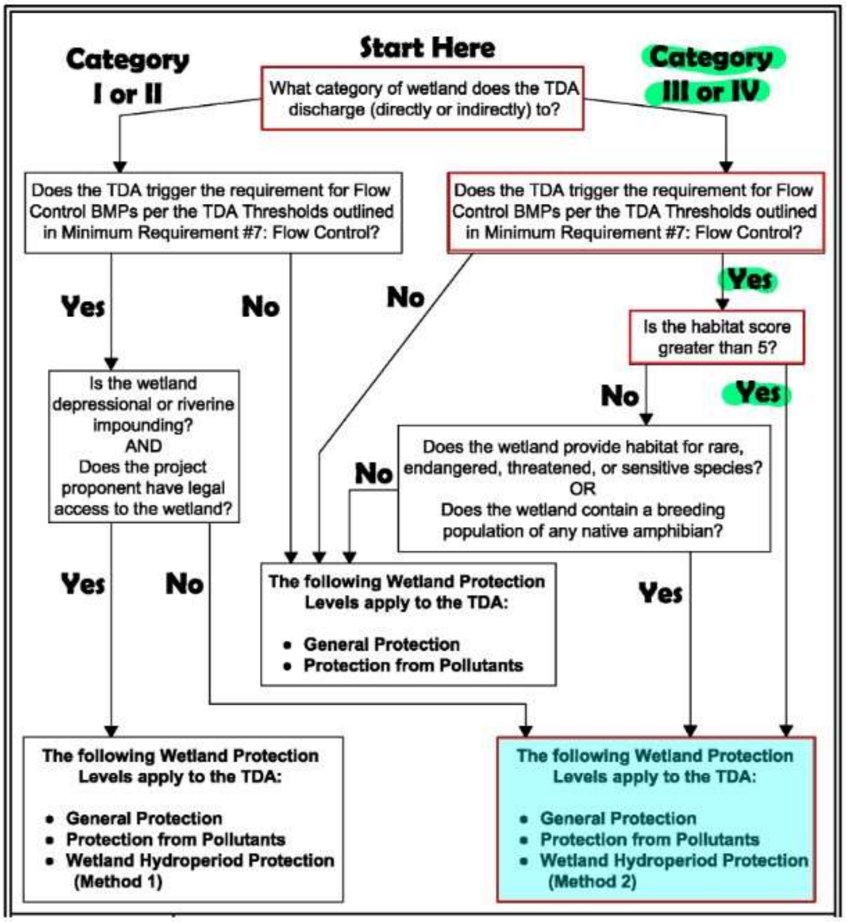
SECTION 6 – WETLANDS PROTECTION SYSTEM

Refer to Section 4 of this chapter and Chapter 5 for justification of the wetlands protection design. When sizing a system to meet the flow control requirement, the smallest passing system fails wetlands protection. For this reason, MR8, Wetlands Protection is prioritized.

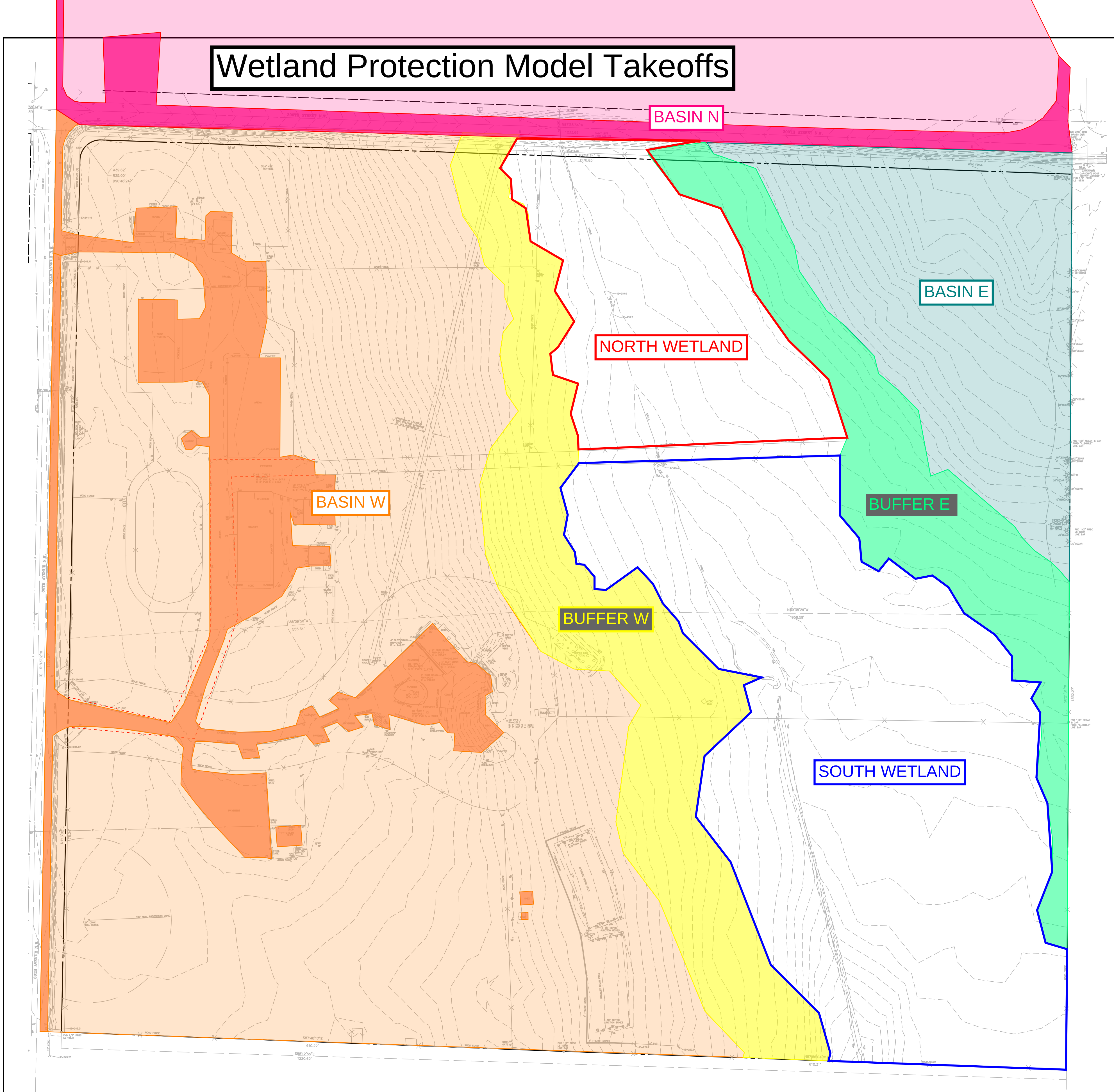
The modeling guidance under Volume I Appendix I-D of the SCDM for Method 2 of evaluation was followed to size the stormwater facilities. Wetland hydroperiod protection is only required for the south wetland due to the Category III and habitat score of 6. All impervious and pervious surfaces, on-site and off-site, that drain to the south wetland were considered when delineating the basins. The north wetland drains to the south wetland thus, both wetlands were considered one wetland subject to wetland hydroperiod protection. Wetland area was not included in the take offs. The existing wetland protection basin areas can be found in Table 5 and the basin map can be found in Figure 13 on the following page. Wetland buffer areas can be found in Table 6.

Figure 12: Wetland Protection Flowchart

Figure 1.4 – Levels of Wetland Protection



Wetland Protection Model Takeoffs



SITE INFORMATION

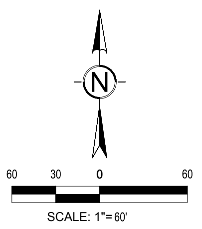
- SITE AND ALL ADJACENT SITES ARE DESIGNATED FEMA FLOOD HAZARD ZONE X.
- NOT A CRITICAL AREA SITE PLAN.
- NOT LOCATED WITHIN A CRITICAL AQUIFER RECHARGE AREA.
- PARKING REQUIREMENTS DETERMINED BY THE DEPARTMENT BASED ON COMPARABLE USES.
- ZONING: R-5

PROJECT PARKING REQUIREMENTS:

STANDARD STALLS:	28 PER BUILDING
ADA STALLS:	2 PER BUILDING
TOTAL:	60 STALLS

SITE AREAS SUMMARY

TOTAL SITE AREA	221,509 SF	5.09 AC
BUILDING FOOTPRINT AREAS	34,734 SF	0.80 AC
EXISTING IMPERVIOUS SURFACE	658 SF	0.02 AC
PROPOSED NEW PLUS REPLACED IMPERVIOUS SURFACE	86,500 SF	1.99 AC



SCALE: 1"= 60'

IF SHEET MEASURES LESS THAN 24"X36", IT IS A REDUCED PRINT. REDUCE SCALE ACCORDINGLY

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bcra
T 253.627.4367 F 253.627.4395 WWW.BCRADESIGN.COM
2106 PACIFIC AVENUE, SUITE 300, TACOMA, WA 98402

SEAL

ZACHARY MERLIN CRUM
52111
PRELIMINARY

PROJECT

RESIDENTIAL TREATMENT FACILITY NORTH
29901 & 29919 80TH AVE NW
STANWOOD, WA 98292

REVISIONS

DATE
01.28.2022

BCRA NO.
21031

DRAWN BY: RJB DESIGNED BY: MB

REVIEWED BY: ZMC

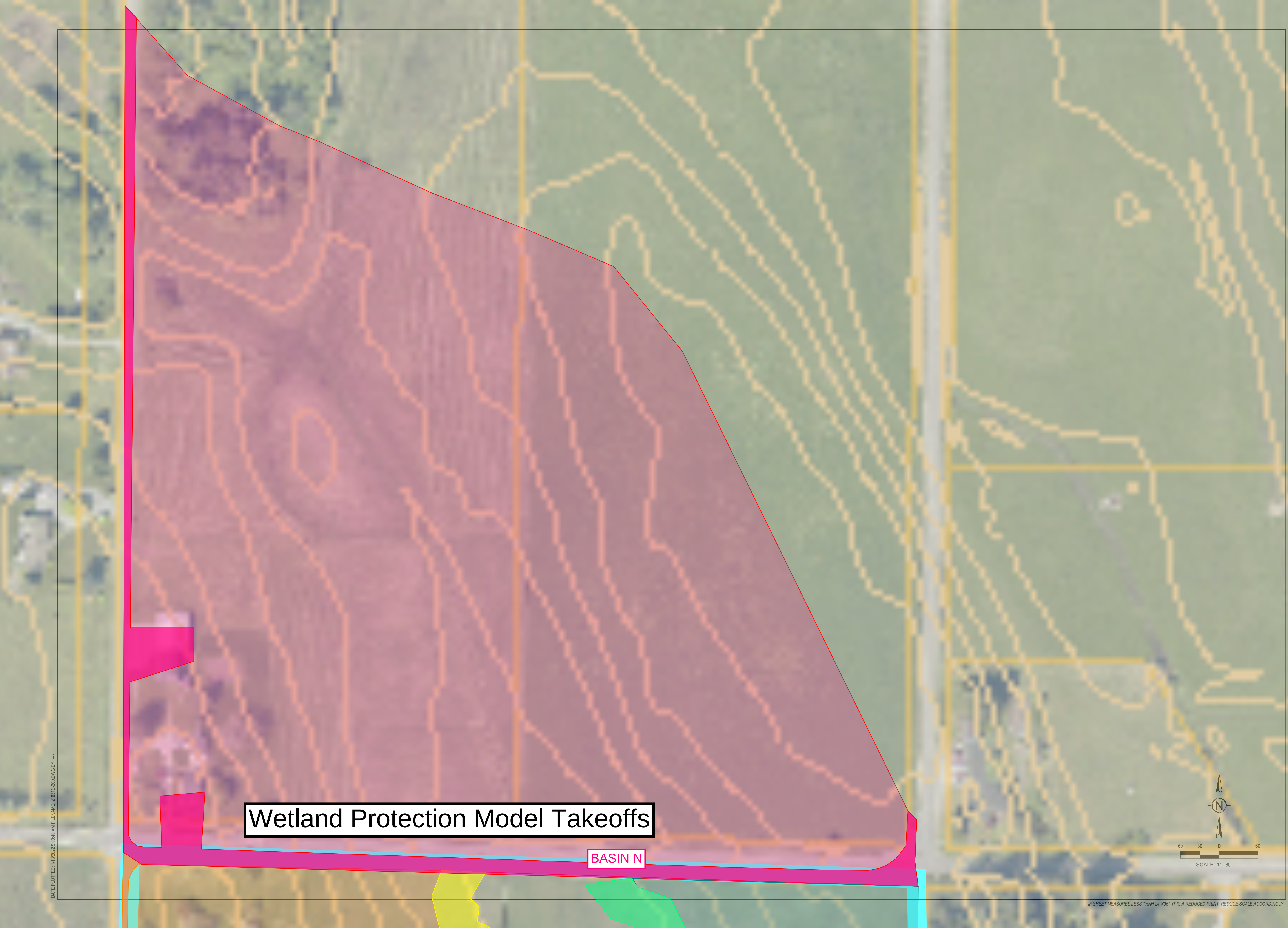
SHEET TITLE
OVERALL SITE PLAN

bcra

SHEET

C-200

LAND USE SUBMITTAL


Wetland Protection Model Takeoffs

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T 253.627.4367 F 253.627.4395 WWW.BCRADESIGN.COM
2106 PACIFIC AVENUE, SUITE 300, TACOMA, WA 98402

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REVIEWED BY: ZMC

SHEET TITLE
OVERALL SITE PLAN

bcra

SHEET
C-200

SD SET

Table 5: Existing Wetland Protection Basin Areas

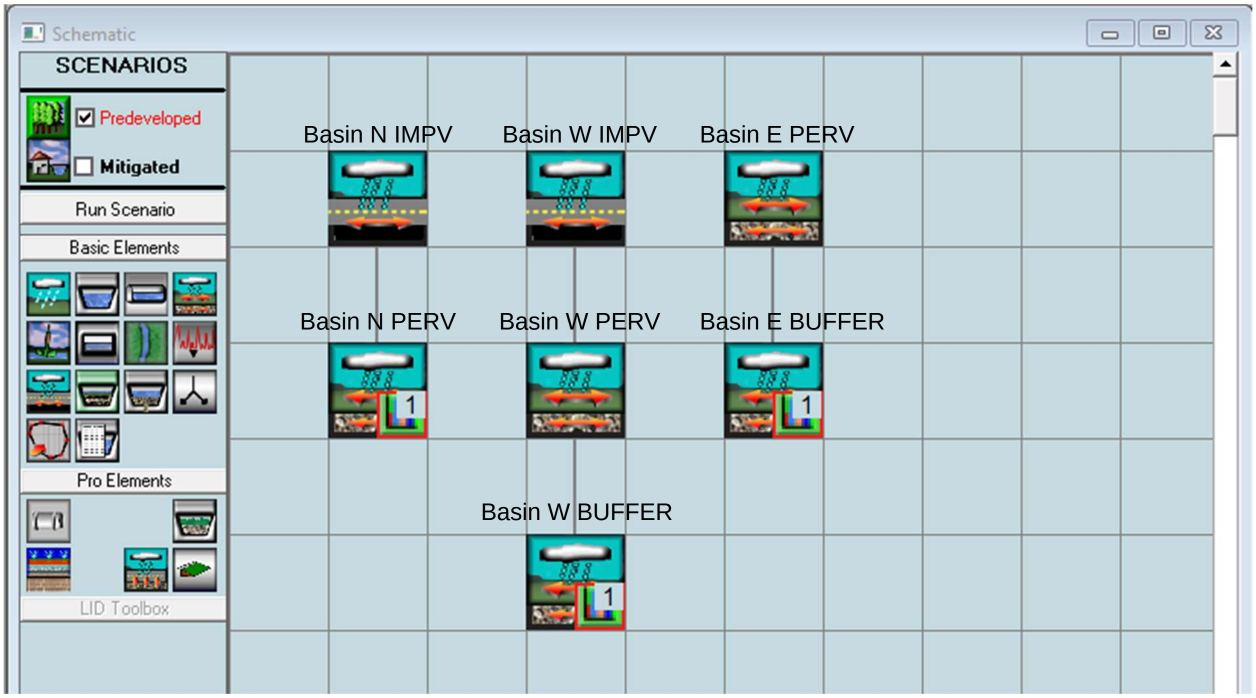
Basin	Description	Impervious (ac)	Pervious (ac)	Total (ac)
Basin W	Existing site buildings W of wetlands	2.5169	13.2913	**15.8082**
Basin N	Pasture and road N of 300th St	1.3725	23.7962	**25.1687**
Basin E	Project site and area E of wetlands	0.0000	2.7835	**2.7835**
Total	Total Wetland Basin	3.8895	39.8710	**43.7605**

Table 6: Existing Wetland Protection Buffer Areas

Basin	Description	Total (ac)
Basin W	Western boundary of wetlands	**2.7481**
Basin N	Runoff enters wetland through culvert	**0.0000**
Basin E	Eastern boundary of wetlands	**1.8303**

Existing impervious surfaces were modeled as impervious lateral flow basins which connect to pervious lateral flow basins. The most downstream pervious lateral flow basins represent the buffer areas. All surface flows, interflow, and groundwater flows from the buffer areas are connected to the POC. See Figure 14 for the predeveloped scenario schematic.

Figure 14: Wetland Protection Predeveloped Scenario Schematic



In the mitigated scenario, Basin E was replaced with the developed site basin areas found in Table 2. Basin W and Basin N remained the same as the predeveloped scenario. See Figure 15 for the mitigated scenario schematic. The detention system was sized to provide a level of flow attenuation for higher flows, while also maintaining minimum volumes to the wetland during periods of low rainfall. The facility volume is the largest possible without failing the wetland protection standard. See Figure 16 for the detention system configuration and see Figure 17 for the Wetland Protection Modeling results.

The flow control modeling results are shown on the following pages. These results are shown with and without the bypass basin included.

Figure 15: Mitigated Scenario Schematic

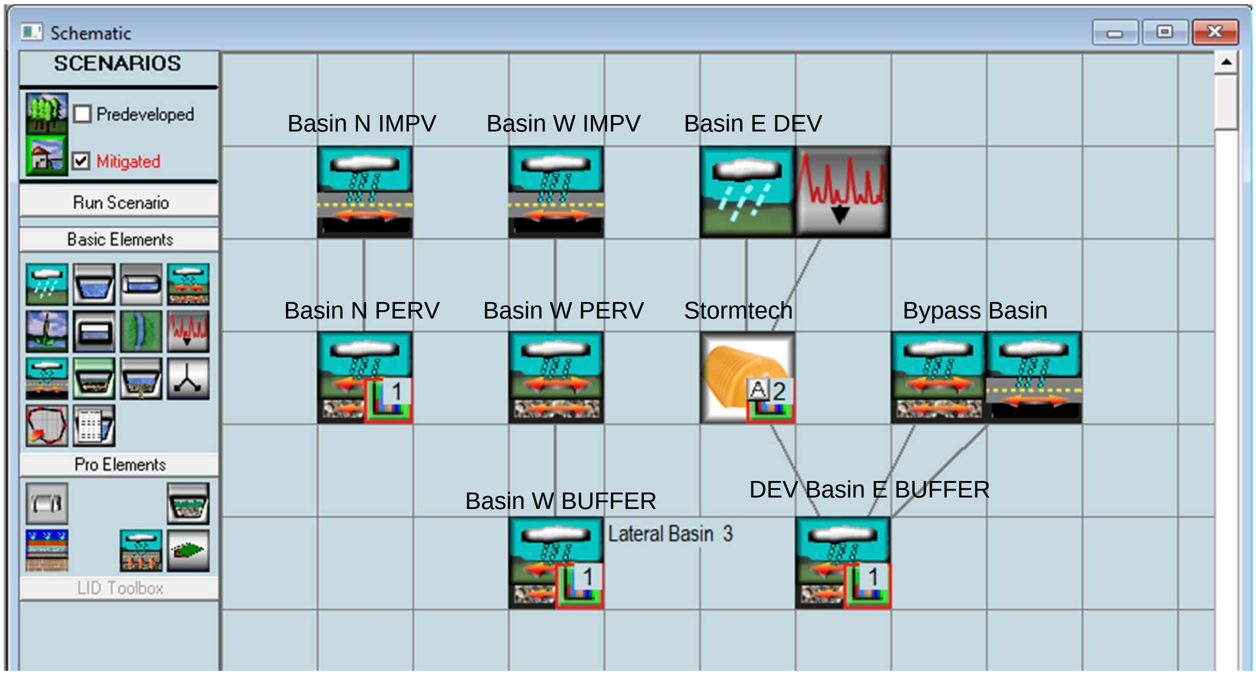
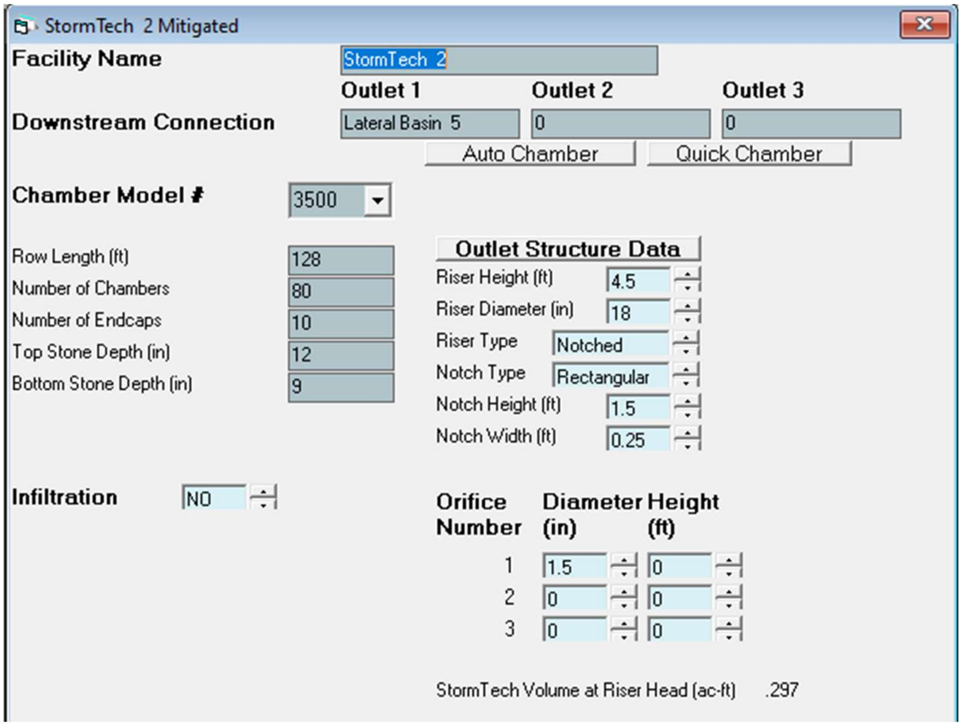


Figure 16: Detention System Configuration



Facility Name StormTech 2

Downstream Connection

	Outlet 1	Outlet 2	Outlet 3
Downstream Connection	Lateral Basin 5	0	0

Chamber Model # 3500

Row Length (ft) 128
Number of Chambers 80
Number of Endcaps 10
Top Stone Depth (in) 12
Bottom Stone Depth (in) 9

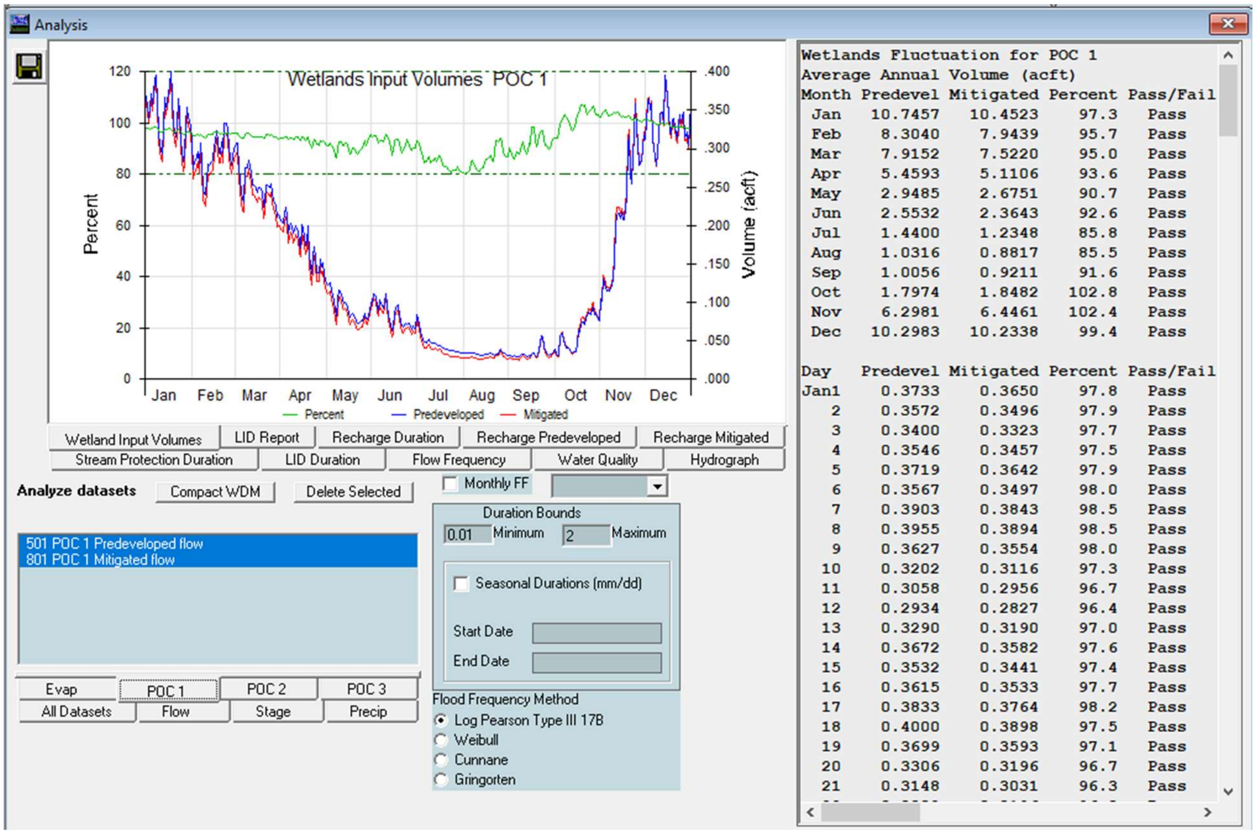
Infiltration NO

Riser Height (ft) 4.5
Riser Diameter (in) 18
Riser Type Notched
Notch Type Rectangular
Notch Height (ft) 1.5
Notch Width (ft) 0.25

Orifice Number	Diameter (in)	Height (ft)
1	1.5	0
2	0	0
3	0	0

StormTech Volume at Riser Head (ac-ft) .297

Figure 17: Wetland Protection Modeling Results



☐ Monthly FF

501 POC 1 Predeveloped flow
801 POC 1 Mitigated flow

Duration Bounds
0.01 Minimum 2 Maximum

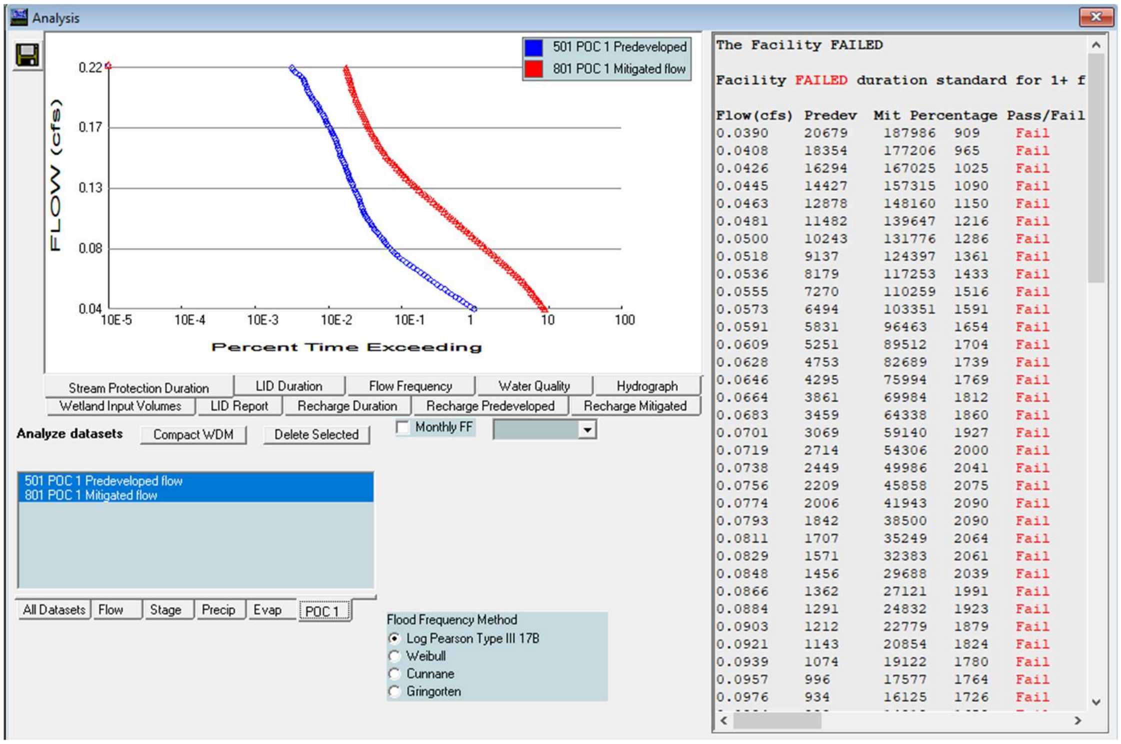
☐ Seasonal Durations (mm/dd)
Start Date
End Date

Flood Frequency Method
● Log Pearson Type III 17B
○ Weibull
○ Cunnane
○ Gringorten

```
Wetlands Fluctuation for POC 1
Average Annual Volume (acft)
Month Predevel Mitigated Percent Pass/Fail
 Jan   10.7457   10.4523    97.3    Pass
 Feb    8.3040    7.9439    95.7    Pass
 Mar    7.9152    7.5220    95.0    Pass
 Apr    5.4593    5.1106    93.6    Pass
 May    2.9485    2.6751    90.7    Pass
 Jun    2.5532    2.3643    92.6    Pass
 Jul    1.4400    1.2348    85.8    Pass
 Aug    1.0316    0.8817    85.5    Pass
 Sep    1.0056    0.9211    91.6    Pass
 Oct    1.7974    1.8482   102.8    Pass
 Nov    6.2981    6.4461   102.4    Pass
 Dec   10.2983   10.2338    99.4    Pass

Day   Predevel Mitigated Percent Pass/Fail
Jan1    0.3733    0.3650    97.8    Pass
   2    0.3572    0.3496    97.9    Pass
   3    0.3400    0.3323    97.7    Pass
   4    0.3546    0.3457    97.5    Pass
   5    0.3719    0.3642    97.9    Pass
   6    0.3567    0.3497    98.0    Pass
   7    0.3903    0.3843    98.5    Pass
   8    0.3955    0.3894    98.5    Pass
   9    0.3627    0.3554    98.0    Pass
  10    0.3202    0.3116    97.3    Pass
  11    0.3058    0.2956    96.7    Pass
  12    0.2934    0.2827    96.4    Pass
  13    0.3290    0.3190    97.0    Pass
  14    0.3672    0.3582    97.6    Pass
  15    0.3532    0.3441    97.4    Pass
  16    0.3615    0.3533    97.7    Pass
  17    0.3833    0.3764    98.2    Pass
  18    0.4000    0.3898    97.5    Pass
  19    0.3699    0.3593    97.1    Pass
  20    0.3306    0.3196    96.7    Pass
  21    0.3148    0.3031    96.3    Pass
```

Figure 18: Flow Control Duration Results

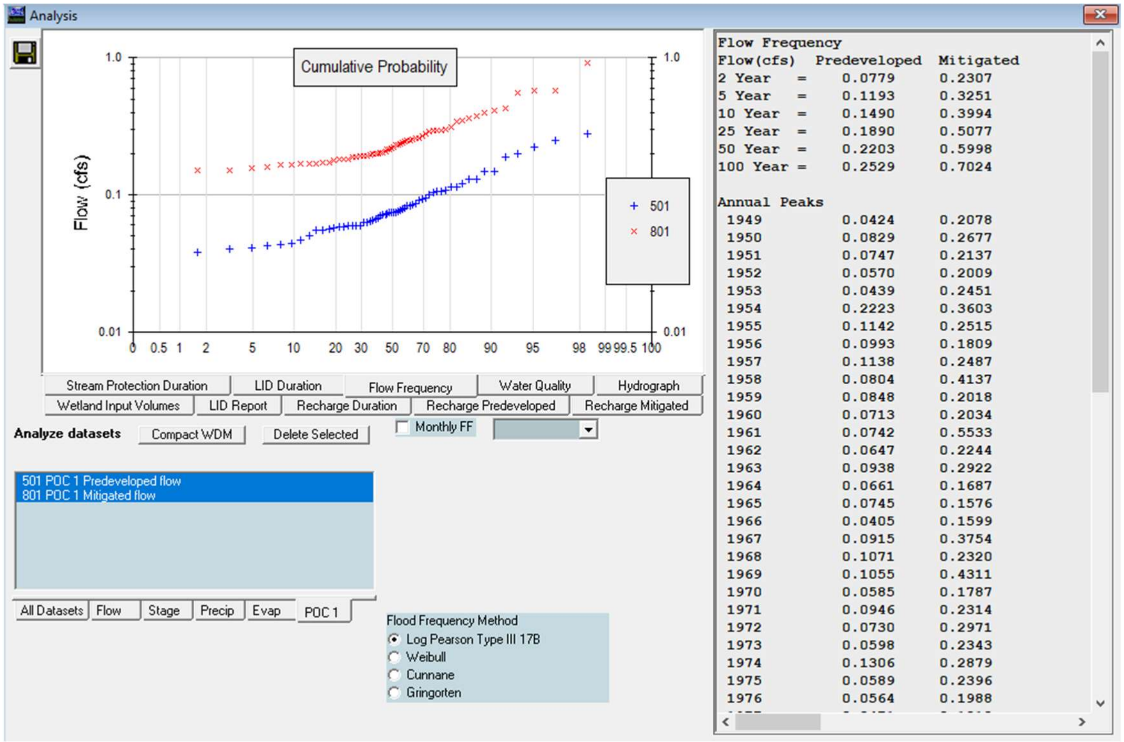


```
The Facility FAILED

Facility FAILED duration standard for 1+ f

Flow(cfs) Predev  Mit Percentage Pass/Fail
0.0390    20679    187986  909      Fail
0.0408    18354    177206  965      Fail
0.0426    16294    167025  1025     Fail
0.0445    14427    157315  1090     Fail
0.0463    12878    148160  1150     Fail
0.0481    11482    139647  1216     Fail
0.0500    10243    131776  1286     Fail
0.0518    9137     124397  1361     Fail
0.0536    8179     117253  1433     Fail
0.0555    7270     110259  1516     Fail
0.0573    6494     103351  1591     Fail
0.0591    5831     96463   1654     Fail
0.0609    5251     89512   1704     Fail
0.0628    4753     82689   1739     Fail
0.0646    4295     75994   1769     Fail
0.0664    3861     69984   1812     Fail
0.0683    3459     64338   1860     Fail
0.0701    3069     59140   1927     Fail
0.0719    2714     54306   2000     Fail
0.0738    2449     49986   2041     Fail
0.0756    2209     45858   2075     Fail
0.0774    2006     41943   2090     Fail
0.0793    1842     38500   2090     Fail
0.0811    1707     35249   2064     Fail
0.0829    1571     32383   2061     Fail
0.0848    1456     29688   2039     Fail
0.0866    1362     27121   1991     Fail
0.0884    1291     24832   1923     Fail
0.0903    1212     22779   1879     Fail
0.0921    1143     20854   1824     Fail
0.0939    1074     19122   1780     Fail
0.0957    996      17577   1764     Fail
0.0976    934      16125   1726     Fail
```

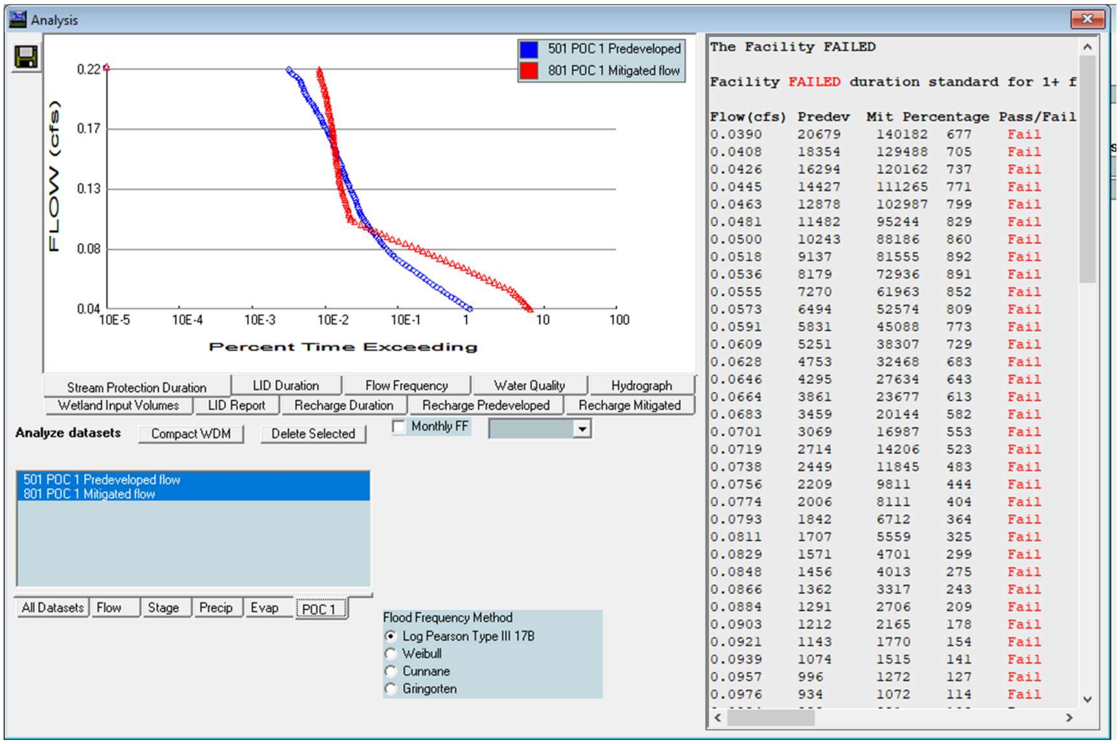
Figure 19: Flow Frequency Results



```
Flow Frequency
Flow(cfs)  Predeveloped  Mitigated
2 Year   =     0.0779     0.2307
5 Year   =     0.1193     0.3251
10 Year  =     0.1490     0.3994
25 Year  =     0.1890     0.5077
50 Year  =     0.2203     0.5998
100 Year =     0.2529     0.7024

Annual Peaks
 1949          0.0424     0.2078
 1950          0.0829     0.2677
 1951          0.0747     0.2137
 1952          0.0570     0.2009
 1953          0.0439     0.2451
 1954          0.2223     0.3603
 1955          0.1142     0.2515
 1956          0.0993     0.1809
 1957          0.1138     0.2487
 1958          0.0804     0.4137
 1959          0.0848     0.2018
 1960          0.0713     0.2034
 1961          0.0742     0.5533
 1962          0.0647     0.2244
 1963          0.0938     0.2922
 1964          0.0661     0.1687
 1965          0.0745     0.1576
 1966          0.0405     0.1599
 1967          0.0915     0.3754
 1968          0.1071     0.2320
 1969          0.1055     0.4311
 1970          0.0585     0.1787
 1971          0.0946     0.2314
 1972          0.0730     0.2971
 1973          0.0598     0.2343
 1974          0.1306     0.2879
 1975          0.0589     0.2396
 1976          0.0564     0.1988
```

Figure 20: Flow Control Duration Results – Without Bypass



☐ Monthly FF

501 POC 1 Predeveloped flow
801 POC 1 Mitigated flow

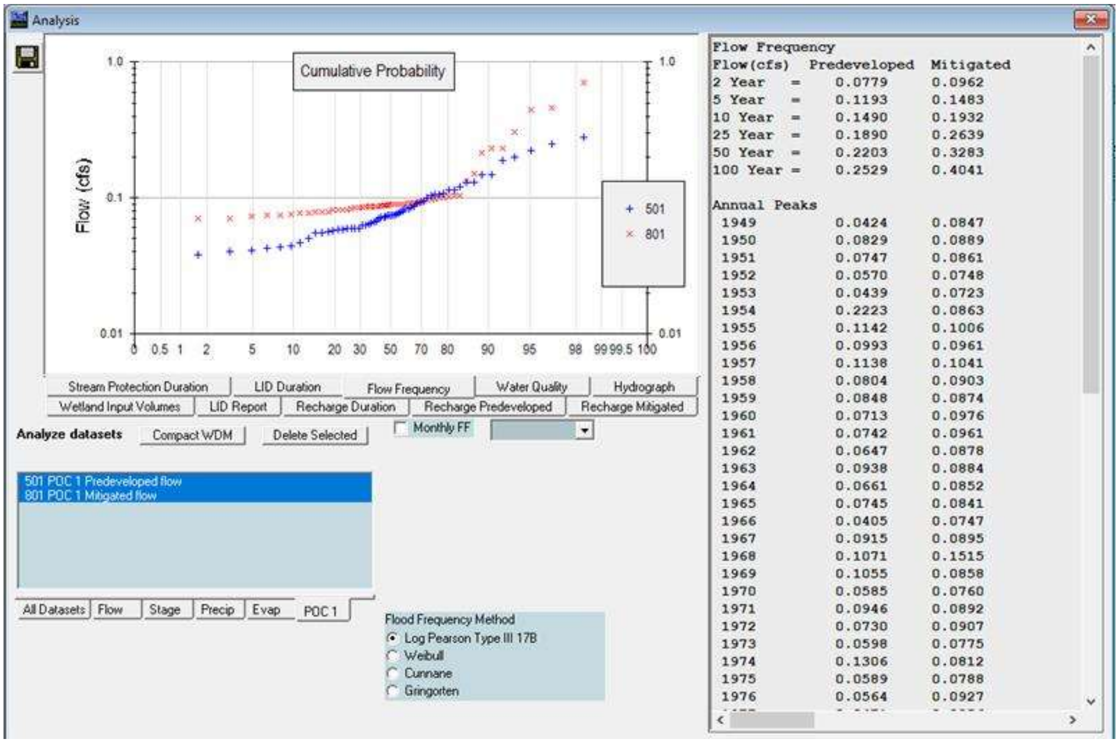
Flood Frequency Method
● Log Pearson Type III 17B
○ Weibull
○ Cunnane
○ Gringorten

```
The Facility FAILED

Facility FAILED duration standard for 1+ f

Flow(cfs) Predev  Mit Percentage Pass/Fail
0.0390    20679    140182  677     Fail
0.0408    18354    129488  705     Fail
0.0426    16294    120162  737     Fail
0.0445    14427    111265  771     Fail
0.0463    12878    102987  799     Fail
0.0481    11482    95244   829     Fail
0.0500    10243    88186   860     Fail
0.0518    9137     81555   892     Fail
0.0536    8179     72936   891     Fail
0.0555    7270     61963   852     Fail
0.0573    6494     52574   809     Fail
0.0591    5831     45088   773     Fail
0.0609    5251     38307   729     Fail
0.0628    4753     32468   683     Fail
0.0646    4295     27634   643     Fail
0.0664    3861     23677   613     Fail
0.0683    3459     20144   582     Fail
0.0701    3069     16987   553     Fail
0.0719    2714     14206   523     Fail
0.0738    2449     11845   483     Fail
0.0756    2209     9811    444     Fail
0.0774    2006     8111    404     Fail
0.0793    1842     6712    364     Fail
0.0811    1707     5559    325     Fail
0.0829    1571     4701    299     Fail
0.0848    1456     4013    275     Fail
0.0866    1362     3317    243     Fail
0.0884    1291     2706    209     Fail
0.0903    1212     2165    178     Fail
0.0921    1143     1770    154     Fail
0.0939    1074     1515    141     Fail
0.0957    996      1272    127     Fail
0.0976    934      1072    114     Fail
```

Figure 21: Flow Frequency Results – Without Bypass



☐ Monthly FF

501 POC 1 Predeveloped flow
801 POC 1 Mitigated flow

Flood Frequency Method
● Log Pearson Type III 17B
○ Weibull
○ Cunnane
○ Gringorten

```
Flow Frequency
Flow(cfs)  Predeveloped  Mitigated
2 Year   =    0.0779     0.0962
5 Year   =    0.1193     0.1483
10 Year  =    0.1490     0.1932
25 Year  =    0.1890     0.2639
50 Year  =    0.2203     0.3283
100 Year =    0.2529     0.4041

Annual Peaks
 1949         0.0424     0.0847
 1950         0.0829     0.0889
 1951         0.0747     0.0861
 1952         0.0570     0.0748
 1953         0.0439     0.0723
 1954         0.2223     0.0863
 1955         0.1142     0.1006
 1956         0.0993     0.0961
 1957         0.1138     0.1041
 1958         0.0804     0.0903
 1959         0.0848     0.0874
 1960         0.0713     0.0976
 1961         0.0742     0.0961
 1962         0.0647     0.0878
 1963         0.0938     0.0884
 1964         0.0661     0.0852
 1965         0.0745     0.0841
 1966         0.0405     0.0747
 1967         0.0915     0.0895
 1968         0.1071     0.1515
 1969         0.1055     0.0858
 1970         0.0585     0.0760
 1971         0.0946     0.0892
 1972         0.0730     0.0907
 1973         0.0598     0.0775
 1974         0.1306     0.0812
 1975         0.0589     0.0788
 1976         0.0564     0.0927
```

CHAPTER 5 – DISCUSSION OF MINIMUM REQUIREMENTS

Minimum Requirement #1: Preparation of Stormwater Stie Plans

This report satisfies Minimum Requirement #1.

Minimum Requirement #2: Stormwater Pollution Prevention Plans (SWPPPs)

The construction SWPPP is contained under a separate cover.

Minimum Requirement #3: Source Control of Pollution

The primary pollution source for stormwater runoff leaving the site is anticipated to be sedimentation during construction. The site does not have pollutants of concern. Construction source control BMPs will be applied according to the SWPPP to prevent sedimentation during construction. Project includes a generator with fuel tank. Fuel tank will include 100% secondary containment and leak detection in accordance with Snohomish County requirements.

Minimum Requirement #4: Preservation of Natural Drainage Systems and Outfalls

The natural drainage patterns experienced on site will be maintained to the maximum extent feasible by maintaining discharge to the wetland. Outfalls will be designed to prevent adverse impacts to the downstream receiving waters.

Minimum Requirement #5: On-Site Stormwater Management

After the BLA, this project will be located on a project site less than 5 acres and is outside the UGA. The project has selected to employ on-site stormwater BMPs in accordance with List #2 to the maximum extent feasible.

Figure 22: OSM Requirements

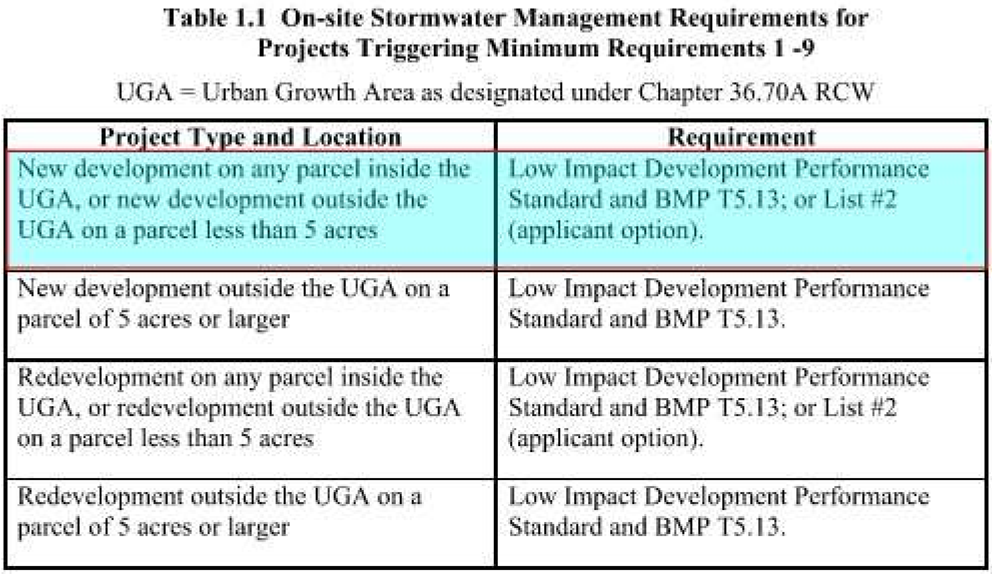


Table 1.1 On-site Stormwater Management Requirements for Projects Triggering Minimum Requirements 1 -9

UGA = Urban Growth Area as designated under Chapter 36.70A RCW

Project Type and Location	Requirement
New development on any parcel inside the UGA, or new development outside the UGA on a parcel less than 5 acres	Low Impact Development Performance Standard and BMP T5.13; or List #2 (applicant option).
New development outside the UGA on a parcel of 5 acres or larger	Low Impact Development Performance Standard and BMP T5.13.
Redevelopment on any parcel inside the UGA, or redevelopment outside the UGA on a parcel less than 5 acres	Low Impact Development Performance Standard and BMP T5.13; or List #2 (applicant option).
Redevelopment outside the UGA on a parcel of 5 acres or larger	Low Impact Development Performance Standard and BMP T5.13.

Lawn and landscaped areas:

1. Post-Construction Soil Quality and Depth in accordance with BMP T5.13 will be applied to all lawn and landscaped areas.

Roofs:

1. Full Dispersion in accordance with BMP T5.30 is infeasible due to less than 65% of the site being retained in a native condition.
2. Downspout Full Infiltration Systems in accordance with BMP T5.10A are infeasible due to the presence of low permeability layers.
3. Bioretention is infeasible for roofs due to the geotechnical recommendation that infiltration not be used on this site. The geotechnical report is included in Appendix A.
4. Roof drainage will be collected and detained prior to being released to a grassed swale dispersion system (EDDS DWG 5-070) and will meet the flow length criteria in accordance with BMP T5.10B. This system was chosen due to the topographic constraints and presence of wetlands on the site to minimize site disturbance.

Other Hard Surfaces:

1. Full Dispersion in accordance with BMP T5.30 is infeasible due to less than 65% of the site being retained in a native condition.
2. Permeable pavement in accordance with BMP T5.15 is infeasible due to the geotechnical recommendation that infiltration is not used on this site. The geotechnical report is included in Appendix A. Most areas for permeable pavements will be located on fill soils, will need to support fire truck and outrigger loading and are adjacent to slopes over 20%. Any infiltration provided by underlying soils would likely move laterally and potentially compromise the proposed building foundations. Permeable pavement was considered in the drainage design but was ultimately not chosen.
3. Bioretention is infeasible due to the geotechnical recommendation that infiltration is not used on this site. The geotechnical report is included in Appendix A. Although bioretention is infeasible and will not be used to meet MR5, it will be used to help meet the water quality requirements of MR6.
4. Sheet Flow Dispersion in accordance with BMP T5.12 and Concentrated Flow Dispersion in accordance with BMP T5.11 will be applied to the maximum extent feasible where runoff from other hard surfaces is not being collected to meet the intention of MR7 and requirements of MR8.

Minimum Requirement #6: Runoff Treatment

Stormwater treatment facilities will be provided because the total of pollution-generating hard surface (PGHS) in the TDA is greater than 5,000 square feet. Infiltration for pollutant removal is not practical. Oil and phosphorus control facilities are not required. Enhanced treatment is required due to discharge to non-exempt fresh water. Bioretention with an underdrain and BioClean Modular Wetland Systems (MWS) will be used to provide enhanced treatment.

Minimum Requirement #7: Flow Control

Infiltration for flow control is considered infeasible for this project. Stormwater flow control facilities were sized to match the fully forested pre-developed condition. However, this project is also subject to MR8 Wetlands Protection and the flow control facilities meeting MR7 failed to release sufficient volumes during summer months to meet MR8. A model showing the passing flow control model fails the wetland protection requirement will be provided in a later version of this report.

Minimum Requirement #8: Wetlands Protection

There are two existing regulatory wetlands on this site. The north wetland is a Category III wetland with five habitat points. The south wetland is Category III with six habitat points. The north wetland flows to the south wetland through a culvert. The project will provide both wetlands with General Protection and Protection from Pollutants. Method 2 will be used to provide Wetland Hydroperiod Protection for the south wetland.

Minimum Requirement #9: Operations and Maintenance

The Operations and Maintenance Manual will be provided with submittal for the Land Disturbing Activities Permit.

CHAPTER 6 - SPECIAL REPORTS AND STUDIES

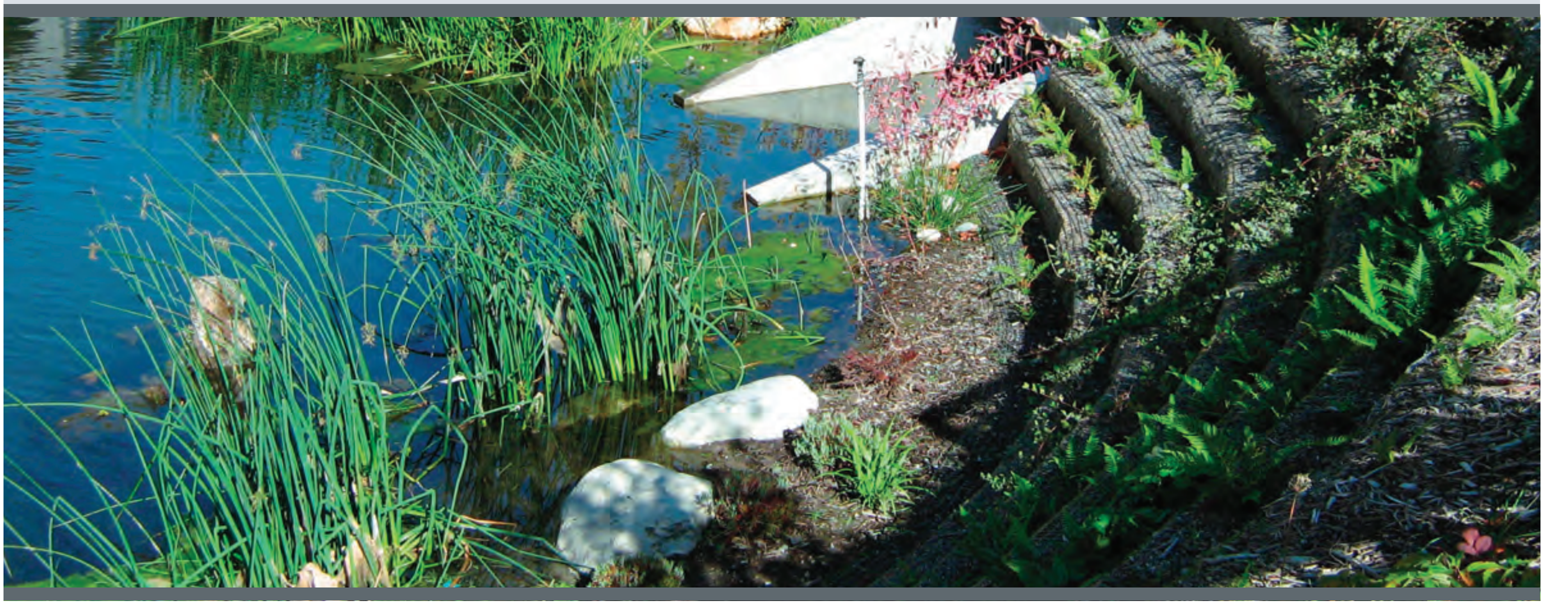
The following reports have been completed for the project site:

- *Subsurface Exploration, Geologic Hazard, Preliminary Geotechnical Engineering and Stormwater Infiltration Feasibility Report* by Associated Earth Sciences, Inc. dated December 9, 2021.
- *Wetland and Fish and Wildlife Habitat Assessment Report* by Soundview Consultants, LLC dated January 26, 2022
- *Traffic Impact Analysis* by Heath & Associates, Inc dated June 23, 2022.

Appendices

APPENDIX A

GEOTECHNICAL REPORT



Subsurface Exploration, Geologic Hazard, Preliminary Geotechnical Engineering, and Stormwater Infiltration Feasibility Report

STANWOOD RESIDENTIAL TREATMENT FACILITY

Stanwood, Washington

Prepared for:

BCRA

Project No. 20210354E001
December 9, 2021

Associated Earth Sciences, Inc.
911 5th Avenue
Kirkland, WA 98033
P (425) 827 7701

associated earth sciences incorporated

December 9, 2021
Project No. 20210354E001

BCRA
2106 Pacific Avenue, Suite 300
Tacoma, Washington 98402

Attention: Mr. Zachary Crum

Subject: Subsurface Exploration, Geologic Hazard,
Preliminary Geotechnical Engineering,
and Stormwater Infiltration Feasibility Report
Stanwood Residential Treatment Facility
29919 80th Avenue NW
Snohomish County, Washington

Dear Mr. Crum:

We are pleased to present this copy of the referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering studies, and offers recommendations for the design and development of the proposed project. The project is in conceptual design at the time this report is written. We recommend that we be allowed to review the recommendations in this report and revise them as needed when the project design nears completion.

We have enjoyed working with you on this study and are confident the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Kurt D. Merriman, P.E.
Senior Principal Geotechnical Engineer

KDM/jh – 20210354E001-003

SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, PRELIMINARY GEOTECHNICAL ENGINEERING, AND STORMWATER INFILTRATION FEASIBILITY REPORT

STANWOOD RESIDENTIAL TREATMENT FACILITY

Stanwood, Washington

Prepared for:
BCRA
2106 Pacific Avenue, Suite 300
Tacoma, Washington 98402

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue
Kirkland, Washington 98033
425-827-7701

December 9, 2021
Project No. 20210354E001

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of Associated Earth Sciences, Inc.'s (AESI's) subsurface exploration, geologic hazard, preliminary geotechnical engineering, and stormwater infiltration feasibility study for the subject project. The location of the subject site is shown on the "Vicinity Map," Figure 1. The approximate locations of the explorations accomplished for this study are presented on the "Site and Exploration Plan," Figure 2. Because the project is early in the design phase, we recommend that we be allowed to review and update the recommendations in this report when the project design is nearing completion.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the design and development of the subject project. Our study included reviewing available geologic literature, completing five exploration borings, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow groundwater. Geotechnical engineering studies were completed to assess seismic hazards, the type of suitable foundations, anticipated settlements, floor support recommendations, stormwater infiltration feasibility, and drainage considerations. This report summarizes our current fieldwork and offers preliminary development recommendations based on our present understanding of the project.

1.2 Authorization

Authorization to proceed with this study was granted by BCRA in the form of a signed agreement of services dated November 3, 2021. This report has been prepared for the exclusive use of BCRA and its agents for specific application to this project. Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made.

2.0 SITE AND PROJECT DESCRIPTION

The subject site consists of two tax parcels (Snohomish County Parcel Numbers: 32041800100100 and 32041800101400) totaling approximately 30.2 acres. Topography across the western half of the site slopes gently down towards the east before rising towards the northeast with an overall vertical relief of approximately 30 feet. The site is vegetated with open grassy fields and pastures containing a few scattered trees and landscaping near the developed areas of the site. The subject site is developed with single-family residences and various agricultural outbuildings and stables within the western portion of the two parcels.

Based on preliminary project information, the site does not appear to include slopes that meet criteria for treatment as a Landslide Hazard Area in accordance with the *Snohomish County Code* (SCC). It should be noted that SCC requires that slope areas meet multiple criteria for treatment as a Landslide Hazard Area, and an area that has only steep geometry without other contributing factors is not considered by the County to be a Landslide Hazard Area.

Based on discussions with BCRA, we understand that project plans consist of the construction of a single-story 32-bed residential treatment facility with roads, utilities, a stormwater facility, and other typical improvements. We understand that stormwater infiltration is being considered, if feasible. The project will rely on water from either existing on-site water supply wells or a new groundwater well.

3.0 SUBSURFACE EXPLORATION

Our field study included advancing five exploration borings to gain subsurface information about the site. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in Appendix A. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field. Our explorations were approximately located in the field relative to known site features. The approximate locations of the exploration borings are shown on Figure 2.

The conclusions and recommendations presented in this report are based, in part, on the exploration borings completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions beyond the field explorations is necessary. It should be noted that subsurface conditions differing from those depicted on the logs may be present at the site due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of variations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Borings

For this study, five hollow-stem auger exploration borings were performed by Advanced Drill Technologies, an independent firm working under subcontract to AESI, at the approximate locations shown on Figure 2. Logs of exploration borings, labeled EB-1 through EB-5, are included with this report. The borings were completed by advancing a 2.25-inch inside-diameter, hollow-stem auger with a track-mounted drill rig. During the drilling process, samples were obtained at 2.5- to 5-foot-depth intervals. After completion of drilling, each borehole was backfilled with bentonite chips.

Disturbed but representative samples were obtained by using the Standard Penetration Test (SPT) procedure. This test and sampling method consists of driving a 2-inch outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded, and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance ("N") or blow count. If a total of 50 is recorded within one 6-inch interval, the blow count is recorded as the number of blows for the corresponding number of inches of penetration. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils; these values are plotted on the attached exploration boring logs.

The exploration borings were continuously observed and logged by a geologist from our firm. The samples obtained from the split-barrel sampler were classified in the field and representative portions placed in watertight containers. The samples were then transported to our laboratory for further visual classification. The exploration logs presented in Appendix A are based on the N-values, field observations, and drilling action.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of applicable geologic literature. The native sediments encountered in our explorations generally consisted of Vashon lodgement till, Vashon advance outwash and pre-Olympia glaciomarine deposits. Native soils were capped by existing fill in EB-2 (fill depth 6 feet) and EB-3 (fill depth 3 feet). The following section presents more detailed subsurface information organized from the shallowest (youngest) to the deepest (oldest) sediment types.

4.1 Stratigraphy

Fill/Topsoil

Surficial topsoil was encountered at the surface within each of the borings and extended to approximately 6 inches below ground surface. The topsoil encountered generally consisted of loose, moist, dark brown, silty fine sand with trace gravel and abundant roots and organics.

Surficial fill was encountered within EB-2 and EB-3 and extended to approximately 6 and 3 feet, respectively, below existing grade. Fill soils encountered generally consisted of loose to medium dense, moist to very moist, dark brown to brown, silty, fine sand to sandy silt with trace gravel. Fill soils are also anticipated to be present in unexplored areas around existing utilities, behind walls or rockeries, and around the foundation of existing structures. Due to its variability, existing fill is not suitable for structural support and warrants remedial preparation below new hardscapes or paving.

Vashon Lodgement Till

Sediments encountered directly below the surficial fill or topsoil in each boring generally consisted of dense to very dense, unsorted, brown to gray, silty, fine sand with some to trace gravel with occasional zones of somewhat cleaner sand. We interpret these sediments to be representative of Vashon lodgement till. This unit included a medium dense weathered zone approximately 3 feet in thickness below the topsoil in EB-4. Vashon lodgement till was deposited directly from basal, debris-laden glacial ice during the Vashon Stade of the Fraser Glaciation, approximately 12,500 to 15,000 years ago. The high relative density characteristic of the Vashon lodgement till is due to its consolidation by the massive weight of the glacial ice from which it was deposited. Lodgement till typically contains a significant fine-grained fraction and is highly sensitive to moisture during placement in structural fill applications. In borings EB-1, EB-2, and EB-3, the lodgement till extended beyond the maximum depths explored of approximately 20.5 feet. In EB-4 and EB-5, the lodgement till extended to approximately 12.5 feet and 22.5 feet, respectively, before encountering Vashon advance outwash and pre-Olympia-age glaciomarine sediments.

Vashon Advance Outwash

Sediments encountered directly below the Vashon lodgement till in EB-4 generally consisted of dense to very dense, bedded sand with silty laminations and variable gravel content. We interpret these sediments to be Vashon advance outwash. Vashon advance outwash consists of sediments that were deposited by meltwater streams that emanated from the advancing glacial ice during the Vashon Stade of the Fraser Glaciation approximately 12,500 to 15,000 years ago. The high relative density characteristic of the Vashon advance outwash is due to its consolidation by the glacial ice that overrode these sediments subsequent to their deposition. The advance outwash was texturally stratified, with interbedded sand/silt layers. At the location of EB-4, the advance outwash extended beyond the maximum depth explored of 26.5 feet.

Where comprised of relatively permeable sand, the Vashon advance outwash deposits can be suitable for infiltration of stormwater if they are unsaturated and laterally and vertically extensive. At this site, the Vashon advance outwash contained numerous thin fine-grained layers. Stormwater infiltration feasibility is discussed in more detail later in this report.

Pre-Olympia Glaciomarine Sediments

Underlying the Vashon lodgement till in EB-5 at a depth of approximately 22.5 feet our explorations encountered gray, very dense, silty, fine sand and gray, hard, sandy silt to the full depths explored of 51 feet. When exposed to hydrochloric acid samples of these sediments reacted to form gas bubbles, which may indicate the presence of calcium carbonate associated with marine deposition. We interpret these sediments as pre-Olympia glaciomarine sediments, deposited in a glacial marine environment and overridden by glacial ice during subsequent glaciation. These sediments were deposited prior to the Olympia nonglacial interval that occurred from 15,000 to 60,000 years before present and have been consolidated by at least one glaciation.

4.2 Regional Geologic and Soils Mapping

Review of the regional geologic map titled *Geologic Map of the Utsalady and Conway 7.5-minute Quadrangles, Skagit, Snohomish, and Island Counties, Washington* by J.D. Dragovich, et al., 2002 indicates that the area of the subject site is underlain by Vashon lodgement till, Vashon advance outwash deposits, and glaciomarine drift. Our interpretation of the sediments encountered at the subject site is generally consistent with the regional map in that we encountered Vashon lodgement till, Vashon advance outwash, and pre-Olympia glaciomarine drift.

A review of regional soils mapping (U.S. Department of Agriculture [USDA] *Web Soil Survey*) indicates that the subject site is underlain by Tokul gravelly medial loam. Tokul soil is commonly derived from the weathering of glacial till, which is consistent with the lodgement till observed at shallow depths in our borings.

4.3 Hydrology

Groundwater was noted at a depth of 17 feet in EB-2 and shallow perched groundwater was noted in EB-1 and EB-5. The seepage in EB-2 was within the very dense silty Vashon lodgement till, likely along a sandy seam, as the unit generally has very low permeability. The shallow perched groundwater observed in EB-1 and EB-5 is expected to occur seasonally as interflow above the dense Vashon lodgement till. Perched groundwater commonly occurs when surface water infiltrates down through relatively permeable soils, such as the topsoil/fill and becomes trapped or “perched” atop a comparatively impermeable barrier, such as the Vashon lodgement till sediments. The perched groundwater may then flow laterally as interflow. We observed varying amounts of weathering and oxidation within the Vashon lodgement till near the contact with the topsoil/fill consistent with intermittent or seasonal groundwater. Perched surface water was observed on the site, accumulating in the lowland areas. We observed moderate surface drainage running northwest to southeast and terminating in a small pond during our visit.

Groundwater was not present in the Vashon advance outwash at the time of drilling but the sediments ranged to very moist to wet above fine-grained interbeds. Groundwater is expected in the Vashon advance outwash perched above the low-permeability pre-Olympia glaciomarine deposits.

Groundwater conditions, including depth, duration, and quantity of seepage, should be expected to vary seasonally, and in response to changes in precipitation, soil grain-size distribution, topography, on- and off-site land usage, and other factors. Explorations for this study were completed on November 4th and 5th, 2021. Recommendations relating to groundwater can be found below in the “Drainage Considerations” section of this report.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and shallow groundwater conditions as observed and discussed herein.

5.0 LANDSLIDE HAZARDS AND MITIGATION

Given the gently sloping topography of the subject site, it is our opinion that the risk of damage to the proposed development by landsliding is low under both static and seismic conditions. The topography of the site is gently sloping from the west and northeast with approximately 30 feet of vertical relief across the length of the property and maximum slope inclinations on the site are less than 15 percent. No indications of historic landslide activity were observed in our explorations or during our reconnaissance of the site. Based on our observations made in the field and review of the topographic survey shown on Figure 2, the site does not meet the conditions to classify as a landslide hazard area as defined by SCC Section 30.91L.040. No quantitative slope stability analysis was completed as part of this study and none is warranted based on our understanding of the project.

6.0 SEISMIC HAZARDS AND MITIGATION

The following discussion is a general assessment of seismic hazards that is intended to be useful to the project design team in terms of understanding regional seismic risks, and to the structural engineer for site-specific design.

All of Western Washington is at risk of strong seismic events resulting from movement of the tectonic plates associated with the Cascadia Subduction Zone (CSZ), where the offshore Juan de Fuca plate subducts beneath the continental North American plate. The site lies within a zone of strong potential shaking from subduction zone earthquakes associated with the CSZ. The CSZ can produce earthquakes up to magnitude 9.0, and the recurrence interval is estimated to be on the order of 500 years. Geologists infer the most recent subduction zone earthquake occurred in 1700 (Goldfinger et al., 2012[1]). Three main types of earthquakes are typically associated with subduction zone environments: crustal, intraplate, and interplate earthquakes. Seismic records in the Puget Sound region document a distinct zone of shallow crustal seismicity (e.g., the Seattle

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[1] Goldfinger, C., Nelson, C.H., Morey, A.E., Johnson, J.E., Patton, J.R., Karabanov, E., Gutierrez-Pastor, J., Eriksson, A.T., Gracia, E., Dunhill, G., Enkin, R.J, Dallimore, A., and Vallier, T.,2012, *Turbidite Event History—Methods and Implications for Holocene Paleoseismicity of the Cascadia Subduction Zone*: U.S. Geological Survey Professional Paper 1661–F, 170.

Fault Zone). These shallow fault zones may include surficial expressions of previous seismic events, such as fault scarps, displaced shorelines, and shallow bedrock exposures. The shallow fault zones typically extend from the surface to depths ranging from 16 to 19 miles. A deeper zone of seismicity is associated with the subducting Juan de Fuca plate. Subduction zone seismic events produce intraplate earthquakes at depths ranging from 25 to 45 miles beneath the Puget Lowland including the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event) and interplate earthquakes at shallow depths near the Washington coast including the 1700 earthquake, which had a magnitude of approximately 9.0. The 1949 earthquake appears to have been the largest in this region during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates indicates that an earthquake of the magnitude between 5.5 and 6.0 is likely within a given 20-year period.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides or lateral spreading, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

6.1 Surficial Ground Rupture

Generally, the largest earthquakes that have occurred in the Puget Sound area are sub-crustal events with epicenters ranging from 25 to 45 miles in depth. Earthquakes that are generated at such depths usually do not result in fault rupture at the ground surface. The site is located in the vicinity of the Devils Mountain Fault Zone (DMFZ). Strands of the DMFZ including the Utsalady Point Fault and the Strawberry Point Fault are mapped west of the site and projecting toward the site but fault mapping stops at a point west of the project. If mapped faults are projected approximately 4 miles eastward from the point where mapping was discontinued, it appears that the site is approximately 2 miles from the nearest known and projected fault trace. Due to the absence of known or mapped faults at or close to the project the risk of damage due to fault surface rupture are low in our opinion.

6.2 Seismically Induced Landslides

As mentioned above in the “Landslide Hazards and Mitigation” section, the site is gently sloping and does not classify as a landslide hazard area under the SCC. Quantitative assessment of static and seismic slope failures is not warranted for the project in our opinion and was not completed as part of this study.

6.3 Liquefaction

Liquefaction is a process through which unconsolidated soil loses strength as a result of vibrations, such as those which occur during a seismic event. During normal conditions, the weight of the soil is supported by both grain-to-grain contacts and by the fluid pressure within the pore spaces of the soil below the water table. Extreme vibratory shaking can disrupt the grain-to-grain contact, increase the pore pressure, and result in a temporary decrease in soil shear strength. The soil is said to be liquefied when nearly all of the weight of the soil is supported by pore pressure alone. Liquefaction can result in deformation of the sediment and settlement of overlying structures. Areas most susceptible to liquefaction include those areas underlain by non-cohesive silt and sand with low relative densities, accompanied by a shallow water table.

Only minor groundwater seepage was observed within the native glacially consolidated sediments. Based on the lack of adverse groundwater conditions and the relatively high density of the native sediments encountered in our explorations at depth, it is our opinion that the risk of settlement due to liquefaction is low. No quantitative liquefaction assessment was completed as part of our study, and none is warranted in our opinion based on our subsurface explorations.

6.4 Ground Motion

Structural design should follow 2018 *International Building Code* (IBC) standards using Site Class "C" as defined in Table 20.3-1 of *American Society of Civil Engineers* (ASCE) *7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*.

7.0 EROSION HAZARDS AND MITIGATION

The site does not qualify as an erosion hazard area as defined by SCC 30.91E.158 as the erosion hazard in areas where construction or disturbance of soils will take place is classified as "slight" by the Natural Resource Conservation Service (NRCS). The natural sediments underlying the site, defined by the USDA Soil Conservation Service as being Tokul gravelly medial loam, generally contain substantial quantities of silt and will be sensitive to disturbance when wet. We recommend the following best management practices (BMPs) to mitigate erosion hazards and potential for off-site sediment transport:

1. Construction activity should be scheduled or phased as much as possible to avoid earthwork activity during the wet season.
2. The winter performance of a site is dependent on a well-conceived plan for control of site erosion and stormwater runoff. The site plan should include ground-cover measures and

staging areas. The contractor should be prepared to implement and maintain the required measures to reduce the amount of exposed ground.

3. Temporary erosion and sedimentation control (TESC) elements and perimeter flow control should be established prior to the start of grading.
4. During the wetter months of the year, or when significant storm events are predicted during the summer months, the work area should be stabilized so that if showers occur, it can receive the rainfall without excessive erosion or sediment transport. The required measures for an area to be "buttoned-up" will depend on the time of year and the duration that the area will be left unworked. During the winter months, areas that are to be left unworked for more than 2 days should be mulched or covered with plastic. During the summer months, stabilization will usually consist of seal-rolling the subgrade. Such measures will aid in the contractor's ability to get back into a work area after a storm event. The stabilization process also includes establishing temporary stormwater conveyance channels through work areas to route runoff to the approved treatment/discharge facilities.
5. All disturbed areas should be revegetated as soon as possible. If it is outside of the growing season, the disturbed areas should be covered with mulch. Straw mulch provides a cost-effective cover measure and can be made wind-resistant with the application of a tackifier after it is placed.
6. Surface runoff and discharge should be controlled during and following development. Uncontrolled discharge may promote erosion and sediment transport.
7. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not limited to, covering stockpiles with plastic sheeting or the use of silt fences around pile perimeters.

It is our opinion that with the proper implementation of the TESC plans and by field-adjusting appropriate erosion mitigation (BMPs) throughout construction, the potential adverse impacts from erosion hazards on the project should be mitigated.

III. DESIGN RECOMMENDATIONS

8.0 INTRODUCTION

Our explorations indicate that, from a geotechnical standpoint, the parcel is suitable for the proposed development provided the recommendations contained herein are properly followed. The foundation bearing stratum is relatively shallow and conventional spread footing foundations may be utilized. The vertical infiltration of stormwater on the subject site will be restricted by the relatively low-permeability and fine-grained nature of the lodgement till and glaciomarine drift at depth, and infiltration is not recommended at this site. Alternatives for stormwater management include the utilization of pervious surfaces and ballast as temporary storage under hardscapes.

9.0 SITE PREPARATION

9.1 Clearing and Stripping

Once clearing of vegetation, topsoil, and other unsuitable material has been completed, existing fill and any loose native soils should be addressed. Existing fill and loose soils should be removed from below buildings and replaced as needed with structural fill. We recommend compacting all areas where new structural fill, paving, or structures are planned, followed by proof-rolling with heavy wheeled equipment such as a loaded dump truck. Any areas that are soft, yielding, or deflect during proof-rolling should receive remedial preparation that is selected based on field conditions at the time of construction. Soil cement treatment is discussed in further detail below and is a time-and-cost-efficient way to correct any areas of yielding subgrade soils, and/or to provide all-weather construction laydown and staging areas.

9.2 Temporary and Permanent Cut Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction based on the local conditions encountered at that time. For planning purposes, we anticipate that temporary, unsupported cut slopes within loose fill or weathered till may be made at a maximum inclination of 1.5H:1V (Horizontal:Vertical). Temporary unsupported cut slopes in the dense to very dense lodgement till deposits can be planned at a maximum slope of 1H:1V. Temporary vertical cut slopes in all of these materials may be planned up to a maximum height of 4 feet. Flatter inclinations may be recommended in areas of seepage. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times. Permanent cut slopes should not exceed an inclination of 2H:1V.

9.3 Site Disturbance

The soils encountered during exploration contain a high percentage of fine-grained material. These sediments are moisture-sensitive and are subject to some disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill.

Consideration should be given to protecting access and staging areas with an appropriate section of crushed rock, soil cement treatment (report section 10.4), or asphalt treated base (ATB) (report section 14.2). If crushed rock is considered for the access and staging areas, it should be underlain by engineering stabilization fabric (such as TenCate Mirafi® 500X or approved equivalent) to reduce the potential of fine-grained materials pumping up through the rock during wet weather and turning the area to mud. The fabric will also aid in supporting construction equipment, thus reducing the amount of crushed rock required. We recommend that at least 10 inches of rock be placed over the fabric. Crushed rock used for access and staging areas should be of at least 2-inch size.

10.0 STRUCTURAL FILL

Structural fill may be necessary to establish desired grades in some areas, to backfill around foundations and utilities, and to reestablish grade after any unsuitable soils are removed. All references to structural fill in this report refer to subgrade preparation, fill type, and placement and compaction of materials, as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

10.1 Subgrade Compaction

Initial site preparation should be completed in accordance with Section 9.0 of this report. After overexcavation/stripping have been performed to the satisfaction of the geotechnical engineer/engineering geologist, the upper 12 inches of exposed ground should be compacted to a firm and unyielding condition. If the subgrade contains too much moisture, suitable compaction may be difficult or impossible to obtain, and should probably not be attempted. In lieu of compaction of the subgrade surface, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After compaction of the exposed ground is tested and approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades.

10.2 Structural Fill Compaction

Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts, with each lift being compacted to at least 95 percent of the modified Proctor maximum dry density using *ASTM International* (ASTM) D-1557 as the standard. Roadway and utility trench backfill in public rights-of-way should be placed and compacted in accordance with applicable municipal codes and standards. The top of the compacted fill should extend horizontally a minimum distance of 3 feet beyond the perimeter footings or pavement edges before sloping down at an angle no steeper than 2H:1V. Fill slopes should either be overbuilt and trimmed back to final grade or surface-compacted to the specified density.

10.3 Moisture-Sensitive Fill

Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions and near-optimum subgrade moisture. Those portions of the existing fill or natural on-site sediments that are free of organic debris or other deleterious materials and exhibit a moisture content compatible with achieving the specified level of compaction are suitable for use as structural fill if allowed by project specifications. These materials contain significant amounts of silt and are considered moisture-sensitive. Compaction of these sediments to the specified level of compaction will only be achievable over a relatively narrow range of moisture contents and only during extended periods of dry site and weather conditions. In addition, construction equipment traversing the site when the soils are wet can cause considerable disturbance. If fill is placed during wet weather or if proper compaction cannot be obtained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction.

10.4 Soil-Cement Treatment

Treating weak soils with Portland cement powder can be a cost-effective and time-efficient method of creating a stable working surface on soils that are soft and wet. Soil-cement treatment generally consists of excavating to the desired subgrade elevation, applying dry Portland cement powder to the surface to be treated, mixing the cement powder into the subgrade soils, then striking off and compacting the cement-amended soils. The cement-treated soils are allowed to cure, and where wheeled construction traffic is planned the cement-treated soils are protected with at least 6 inches of crushed rock. Cement treatment is typically significantly less expensive than removal and replacement of soft soils with structural fill. For the purposes of site planning we recommend:

- Verify that Snohomish County will allow soil-cement treatment.

- If cement treatment will be proposed in proximity to buried utility lines under the jurisdiction of the municipal utility purveyors, confirm that the utilities allow the use of cement-treated soils in contact with new buried utilities.

- Assume that construction stormwater runoff from cement-treated areas will be subject to pH monitoring, and that pH reduction using dry ice or carbon dioxide bubblers might be needed.

- Assume a Portland cement application rate of 5 percent by weight, that the upper 12 inches of subgrade soil will be treated, and that the existing soil density is 125 pounds per cubic foot (pcf).

- Soil-cement treatment should only be completed when site conditions are above 40 degrees Fahrenheit and when no substantial rainfall is occurring.

10.5 Structural Fill Testing

The contractor should note that any proposed structural fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material at least 3 business days in advance to perform a Proctor test and determine its field compaction standard.

A representative from our firm should observe the stripped subgrade and be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses and any problem areas may be corrected at that time. It is important to understand that taking random compaction tests on a part-time basis will not assure uniformity or acceptable performance of a fill. As such, we are available to aid the owner in developing a suitable monitoring and testing frequency.

11.0 FOUNDATIONS

For this project, we recommend that new structures be supported by conventional spread footings. Footings should bear on medium dense to very dense native lodgement till or structural fill placed above competent native soils. Any existing fill within the building footprints should be removed and replaced with structural fill in accordance with the recommendations discussed in this report.

11.1 Spread Footings

Conventional spread footings may be utilized for building support when founded on medium dense to very dense, native sediments or on structural fill placed over these materials. Structural fill placed below footing areas should extend outward from the footing edges a distance equal to or greater than the thickness of the fill placed or 2 feet, whichever is less. We recommend that an allowable foundation soil bearing pressure of 2,500 pounds per square foot (psf) be utilized for design purposes, including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings for the proposed buildings should be buried a minimum of 18 inches into the surrounding soil for frost protection. No minimum burial depth is required for interior footings; however, all footings must penetrate to the prescribed stratum and no footings should be founded in or above loose, organic, or existing fill soils. Higher foundation soil bearing pressures are possible at this site but are not expected to be needed for the project as proposed. If higher foundation bearing pressures would be valuable to the project, we should be contacted to discuss situation-specific recommendations.

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area which has not been compacted to at least 95 percent of ASTM D-1557. In addition, a 1.5H:1V line extending down from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edge of steps or cuts in the bearing soils.

Anticipated settlement of footings founded as described above should be on the order of 1 inch or less. However, disturbed soil not removed from footing excavations prior to footing placement could result in increased settlements. All footing areas should be observed by AESI prior to placing concrete to verify that the design bearing capacity of the soils has been attained and that construction conforms to the recommendations contained in this report. Such inspections may be required by the governing municipality. Perimeter footing and wall drains should be provided as discussed under the “Drainage Considerations” section of this report.

12.0 LATERAL WALL PRESSURES

All backfill behind walls or around foundations should be placed following our recommendations for structural fill and as described in this section of the report. Horizontally backfilled walls that are free to yield laterally at least 0.1 percent of their height may be designed to resist active earth pressures represented by an equivalent fluid equal to 35 pcf. Fully restrained, horizontally backfilled, rigid walls that cannot yield should be designed for at-rest pressure represented by an equivalent fluid of 55 pcf. Walls that retain sloping backfill at a maximum angle of 50 percent (2H:1V) should be designed for 55 pcf for yielding conditions and 75 pcf for restrained conditions. If areas to receive vehicle traffic (e.g., parking areas or driveways) are located adjacent to walls, a surcharge equivalent to 2 feet of retained soil should be added to the wall height in determining lateral design forces.

In accordance with the 2018 IBC, retaining wall design should include seismic design parameters. Based on the site soils and assumed wall backfill materials, we recommend a seismic surcharge pressure in addition to the equivalent fluid pressures presented above. A rectangular pressure distribution of 10H and 13H psf (where H is the height of the wall in feet) should be included in design for the “active” and “at-rest” loading conditions, respectively. The resultant of the rectangular seismic surcharge should be applied at the mid-point of the walls.

12.1 Wall Backfill

The lateral pressures presented above are based on the conditions of a uniform backfill consisting of either the on-site natural sediments, or imported sand and gravel compacted to 90 to 95 percent of ASTM D-1557. A higher degree of compaction is not recommended, as this will increase the pressure acting on the walls. A lower compaction may result in unacceptable settlement behind the walls. Thus, the compaction level is critical and must be tested by our firm during placement.

12.2 Wall Drainage

It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. This would involve installation of a minimum 1-foot-wide blanket drain for the full wall height using imported, washed gravel against the walls.

12.3 Passive Resistance and Friction Factor

Lateral loads can be resisted by friction between the foundation and the supporting natural sediments or structural fill soils, or by passive earth pressure acting on the buried portions of the foundations. The foundations must be backfilled with compacted crushed rock to achieve the passive resistance provided below. We recommend the following design parameters:

- Passive equivalent fluid = 250 pcf
- Base friction coefficient = 0.35

13.0 FLOOR SUPPORT

Slab-on-grade floors may be constructed either directly on the natural, medium dense to dense, native sediments, or on structural fill placed over these materials. Areas of the slab subgrade that are disturbed (loosened) during construction should be recompacted to an unyielding condition prior to placing the pea gravel, as described below.

Interior slabs where control of moisture migration through the slab is needed should be cast atop a minimum of 4 inches of clean washed crushed rock or pea gravel to act as a capillary break. Areas of subgrade that are disturbed (loosened) during construction should be compacted to a non-yielding condition prior to placement of capillary break material. It should also be protected from dampness by an impervious moisture barrier at least 10 mils thick. The impervious barrier should be placed between the capillary break material and the concrete slab.

14.0 PAVEMENT AND HARDSCAPE RECOMMENDATIONS

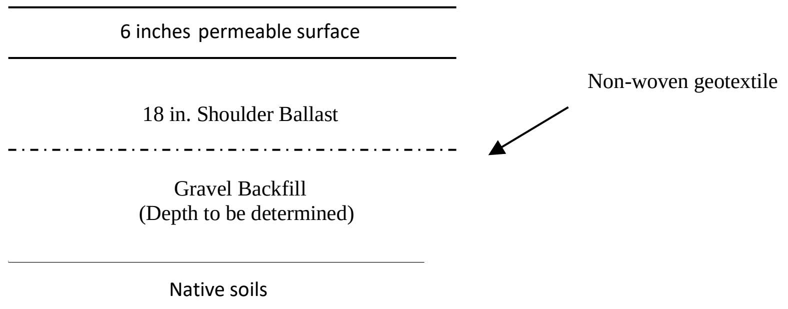
We understand that the current concept will likely include construction of new paved parking lots, access roads, and walkways. At this time we do not anticipate that new paving will be completed on public streets. If new paving is planned on public streets we should be allowed to make situation-specific paving recommendations.

14.1 Porous Asphalt and Portland Cement Paving

We do not anticipate that subgrade soils will accept infiltration at a substantial rate. Snohomish County strongly advocates for Low Impact Development (LID) features in all new projects. Permeable paving could potentially be used to demonstrate LID features to the County. If permeable paving is used, we recommend that a conventional stormwater collection system also be included to accept surface water when the storage capacity of the paving layer and the underlying storage aggregate are full. It is likely that most precipitation will eventually be routed to the conventional stormwater collection system. A small amount of water will likely infiltrate into native soils below the storage aggregate at a very slow rate. Water may stand in the storage aggregate for a long period of time after rain events. We recommend that the storage aggregate layers be arranged to be lower than adjacent buildings and any other structures that could be adversely affected if lateral flow of groundwater seepage out of the pavement drainage aggregate were to occur.

At this time, there are two alternatives to the surfacing including concrete and asphaltic concrete products. Regardless of the surface type, however, the base supporting the surface will have to have adequate storage and be able to infiltrate with a rate greater than the subgrade. The porous pavement section is a combination of storage area for storm events and stability for the paving section and is divided up into two layers. The upper layer is the storage layer and the lower layer is for raising the grade and stability. Between these two layers is a non-woven geotextile for separation; for a typical section, see Sketch 1.

SKETCH 1
TYPICAL POROUS SURFACE SECTION



The upper storage portion must be clean with nominal fines meeting a specification similar to the Washington State Department of Transportation (WSDOT) specification for Shoulder Ballast 9-03.9 (2), which is a clean, 2-inch crushed rock with a porosity of about 0.3. Thickness of the ballast layer could vary depending upon the amount of storage needed for the design storm event and the amount of area draining into the section. However, a minimum of 18 inches is recommended. This material should be rolled to a firm and unyielding condition as the compaction standard since conventional compaction testing methods are impractical on such coarse material. Quality control of the gradation of this material should be of high importance since this layer will affect the ability of the surface to drain readily and provide adequate storage for the design event.

Beneath the storage layer there should be a non-woven geotextile with adequate strength to support the crushed rock layer and prevent migration into the subgrade. Since the geotextile may also have a filtering effect, the geotextile must have an Apparent Opening Size (AOS) greater than the porous surface. This will allow the particles that pass through the surface to also pass through the geotextile and avoid clogging or silting up the geotextile. Therefore the properties of the geotextile will be dependent upon the product chosen for the surface and will be specified at that time. During construction, it will be important to avoid tracking onto the geotextile and the native surface and the crushed surface should be pushed out ahead of placement. No equipment is allowed in direct contact with the geotextile.

The base material below the geotextile will be imported in order to raise the grade of the site and provide stability from the subgrade, if needed. Since this material is also an integral part of the drainage system, strict quality control will be necessary. This material will have to be coarse enough to allow adequate compaction, but yet allow an infiltration rate that meets or exceeds the subgrade infiltration capacity. Therefore we recommend a material that is similar to the WSDOT specification for Sand Drainage Blanket 9-03.13(1), unless otherwise approved. This material should be compacted to 95 percent of maximum dry density. This can be placed in an initial lift of 12 to 18 inches over the native surface to allow the material to be placed on a native surface that has not been recompacted. This is necessary to avoid over-densifying the native surface and disrupting the ability of the native soils to infiltrate. In some cases, depending upon finished grade, this material may not be needed. If this is the case, the geotextile will be placed directly on the stripped surface.

14.2 Standard (Non-Permeable) Pavement

After the area to be paved is stripped, any existing fill soils should be addressed as recommended in the “Site Preparation” section of this report. Upon completion of subgrade recompaction and structural fill, a pavement section consisting of 3 inches of asphaltic concrete pavement (ACP) underlain by 4 inches of 1¼-inch crushed surfacing base course is the recommended minimum in areas of planned passenger car driving and parking. In heavy traffic areas, a minimum pavement section consisting of 4 inches of ACP underlain by 2 inches of $^5/_8$-inch crushed surfacing top course and 4 inches of 1¼-inch crushed surfacing base course is recommended. The crushed rock courses must be compacted to 95 percent of the maximum density, as determined by ASTM D-1557. All paving materials should meet gradation criteria contained in the current WSDOT Standard Specifications.

Depending on construction staging and desired performance, the crushed base course material may be substituted with ATB beneath the final asphalt surfacing. The substitution of ATB should be as follows: 4 inches of crushed rock can be substituted with 3 inches of ATB, and 6 inches of crushed rock may be substituted with 4 inches of ATB. ATB should be placed over a native or structural fill subgrade compacted to a minimum of 95 percent relative density, and a 1½- to 2-inch thickness of crushed rock to act as a working surface. If ATB is used for construction access and staging areas, some rutting and disturbance of the ATB surface should be expected. The contractor should remove affected areas and replace them with properly compacted ATB prior to final surfacing.

15.0 DRAINAGE CONSIDERATIONS

15.1 Wall/Foundation Drains

All retaining and perimeter footing walls should be provided with a drain at the footing elevation. The drains should consist of rigid, perforated polyvinyl chloride (PVC) pipe surrounded by washed gravel. The level of the perforations in the pipe should be set approximately 2 inches below the bottom of the footing, and the drains should be constructed with sufficient gradient to allow gravity discharge away from the buildings. All retaining walls should be lined with a minimum, 12-inch-thick, washed gravel blanket provided to within 1 foot of finish grade, and which ties into the footing drain. Roof and surface runoff should not discharge into the footing drain system but should be handled by a separate, rigid, tightline drain.

Exterior grades adjacent to walls should be sloped downward away from the structures to achieve surface drainage. Final exterior grades should promote free and positive drainage away from the buildings at all times. Water must not be allowed to pond or to collect adjacent to the foundation or within the immediate building area. It is recommended that a gradient of at least 3 percent for a minimum distance of 10 feet from the building perimeter be provided, except in paved locations. In paved locations, a minimum gradient of 1 percent should be provided unless provisions are included for collection and disposal of surface water adjacent to the structures. Additionally, pavement subgrades should be crowned to provide drainage toward catch basins and pavement edges.

16.0 INFILTRATION FEASIBILITY

The results of our subsurface explorations indicate that the shallow infiltration of stormwater on the subject site will be restricted by the relatively low-permeability and fine-grained nature of the lodgement till and shallow infiltration is not recommended at this site. The dense, silty native sediments act as a hydraulically restrictive layer, perching shallow groundwater. Consequently, the infiltrated stormwater will largely move laterally in the shallow subsurface through the overlying fill and weathered soils as interflow. Shallow infiltration of stormwater could result in conditions such as emergent seepage or accumulation of seepage in building crawl spaces, below floor slabs, or around building foundations either on the subject site or on nearby properties.

Deeper infiltration strategies typically target Vashon advance outwash beneath the surficial low-permeability till. Infiltration into the Vashon advance outwash is also considered to have low potential at this site. The advance outwash is expected to have insufficient vertical and lateral extent, and to contain groundwater based on the presence of older pre-Olympia glaciomarine deposits encountered in nearby borings.

17.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. We recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of this current scope of work. If these services are desired, please let us know, and we will prepare a proposal.

We have enjoyed working with you on this study and are confident these recommendations will aid in the successful completion of your project. If you should have any questions or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Peter E. Linton, L.G.
Senior Staff Geologist

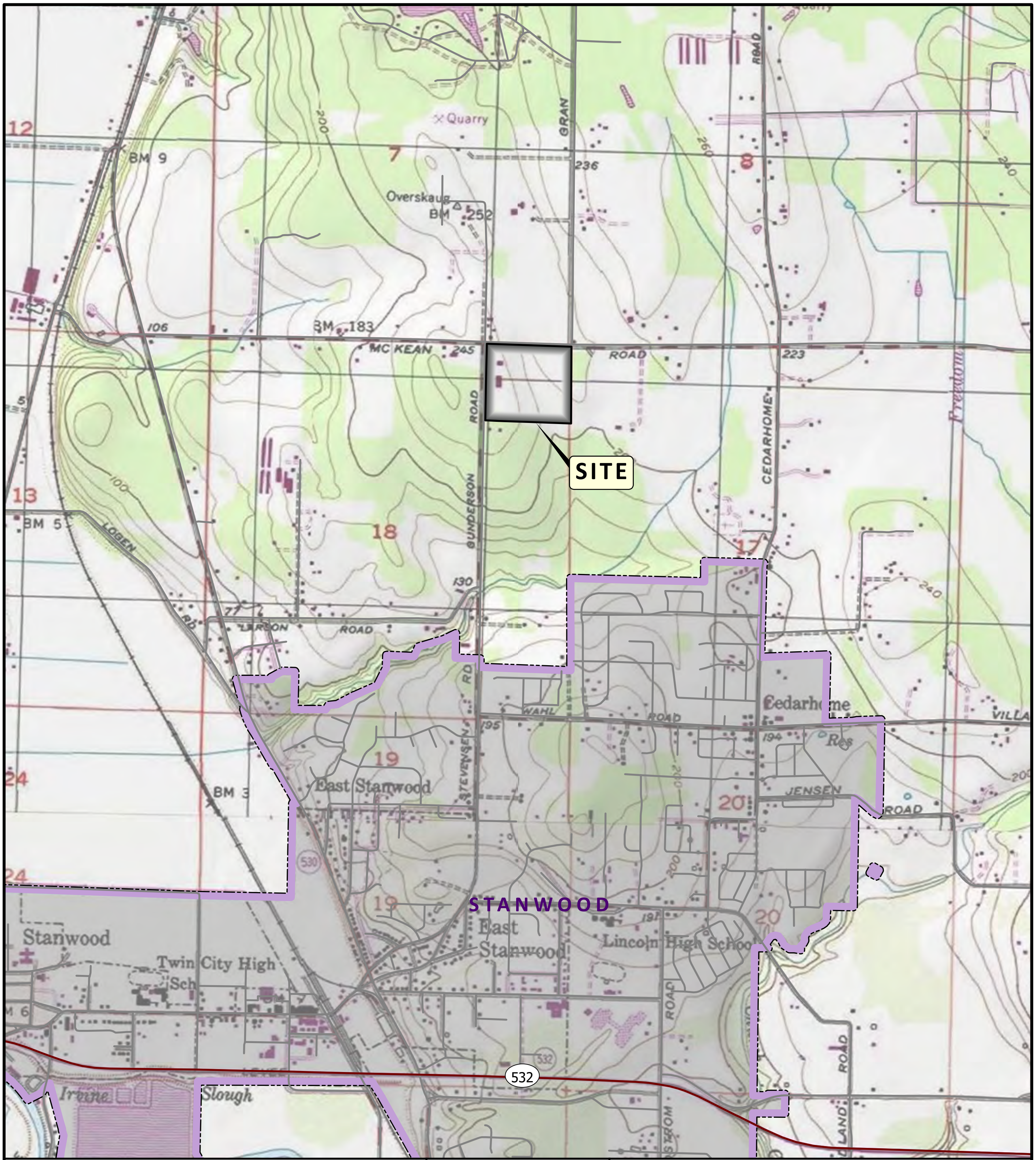
KURT D. MERRIMAN
23580

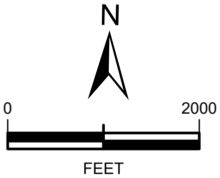
Bruce Guenzler

Bruce W. Guenzler, L.E.G.
Senior Associate Geologist

Kurt D. Merriman, P.E.
Senior Principal Geotechnical Engineer

Attachments:	Figure 1:	Vicinity Map
	Figure 2:	Site and Exploration Plan
	Appendix A:	Exploration Logs





DATA SOURCES / REFERENCES:
USGS: 7.5' SERIES TOPOGRAPHIC MAPS, ESRI/I-CUBED/NATIONAL GEOGRAPHIC SOCIETY 2013
SNOHOMISH CO: STREETS, CITY LIMITS, PARCELS, 3/21
LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE

NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

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VICINITY MAP

STANWOOD RESIDENTIAL TREATMENT FACILITY
SNOHOMISH COUNTY, WASHINGTON

PROJ NO.	DATE:	FIGURE:
20210354E001	11/21	1

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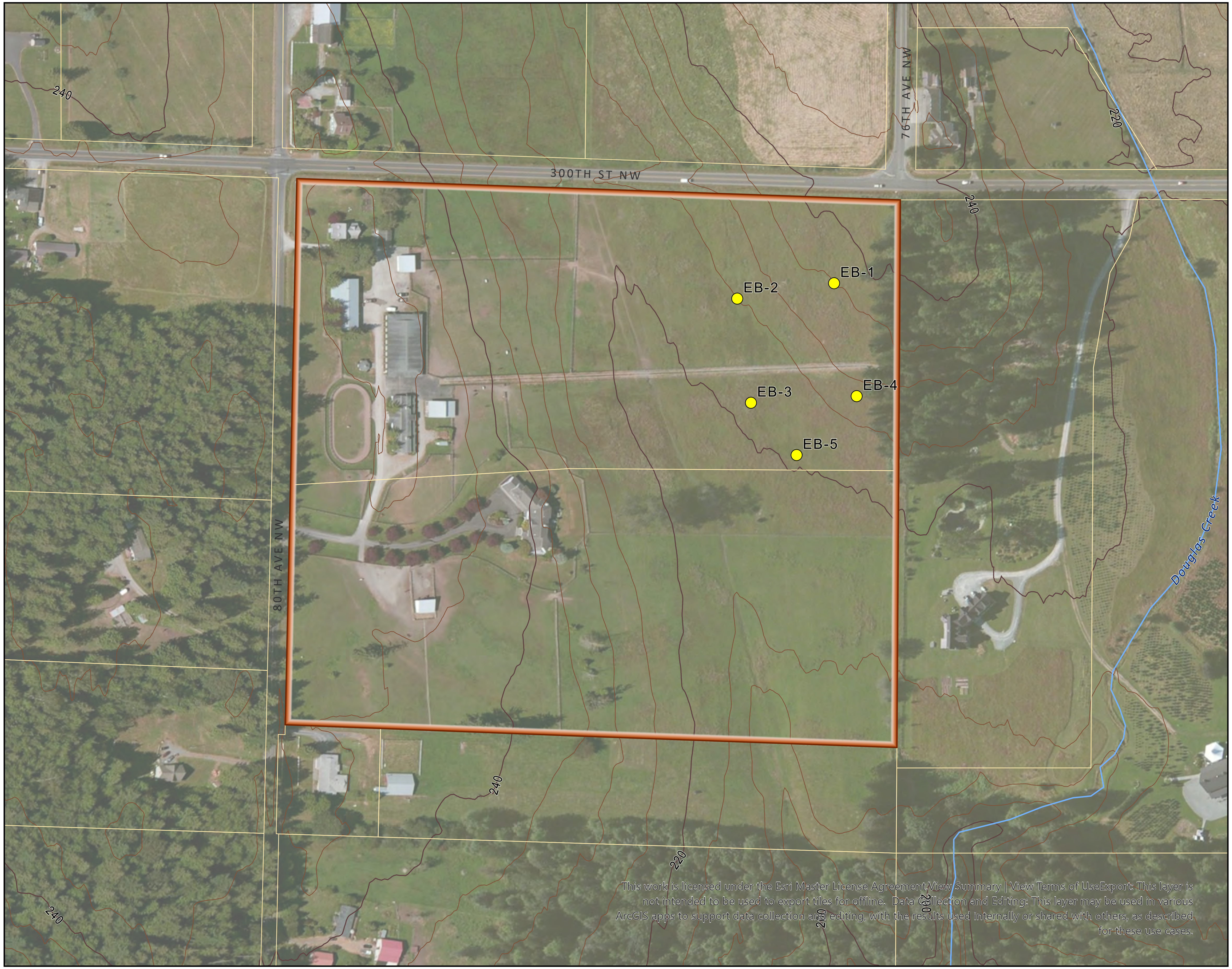


LEGEND

🟥 SITE

● EXPLORATION BORING

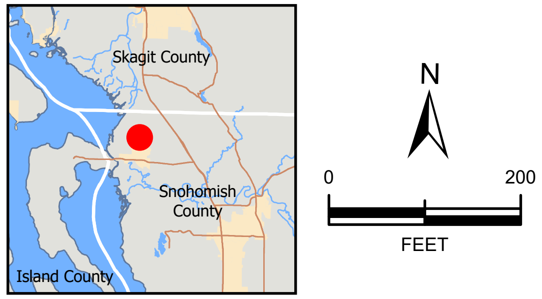
☐ PARCEL

∿ CONTOUR 20 FT

∿ CONTOUR 5 FT

DATA SOURCES / REFERENCES:
WA STATE LIDAR PORTAL: NORTH PUGET SOUND 2016
ACQUIRED MARCH - SEPT 2016, GRID CELL SIZE IS 3'
CONTOURS FROM LIDAR
SNOHOMISH CO: STREETS, PARCELS 3/21
AERIAL: WORLD IMAGERY, ESRI, DIGITAL GLOBE DATEXXXX

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE



BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

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EXISTING SITE AND EXPLORATION PLAN

STANWOOD RESIDENTIAL TREATMENT FACILITY
SNOHOMISH COUNTY, WASHINGTON

PROJ NO.	DATE:	FIGURE:
20210354E001	11/21	2

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APPENDIX A

Exploration Logs

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Exploration Boring

Project Number	Exploration Number	Sheet
20210354E001	EB-1	1 of 1

Project Name Stanwood Residential Treatment Facility
Location Snohomish County, WA
Driller/Equipment ADT / Track Drill
Hammer Weight/Drop 140# / 30
Ground Surface Elevation (ft) 235
Datum NAVD 88
Date Start/Finish 11/4/21,11/4/21
Hole Diameter (in) 7

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot (10 20 30 40)
		Topsoil - 6 inches			
		Vashon Lodgement Till			
		Very moist, brown to dark brown, silty, fine SAND, contains small roots (SM).			
	S-1	Moist, brownish gray with layer (in upper 6 inches of sample) of banded oxidation, silty, fine SAND, trace gravel; unsorted (SM).		7 12 22	▲34
5			▼		
	S-2	Moist, brownish gray, fine to medium SAND, some silt to silty, trace to some gravel; sorted (SP-SM/SM). Grinding drill action 6 to 6.5 feet.		12 23 50/6"	▲50/6"
	S-3	Moist, gray, silty, fine SAND, trace gravel; occasional beds of moist, light gray, sandy, silt; stratified; water in the top of sampler; unsorted (SM).		13 24 37	▲61
10	S-4	Moist, gray, silty, fine SAND, trace gravel; beds of fine SAND, some silt,some gravel; stratified (SP-SM).		17 46 50/6"	▲50/6"
15	S-5	Moist, gray, fine SAND, some silt to silty; interbeds (0.5 to 1 inch thick) of sandy, silt, trace gravel (SP/SP-SM).		50/5"	▲50/5"
20	S-6	Moist, gray, silty, fine SAND, some gravel; unsorted (SM).		37 50/4"	▲50/4"
		Bottom of exploration boring at 20.8 feet Groundwater encountered at 5 feet ATD.			
25					

Sampler Type (ST):

- ◫ 2" OD Split Spoon Sampler (SPT)
- ◫ 3" OD Split Spoon Sampler (D & M)
- ☐ Grab Sample
- ☐ No Recovery
- ◧ Ring Sample
- ▨ Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

Logged by: PL
Approved by: JHS

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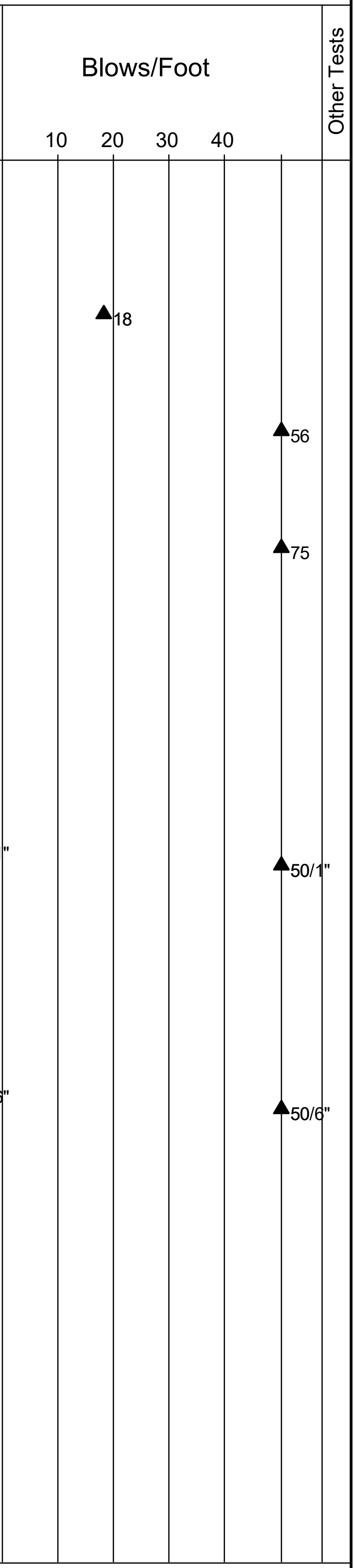
Exploration Boring

Project Number	Exploration Number	Sheet
20210354E001	EB-2	1 of 1

Project Name: Stanwood Residential Treatment Facility
Location: Snohomish County, WA
Driller/Equipment: ADT / Track Drill
Hammer Weight/Drop: 140# / 30
Ground Surface Elevation (ft): 225
Datum: NAVD 88
Date Start/Finish: 11/4/21,11/4/21
Hole Diameter (in): 7

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Other Tests
				Topsoil - 6 inches				
				Fill Very moist to wet, brown to dark brown, silty, fine SAND, trace gravel; contains small roots (SM).				
		S-1		Moist, light brown to brown with reddish brown oxidation staining, silty, fine SAND, trace gravel; non-horizontal zones of sandy, silt (ML) .			6 8 10	
5		S-2		Moist, brown with laminations of dark brown to purplish black in tip of sampler, silty, fine SAND, some gravel (ML).			13 27 29	
				Vashon Lodgement Till Grinding action 6 to 7 feet.				
		S-3		Moist, brownish gray to gray, silty, fine to medium SAND, some gravel; unsorted (SM).			41 36 39	
10								
15		S-4		Moist, gray, silty, fine SAND, trace gravel; poor recovery (SM).			50/1"	
						▼		
				Wet cuttings.				
20		S-5		Moist, gray, silty, fine SAND, trace gravel; unsorted; water in sampler (SM).			50/6"	
				Bottom of exploration boring at 20.5 feet Groundwater encountered from 17 to 18 feet.				
25								



Sampler Type (ST):

- ▯ 2" OD Split Spoon Sampler (SPT)
- ▯ 3" OD Split Spoon Sampler (D & M)
- ☐ Grab Sample
- ☐ No Recovery
- ◧ Ring Sample
- ▨ Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

Logged by: PL
Approved by: JHS

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Exploration Boring

Project Number	Exploration Number	Sheet
20210354E001	EB-3	1 of 1

Project Name Stanwood Residential Treatment Facility
Location Snohomish County, WA
Driller/Equipment ADT / Track Drill
Hammer Weight/Drop 140# / 30

Ground Surface Elevation (ft) 222
Datum NAVD 88
Date Start/Finish 11/4/21,11/4/21
Hole Diameter (in) 7

Depth (ft)	Samples	DESCRIPTION	Blows/6"	Blows/Foot
0		**Topsoil - 6 inches**		
		Fill Very moist, tan to brown, silty, fine SAND, trace gravel (SM).		
		Moist, reddish brown, sandy, SILT (ML).		
	S-1	**Vashon Lodgement Till** Lower 12 inches: brown, fine SAND, some silt to silty; stratified; sharp intact (SM/SP-SM).	22 37 50/5"	50/5"
5	S-2	Moist, gray, silty, fine SAND, trace gravel; unsorted (SM).	17 30 34	64
	S-3	Moist, gray, silty, fine SAND, trace gravel; unsorted (SM).	13 50/6"	50/6"
10	S-4 S-5 S-6	Most, gray, silty, fine SAND, trace gravel; unsorted (SM).	50/6" 50/6" 50/2"	50/6" 50/6" 50/2"
15	S-7	Moist, gray, silty, fine SAND, ranging to sandy, SILT, trace gravel; unsorted (ML).	24 50/5"	50/5"
		Grinding on rock at 17 feet.		
20	S-8	Moist, gray, silty, fine SAND, trace gravel; unsorted; bed (in lower 6 inches of sample) of fine to medium SAND, some silt (SP/SP-SM).	50/4"	50/4"
		Bottom of exploration boring at 20.4 feet No groundwater encountered.		
25				

Sampler Type (ST):

▯ 2" OD Split Spoon Sampler (SPT)	☐ No Recovery	M - Moisture
▯ 3" OD Split Spoon Sampler (D & M)	◧ Ring Sample	▽ Water Level ()
✋ Grab Sample	▨ Shelby Tube Sample	▼ Water Level at time of drilling (ATD)

Logged by: PL
Approved by: JHS

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Exploration Boring

Project Number	Exploration Number	Sheet
20210354E001	EB-4	1 of 1

Project Name Stanwood Residential Treatment Facility
Location Snohomish County, WA
Driller/Equipment ADT / Track Drill
Hammer Weight/Drop 140# / 30

Ground Surface Elevation (ft) 230
Datum NAVD 88
Date Start/Finish 11/4/21,11/4/21
Hole Diameter (in) 7

Depth (ft)	Samples	DESCRIPTION	Blows/6"	Blows/Foot (10 20 30 40)
		Topsoil - 6 inches		
		Weathered Vashon Lodgement Till		
	S-1	Moist to very moist, brownish gray with banding (in upper 12 inches) of reddish brown oxidation, silty, fine SAND, trace gravel; unsorted (SM).	3 7 4	11
		Vashon Lodgement Till		
5	S-2	Moist, brownish gray, silty, fine SAND ranging to sandy, SILT; unsorted (SM/ML) . Driller notes grinding 6 to 7 feet.	11 12 18	30
	S-3	Moist, gray, silty, fine SAND, trace gravel; unsorted (SM).	18 37 50/6"	50/6"
10	S-4	Moist, gray, silty, fine SAND, trace gravel; unsorted; bed of brown to dark brown, fine SAND, trace to some silt (SP-SM). Sharp upper contact; horizontal lower contact.	17 25 50/5"	50/5"
		Vashon Advance Outwash		
15	S-5	Moist, grayish brown, SAND, some silt; laminations of sandy, silt near tip of sampler; stratified (SP-SM).	31 38 31	69
20	S-6	Moist, grayish brown, fine SAND, some silt; laminations of silt, becoming sandy, silt in lower 4 inches; sand above silt is very moist to wet; gradational contact between sand and silt (SP-SM).	20 36 50/6"	50/6"
25	S-7	Moist, brownish gray, fine SAND, some silt with beds of fine sand, trace silt and fine to medium sand, trace silt; stratified (SP-SM).	14 24 37	61

Bottom of exploration boring at 26.5 feet
No groundwater encountered.

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- ☐ No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

Logged by: PL
Approved by: JHS

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Exploration Boring

Project Number	Exploration Number	Sheet
20210354E001	EB-5	1 of 2

Project Name: Stanwood Residential Treatment Facility
Location: Snohomish County, WA
Driller/Equipment: ADT / Track Drill
Hammer Weight/Drop: 140# / 30

Ground Surface Elevation (ft): 220
Datum: NAVD 88
Date Start/Finish: 11/4/21,11/4/21
Hole Diameter (in): 7

Depth (ft)	Samples	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot	Other Tests
	S-1	**Topsoil - 6 inches**					
		Vashon Lodgement Till Very moist, brown and tan, silty, fine SAND, trace gravel; occasional roots (SM). Mud in cuttings.					
	S-2	Moist, brown, fine SAND, some silt to silty, some gravel; gradational contact between upper 12 inches of fine SAND, some silt to lower 6 inches of silty, fine SAND; stratified (SP-SM/SM).			9 20 21	41	
5	S-3	Moist, brownish gray, silty, fine SAND, some gravel; unsorted (SM). Driller notes grinding drill action.		▼	19 35 46	81	
	S-4	Moist, brownish gray, silty, fine SAND, trace gravel; bed (1 to 2 inches thick) of fine sand, some silt, some gravel; diamict; stratified; sharp contact; water in sampler (SM).			16 25 37	62	
10	S-5	Moist, brownish gray, silty, fine SAND, trace gravel; bed (1 inch thick) of fine to medium sand, some silt, some gravel; stratified; silty, fine sand is diamict/unsorted (SP-SM). Muddy cuttings.			18 41 50/5"	50/5"	
15	S-6	Moist, brownish gray to gray, silty, fine SAND, trace to some gravel; unsorted (SM).			18 41 50/6"	50/6"	
20	S-7	Moist, brownish, gray, silty, fine SAND, trace to some gravel; bed (in upper 6 inches of sampler) of sandy, silt ; gradational contact; unsorted (SM).			38 50/4"	50/4"	
		Pre-Olympia Glaciomarine					
25	S-8	Poor recovery; driller notes sand and gravel in cuttings. Driller notes easier drilling 26 to 28.5 feet. Driller notes gravel at 28.5.			50/5"	50/5"	

Blows/Foot: 10 20 30 40

Sampler Type (ST):

▯ 2" OD Split Spoon Sampler (SPT)	☐ No Recovery	M - Moisture
▯ 3" OD Split Spoon Sampler (D & M)	◧ Ring Sample	▽ Water Level ()
✋ Grab Sample	▱ Shelby Tube Sample	▼ Water Level at time of drilling (ATD)

Logged by: PL
Approved by: JHS

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Exploration Boring

Project Number	Exploration Number	Sheet
20210354E001	EB-5	2 of 2

Project Name Stanwood Residential Treatment Facility
Location Snohomish County, WA
Driller/Equipment ADT / Track Drill
Hammer Weight/Drop 140# / 30

Ground Surface Elevation (ft) 220
Datum NAVD 88
Date Start/Finish 11/4/21,11/4/21
Hole Diameter (in) 7

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot 10 20 30 40	Other Tests
		S-9		Moist, gray, silty, fine SAND, some gravel; unsorted; poor recovery with 2 inch sampler; effervesces with diluted hydrochloric acid (SM).			50/4"	50/4"	
35		S-10		Moist, gray, silty, fine SAND ranging to sandy, SILT, trace gravel; unsorted; effervesces with diluted hydrochloric acid (SM/ML).			39 50/5"	50/5"	
40		S-11		Moist, gray, silty, fine SAND, trace gravel; unsorted; effervesces with diluted hydrochloric acid (SM).			47 50/6"	50/6"	
45		S-12 S-13 S-14		Moist, gray, silty, fine SAND, trace gravel; unsorted; effervesces with diluted hydrochloric acid (SM).			50/4" 67/6" 50/4"	50/4" 67/6" 50/4"	
50		S-15		Moist, gray, silty, fine SAND, trace gravel; unsorted; effervesces with diluted hydrochloric acid (SM). Bottom of exploration boring at 50.8 feet Groundwater encountered from 0 to 5 feet.			50/3"	50/3"	
55									

Sampler Type (ST):

2" OD Split Spoon Sampler (SPT)	☐ No Recovery	M - Moisture
3" OD Split Spoon Sampler (D & M)	◧ Ring Sample	▽ Water Level ()
Grab Sample	Shelby Tube Sample	▼ Water Level at time of drilling (ATD)

Logged by: PL
Approved by: JHS

APPENDIX B

GULD DOCUMENT

WASHINGTON STATE
DEPARTMENT OF
ECOLOGY

August 2021

GENERAL USE LEVEL DESIGNATION FOR BASIC (TSS) ENHANCED AND PHOSPHORUS TREATMENT

For

MWS-Linear Modular Wetland

Ecology's Decision

Based on Modular Wetland Systems, Inc, application submissions, including the Technical Evaluation Report, dated April 1, 2014, Ecology hereby issues the following use level designation:

1. General Use Level Designation (GULD) for the MWS-Linear Modular Wetland Stormwater Treatment System for Basic, Phosphorus, and Enhanced treatment
   - Sized at a hydraulic loading rate of:
     - 1 gallon per minute (gpm) per square foot (sq ft) of Wetland Cell Surface Area
     - Prefilter box (approved at either 22 inches or 33 inches tall)
       - 3.0 gpm/sq ft of prefilter box surface area for moderate pollutant loading rates (low to medium density residential basins).
       - 2.1 gpm/sq ft of prefilter box surface area for high pollutant loading rates (commercial and industrial basins).
2. Ecology approves the MWS – Linear Modular Wetland Stormwater Treatment System units for Basic, Phosphorus, and Enhanced treatment at the hydraulic loading rate listed above. Designers shall calculate the water quality design flow rates using the following procedures:
   - Western Washington: For treatment installed upstream of detention or retention, the water quality design flow rate is the peak 15-minute water quality treatment design flow rate as calculated using the latest version of the Western Washington Hydrology Model or other Ecology- approved continuous runoff model.

- Eastern Washington: For treatment installed upstream of detention or retention, the water quality design flow rate is the peak 15-minute water quality treatment design flow rate as calculated using one of the three methods described in Chapter 2.2.5 of the Stormwater Management Manual for Eastern Washington (SWMMEW) or local manual.
- Entire State: For treatment installed downstream of detention, the water quality treatment design flow rate is the full 2-year release rate of the detention facility.

3. These use level designations have no expiration date but may be amended or revoked by Ecology, and are subject to the conditions specified below.

Ecology's Conditions of Use

Applicants shall comply with the following conditions:

1) Design, assemble, install, operate, and maintain the MWS – Linear Modular Wetland Stormwater Treatment System units, in accordance with Modular Wetland Systems, Inc. applicable manuals and documents and the Ecology Decision.
2) Each site plan must undergo Modular Wetland Systems, Inc. review and approval before site installation. This ensures that site grading and slope are appropriate for use of a MWS – Linear Modular Wetland Stormwater Treatment System unit.
3) MSW – Linear Modular Wetland Stormwater Treatment System media shall conform to the specifications submitted to and approved by Ecology.
4) The applicant tested the MWS – Linear Modular Wetland Stormwater Treatment System with an external bypass weir. This weir limited the depth of water flowing through the media, and therefore the active treatment area, to below the root zone of the plants. This GULD applies to MWS – Linear Modular Wetland Stormwater Treatment Systems whether plants are included in the final product or not.
5) Maintenance: The required maintenance interval for stormwater treatment devices is often dependent upon the degree of pollutant loading from a particular drainage basin. Therefore, Ecology does not endorse or recommend a "one size fits all" maintenance cycle for a particular model/size of stormwater treatment technology.
    - Typically, Modular Wetland Systems, Inc. designs MWS – Linear Modular Wetland systems for a target prefilter media life of 6 to 12 months.
    - Indications of the need for maintenance include effluent flow decreasing to below the design flow rate or decrease in treatment below required levels.
    - Owners/operators must inspect MWS – Linear Modular Wetland systems for a minimum of twelve months from the start of post-construction operation to determine site-specific maintenance schedules and requirements. You must conduct inspections monthly during the wet season, and every other month during the dry season (According to the SWMMWW, the wet season in western Washington is October 1 to April

30. According to the SWMMEW, the wet season in eastern Washington is October 1 to June 30). After the first year of operation, owners/operators must conduct inspections based on the findings during the first year of inspections.

- Conduct inspections by qualified personnel, follow manufacturer's guidelines, and use methods capable fo determining either a decrease in treated effluent flowrate and/or a decrease in pollutant removal ability.
- When inspections are performed, the following findings typically serve as maintenance triggers:
  - Standing water remains in the vault between rain events, or
  - Bypass occurs during storms smaller than the design storm.
  - If excessive floatables (trash and debris) are present (but no standing water or excessive sedimentation), perform a minor maintenance consisting of gross solids removal, not prefilter media replacement.
  - Additional data collection will be used to create a correlation between pretreatment chamber sediment depth and pre-filter clogging (see *Issues to be Addressed by the Company* section below)

6) Discharges from the MWS – Linear Modular Wetland Stormwater Treatment System units shall not cause or contribute to water quality standards violations in receiving waters.

Applicant: Modular Wetland Systems, Inc.

Applicant's Address: 5796 Armada Drive, Suite 250
Carlsbad, CA 92008

Application Documents:

Original Application for Conditional Use Level Designation, Modular Wetland System, Linear Stormwater Filtration System Modular Wetland Systems, Inc., January 2011

Quality Assurance Project Plan: Modular Wetland System – Linear Treatment System Performance Monitoring Project, draft, January 2011

Revised Application for Conditional Use Level Designation, Modular Wetland System, Linear Stormwater Filtration System Modular Wetland Systems, Inc., May 2011

Memorandum: Modular Wetland System-Linear GULD Application Supplementary Data, April 2014

Technical Evaluation Report: Modular Wetland System Stormwater Treatment System Performance Monitoring, April 2014

Applicant's Use Level Request:

- General Use Level Designation as a Basic, Enhanced, and Phosphorus treatment device in accordance with Ecology's Guidance for Evaluating Emerging Stormwater Treatment Technologies Technology Assessment Protocol – Ecology (TAPE) January 2011 Revision.

Applicant's Performance Claims:

- The MWS – Linear Modular wetland is capable of removing a minimum of 80-percent of TSS from stormwater with influent concentrations between 100 and 200 mg/L.
- The MWS – Linear Modular wetland is capable of removing a minimum of 50-percent of total phosphorus from stormwater with influent concentrations between 0.1 and 0.5 mg/L.
- The MWS – Linear Modular wetland is capable of removing a minimum 30-percent of dissolved copper from stormwater with influent concentrations between 0.005 and 0.020 mg/L.
- The MWS – Linear Modular wetland is capable of removing a minimum 60-percent of dissolved zinc from stormwater with influent concentrations between 0.02 and 0.30 mg/L.

Ecology's Recommendations:

- Modular Wetland System, Inc. has shown Ecology, through laboratory and field-testing, that the MWS – Linear Modular Wetland Stormwater Treatment System filter system is capable of attaining Ecology's Basic, Phosphorus, and Enhanced treatment goals.

Findings of Fact:

Laboratory Testing

The MWS-Linear Modular wetland has the:

- Capability to remove 99 percent of total suspended solids (using Sil-Co-Sil 106) in a quarter-scale model with influent concentrations of 270 mg/L.
- Capability to remove 91 percent of total suspended solids (using Sil-Co-Sil 106) in laboratory conditions with influent concentrations of 84.6 mg/L at a flow rate of 3.0 gpm per square foot of media.
- Capability to remove 93 percent of dissolved Copper in a quarter-scale model with influent concentrations of 0.757 mg/L.
- Capability to remove 79 percent of dissolved Copper in laboratory conditions with influent concentrations of 0.567 mg/L at a flow rate of 3.0 gpm per square foot of media.

- Capability to remove 80.5-percent of dissolved Zinc in a quarter-scale model with influent concentrations of 0.95 mg/L at a flow rate of 3.0 gpm per square foot of media.
- Capability to remove 78-percent of dissolved Zinc in laboratory conditions with influent concentrations of 0.75 mg/L at a flow rate of 3.0 gpm per square foot of media.

Field Testing

- Modular Wetland Systems, Inc. conducted monitoring of an MWS-Linear (Model # MWS-L-4-13) from April 2012 through May 2013, at a transportation maintenance facility in Portland, Oregon. The manufacturer collected flow-weighted composite samples of the system's influent and effluent during 28 separate storm events. The system treated approximately 75 percent of the runoff from 53.5 inches of rainfall during the monitoring period. The applicant sized the system at 1 gpm/sq ft. (wetland media) and 3gpm/sq ft. (prefilter).
- Influent TSS concentrations for qualifying sampled storm events ranged from 20 to 339 mg/L. Average TSS removal for influent concentrations greater than 100 mg/L (n=7) averaged 85 percent. For influent concentrations in the range of 20-100 mg/L (n=18), the upper 95 percent confidence interval about the mean effluent concentration was 12.8 mg/L.
- Total phosphorus removal for 17 events with influent TP concentrations in the range of 0.1 to 0.5 mg/L averaged 65 percent. A bootstrap estimate of the lower 95 percent confidence limit (LCL95) of the mean total phosphorus reduction was 58 percent.
- The lower 95 percent confidence limit of the mean percent removal was 60.5 percent for dissolved zinc for influent concentrations in the range of 0.02 to 0.3 mg/L (n=11). The lower 95 percent confidence limit of the mean percent removal was 32.5 percent for dissolved copper for influent concentrations in the range of 0.005 to 0.02 mg/L (n=14) at flow rates up to 28 gpm (design flow rate 41 gpm). Laboratory test data augmented the data set, showing dissolved copper removal at the design flow rate of 41 gpm (93 percent reduction in influent dissolved copper of 0.757 mg/L).

Issues to be addressed by the Company:

1. Modular Wetland Systems, Inc. should collect maintenance and inspection data for the first year on all installations in the Northwest in order to assess standard maintenance requirements for various land uses in the region. Modular Wetland Systems, Inc. should use these data to establish required maintenance cycles.
2. Modular Wetland Systems, Inc. should collect pre-treatment chamber sediment depth data for the first year of operation for all installations in the Northwest. Modular Wetland Systems, Inc. will use these data to create a correlation between sediment depth and pre-filter clogging.

Technology Description:
Download at http://www.modularwetlands.com/

Contact Information:

Applicant:	Zach Kent BioClean A Forterra Company 5796 Armada Drive, Suite 250 Carlsbad, CA 92008 zach.kent@forterrabp.com
Applicant website:	http://www.modularwetlands.com/

Ecology web link: http://www.ecy.wa.gov/programs/wg/stormwater/newtech/index.html

Ecology:	Douglas C. Howie, P.E. Department of Ecology Water Quality Program (360) 870-0983 douglas.howie@ecy.wa.gov

Revision History

Date	**Revision**
June 2011	Original use-level-designation document
September 2012	Revised dates for TER and expiration
January 2013	Modified Design Storm Description, added Revision Table, added maintenance discussion, modified format in accordance with Ecology standard
December 2013	Updated name of Applicant
April 2014	Approved GULD designation for Basic, Phosphorus, and Enhanced treatment
December 2015	Updated GULD to document the acceptance of MWS – Linear Modular Wetland installations with or without the inclusion of plants
July 2017	Revised Manufacturer Contact Information (name, address, and email)
December 2019	Revised Manufacturer Contact Address
July 2021	Added additional prefilter sized at 33 inches
August 2021	Changed “Prefilter” to “Prefilter box”