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CITY OF MONROE

BARGHAUSEN

Stormwater Site Plan

Preliminary Plat of Milt Smith Townhomes

523 Park Street
Monroe, Washington 98272

City File No. TBD

Prepared for:
Milt Smith & Associates, Constructors, Inc.
500 108th Ave NE, #2400
Bellevue, WA 98004

03/07/23

March 7, 2023
Our Job No. 22104

BARGHAUSEN CONSULTING ENGINEERS, INC.
18215 72ND AVENUE SOUTH KENT, WA 98032 P) 425.251.6222 F) 425.251.8782
BRANCH OFFICES: CHEHALIS, WA KLAMATH FALLS, OR LONG BEACH, CA RICHLAND, WA ROSEVILLE, CA
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Tab 1.0

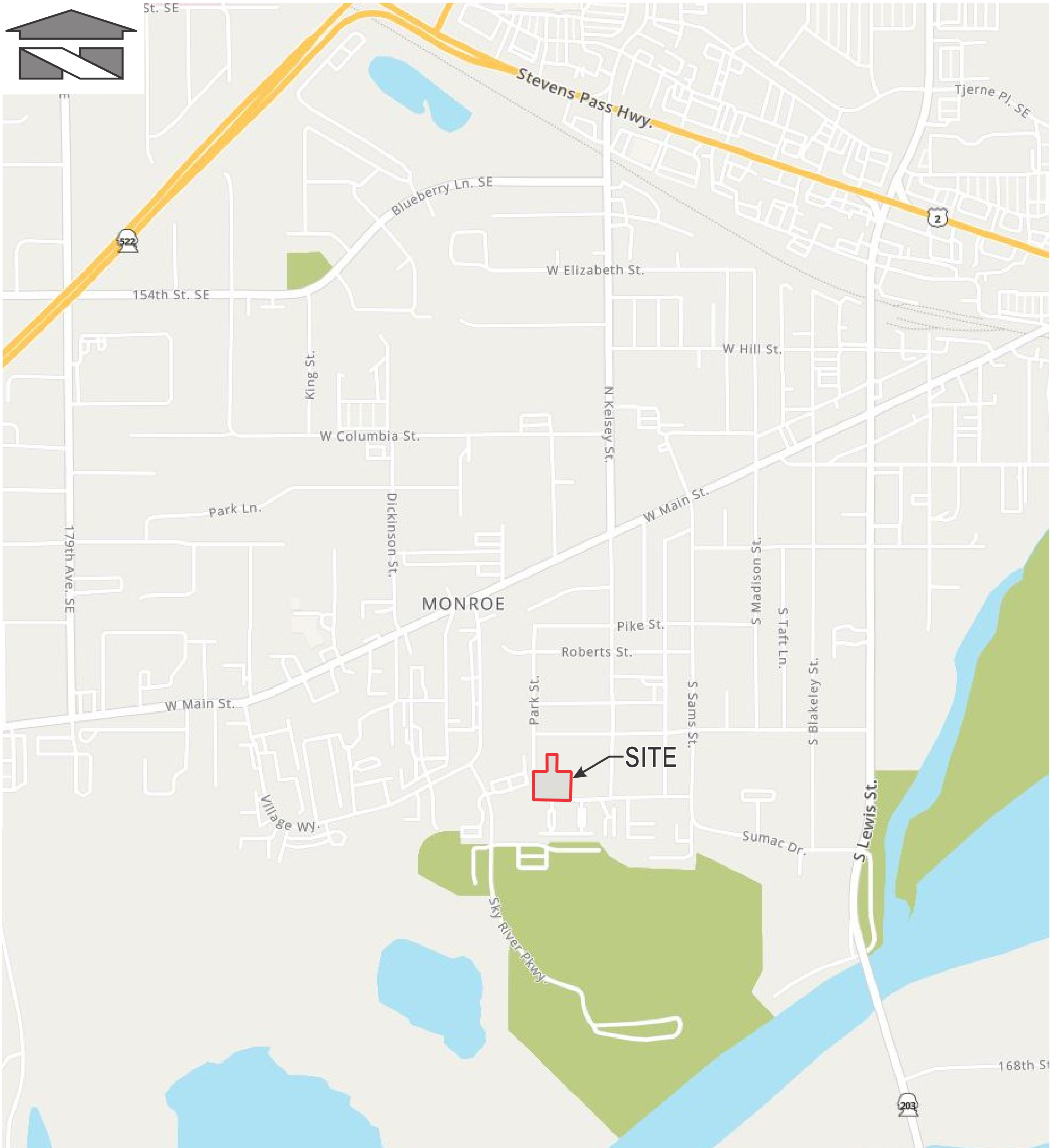
1.0 PROJECT OVERVIEW

The Milt Smith Townhomes is a proposed Multi Family development consisting of 1.02 acres located at 523 Park Street, Monroe, WA 98272. The proposed development will construct one apartment building and one group of townhomes with associated parking, sewer, water and stormwater facilities. The project is located in a portion of the southwest quarter of the southeast quarter of Section 1, Township 27 North, Range 6 East Willamette Meridian. The developed tax parcel is 005583009-00400. A Vicinity Map has been included for reference (see Figure – 1.0.1 located within this section.

The existing property consists of one single family home with several outbuildings on mostly forested land. The site consists of a single drainage basin with the topography sloping gradually and then sharply to the southern boundary. Surface runoff can generally expect to flow towards the south while infiltrating wherever suitable soils are exposed. In the developed condition runoff will be routed via an underground pipe network to an infiltration vault within the center of the site. A modular wetland system or approved equal will provide water quality preceding the infiltration vault.

The NRCS soil survey identifies the underlying soils as Sultan silt loam. Test pits dug by the geotechnical engineer discovered that this matches with the silty soil within the first few feet over the site. Beneath this silty layer more coarse grained soils were discovered to a depth of 15 feet beneath the existing ground. This soil has a higher permeability and is suitable for infiltration designs. For further reference please refer to the Geotechnical Report included in Section 7.0 of this report.

Site drainage designs are based on the 2019 Stormwater Management Manual for Western Washington (2019 SWMMWW). Please refer to Section 5.5 of this Stormwater Site Plan for further details regarding the drainage facility design.



REFERENCE: MapQuest (2023)

Scale:
Horizontal: **N.T.S.** Vertical: **N/A**

For:
Milt Smith Townhomes
Monroe, Washington

Job Number
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Title:
VICINITY MAP

DATE: 02/24/23

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Tab 2.0

2.0 CONDITIONS AND REQUIREMENTS SUMMARY

This section contains the following information:

2.1 Analysis of the Minimum Requirements

2.1 Analysis of the Minimum Requirements

Minimum Requirements	How Project Has Addressed Requirement
No. 1: Preparation of Stormwater Site Plans	This Minimum Requirement has been fulfilled by the preparation and completion of this Stormwater Site Plan.
No. 2: Construction Stormwater Pollution Prevention (SWPP)	A completed Construction Stormwater Pollution Prevention Plan (SWPPP) will be prepared during final engineering.
No. 3: Source Control of Pollution	The project is not classified as a high-use site, and no hazardous materials requiring source control BMPs are proposed to be stored on-site.
No. 4: Preservation of Natural Drainage Systems and Outfalls	The existing site topography is mostly flat sloping generally to the south at a slight grade before increasing to a sharp slope within 50' of the southern boundary. Surface runoff will generally follow this flow path while infiltrating into the onsite soil where suitable soil is exposed. The project proposes to collect surface runoff and provide flow control via an infiltration facility located near the center of the site beneath the flatter portion of the existing site. All runoff generated by the proposed site will be routed to this facility and infiltrated into the ground.
No. 5: On-site Stormwater Management	Low Impact Development (LID) requirements will be met through full infiltration of the onsite runoff. This will meet the pre developed discharge durations for the range from 8% of the 2-year to 50% of the 2-year peak flow. All soil in the lawn and landscaped areas for the site will be amended to meet the Post-Construction Soil Quality and Depth requirement. Each individual lot will have a perforated stub-out connection for rooftops.
No. 6: Runoff Treatment	This project proposes the use of a Modular Wetland System or approved equal to treat the runoff tributary to the developed site.
No. 7: Flow Control	Flow Control for this site is provided by the Infiltration Vault which is sized to fully infiltrate the 100-yr storm as required by the 2019 SWMMWW.
No. 8: Wetlands Protection	There are no documented wetlands recorded for the proposed site.
No. 9: Operation and Maintenance	The drainage facility for this project will be a private facility, owned and maintained by the Property Owner. Operations and Maintenance Manual has been included in Section 9 of this report.

Tab 3.0

3.0 EXISTING CONDITIONS SUMMARY

The Milt Smith Townhomes is a proposed Multi Family development consisting of 1.02 acres located at 523 Park Street, Monroe, WA 98272. The proposed development will construct one apartment building and one group of townhomes with associated parking, sewer, water and stormwater facilities. The project is located in a portion of the southwest quarter of the southeast quarter of Section 1, Township 27 North, Range 6 East Willamette Meridian. The developed tax parcel is 005583009-00400. A Vicinity Map has been included for reference (see Figure 1.0.1), located in Section 1.0 of this report.

In the existing condition the site contains a vacant single family home with some out buildings and an asphalt driveway that connects to the existing road within Park street. On-site vegetation consists of mainly forested land with some smaller areas of lawn. The site is bounded by the city owned Park Street to the west, single and multi-family homes to the north and east and the private Terrace Street to the south. These roads all contain their own conveyance systems that route stormwater runoff away from the site. It is because of this that no upstream drainage is expected to contribute to the site.

The site can be divided into two distinct areas. The northern portion of the site is mostly flat with gradual slopes draining to the south towards the southern portion of the site where the grade sharply increases to slope down towards Terrace Street at the southern boundary of the site. Soils on-site are mapped by the Soil Survey Map as Sultan Silt loam. A geotechnical study of the areas found this is consistent with the initial layers of soil but within the top 3 feet of test pits dug much coarser sandy soils were discovered which are much more amenable to infiltration. The soil survey map is included as figure 3.0.1 in Section 3.0 while the complete geotechnical report is included in Section 7.0.

The site is mostly located in Zone X outside of the floodplain, however the Zone AE floodplain encroaches along the southern boundary of the site which places the project within a special flood hazard area. A Base Flood Elevation has been established at 57.9'. Please reference the FEMA map (Figure 3.0.4) located within this report.

REFERENCE: USDA, Natural Resources Conservation Service

LEGEND:		HSG
66	= Sultan silt loam	C

Scale:
Horizontal: **N.T.S.** Vertical: **N/A**

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For:
Milt Smith Townhomes
Monroe, Washington

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Title:
SOIL SURVEY MAP

DATE: 02/24/23

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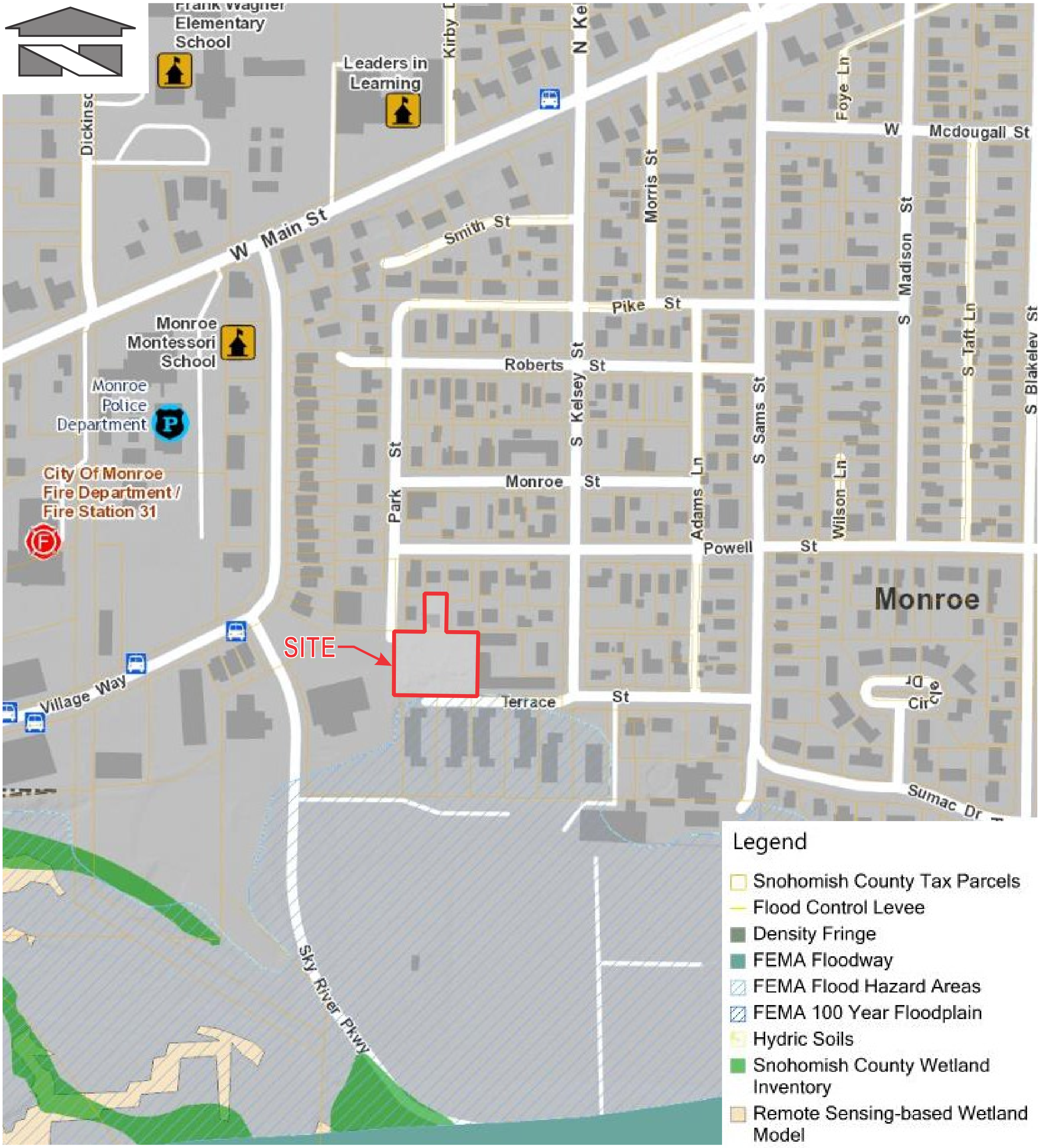

REFERENCE: Snohomish County PDS Map Portal (2023)

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For:
Milt Smith Townhomes
Monroe, Washington

Title:
SENSITIVE AREAS MAP

Job Number
22104

DATE: 02/24/23

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REFERENCE: Snohomish County Department of Assessments (Jan. 2023)

Scale: Horizontal: **N.T.S.** Vertical: **N/A**	*For:* Milt Smith Townhomes Monroe, Washington	Job Number 22104
Barghausen Consulting Engineers, Inc. 18215 72nd Avenue South Kent, WA 98032 425.251.6222 **barghausen.com**	*Title:* ASSESSOR MAP	DATE: 02/24/23

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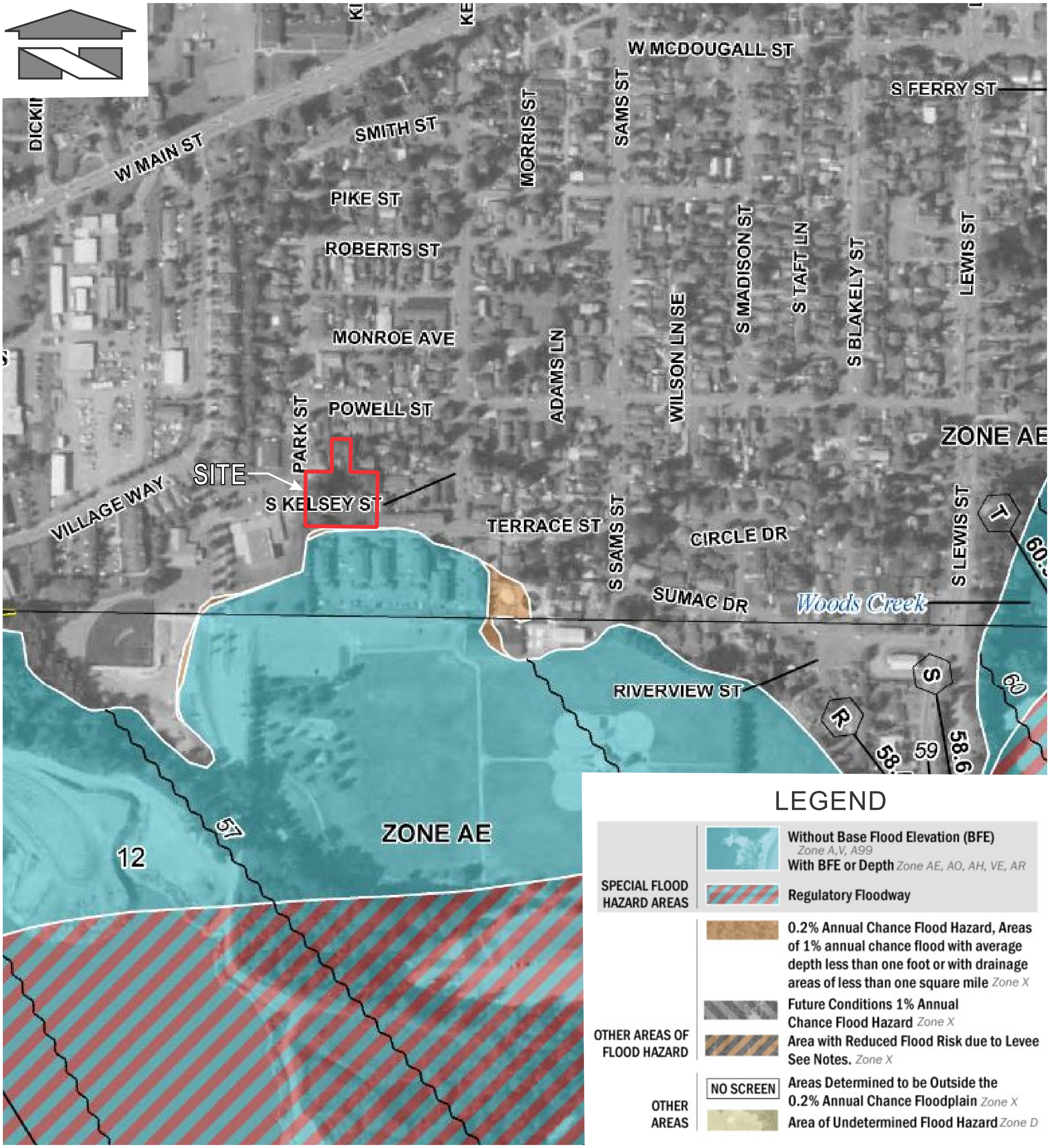

REFERENCE: Federal Emergency Management Agency (Portion of Map 53061C1376G, June 2020)

Scale:

Horizontal: **N.T.S.** Vertical: **N/A**

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For:
Milt Smith Townhomes
Monroe, Washington

Job Number
22104

Title:
FEMA MAP

DATE: 02/24/23

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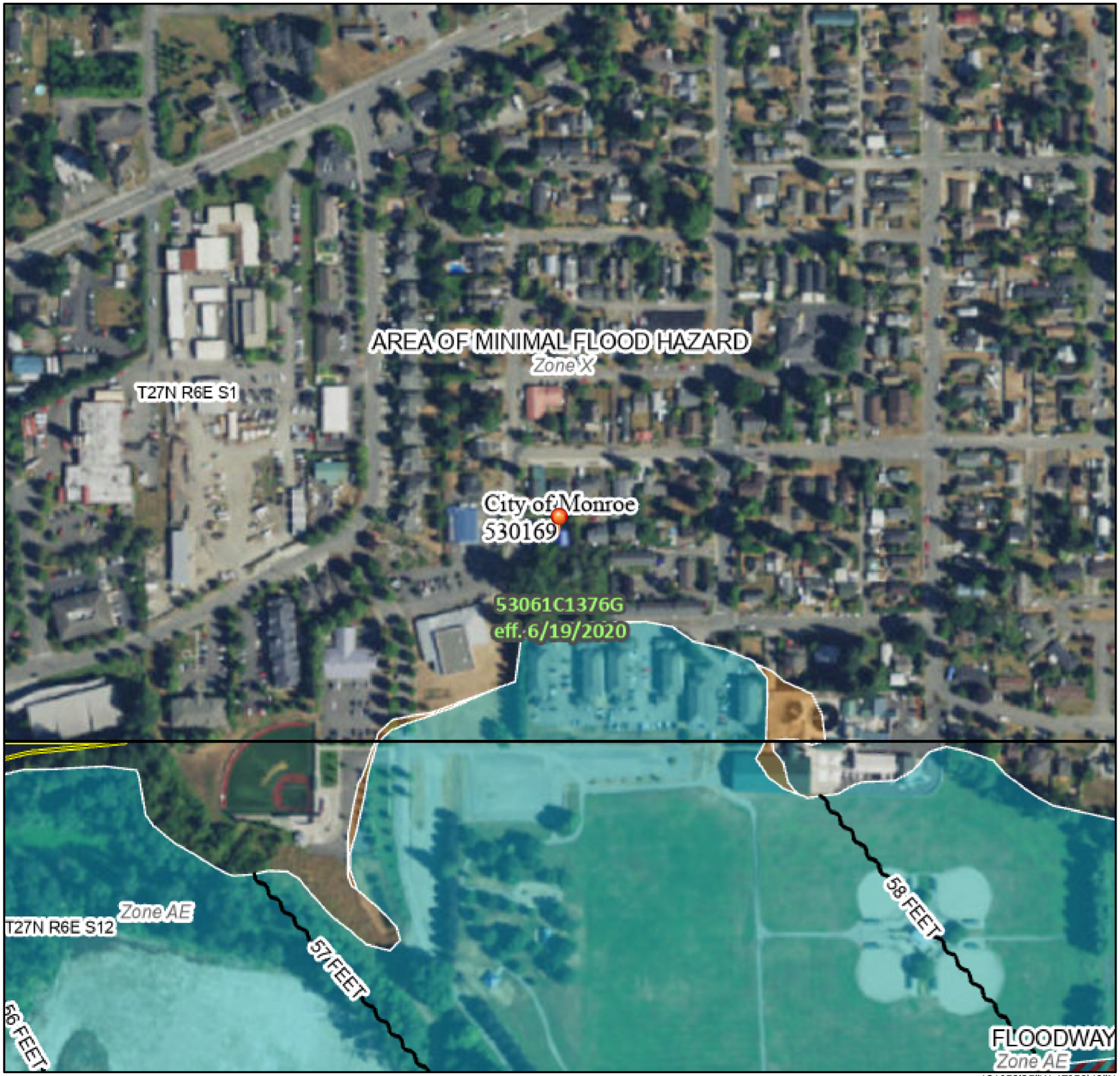
National Flood Hazard Layer FIRMette

FEMA

121°59'2"W 47°51'12"N



121°58'25"W 47°50'48"N

Feet 0 250 500 1,000 1,500 2,000

1:6,000

Basemap: USGS National Map: Orthoimagery: Data refreshed October, 2020

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS	Without Base Flood Elevation (BFE) *Zone A, V, A99*
	With BFE or Depth *Zone AE, AO, AH, VE, AR*
	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
	Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN Area of Minimal Flood Hazard *Zone X*
	Effective LOMRs
	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
OTHER FEATURES	B 20.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
	8 Coastal Transect
	513 Base Flood Elevation Line (BFE)
	Limit of Study
	Jurisdiction Boundary
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
MAP PANELS	Digital Data Available
	No Digital Data Available
	Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 2/22/2023 at 3:45 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Tab 4.0

4.0 OFF-SITE ANALYSIS REPORT

The project is proposing to fully infiltrate the runoff generated by the site impervious area, and as such there will are no anticipated impacts to the downstream areas of the project.

Tab 5.0

5.0 PERMANENT STORMWATER CONTROL PLAN

This section contains the following information:

5.1 Existing Site Hydrology

5.2 Developed Site Hydrology

5.3 Performance Standards and Goals

5.4 Low Impact Development Features

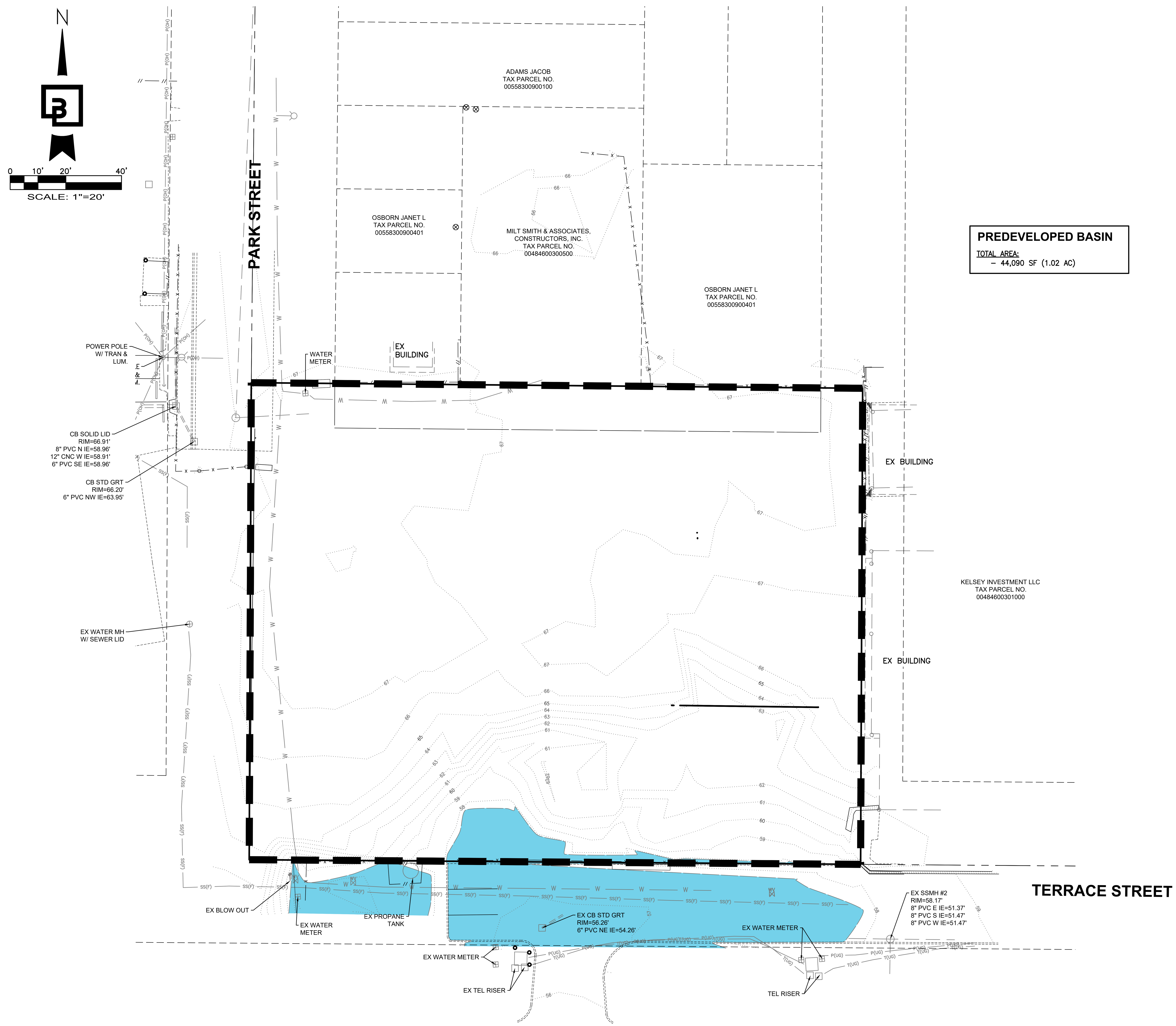
5.5 Flow Control System

5.6 Water Quality System

5.1 Existing Site Hydrology

The existing drainage basin for the project consists of 1.02 acres of area and has been modeled as forested till soil. See Figure 5.1.1 – Predeveloped Drainage Basin for more detailed information.

PREDEVELOPED DRAINAGE BASIN



No.	Date	By	Ckd.	Appr.	Revision

Title:
PREDEVELOPED DRAINAGE BASIN

For:
MILT SMITH & ASSOCIATES, CONSTRUCTORS, INC
500 108TH NE, #2400
BELLEVUE, WA 98004
JASON SMITH (425) 450-1106

Scale: Horizontal 1"=20' Vertical N/A

Designed TRA
Drawn TRA
Checked TRA
Approved BJT
Date 3/7/23

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Job Number 22104

Sheet 1 of 1

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5.2 Developed Site Hydrology

The developed site will create two multi family and apartment buildings with impervious alleys and pavement for parking and emergency access. The drainage facility will be located under the pavement in the north central portion of the site.

A conveyance system consisting of catch basins and buried pipes will route runoff from the newly created impervious alleys, sidewalks, parking and roof areas. Water quality will be provided by a Modular Wetland system (or approved equal) preceding the Infiltration Facility. For further information of the water quality treatment design and sizing please reference Section 5.6 of this report.

The infiltration facility has been sized using the WWHM2012 program. The facility has been sized to infiltrate the full 100-yr storm. By infiltrating the entirety of the site this facility meets the LID performance standards of matching 8% of the predeveloped 2-yr storm through 50% of the predeveloped 2-yr storm.

Drainage Basin

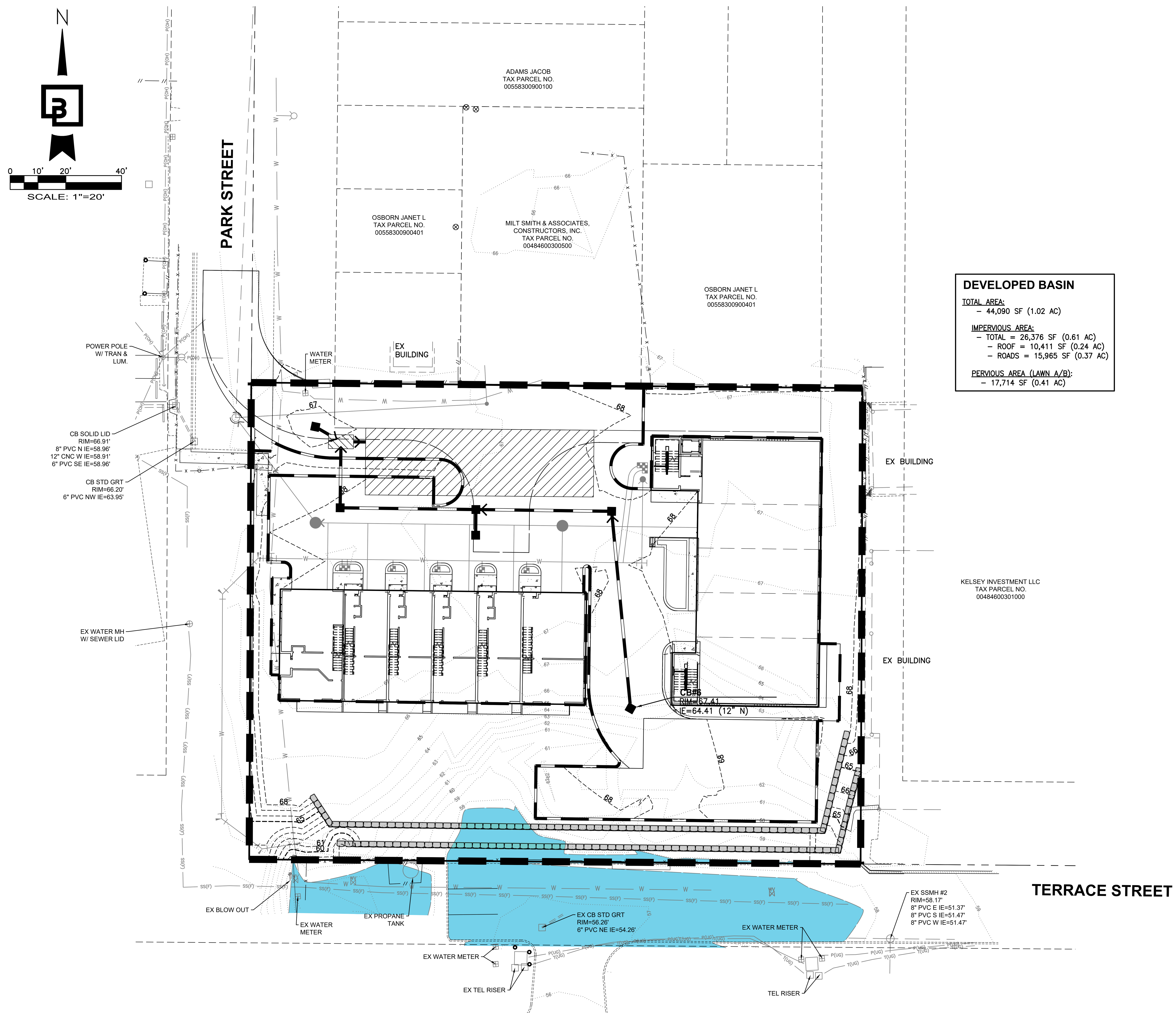
The infiltration facility has been sized to accommodate the drainage basin tributary to the site. The developed basin map has been included as Figure 5.2.1. To size the facility the developed basin has been described in the table below:

Impervious	Pervious	Total Area
0.61 Ac(1)	0.41 Ac(2)	1.02 Ac

Notes:
1. Onsite Alleys, Parking and Sidewalks
2. Modeled as till lawn

Per the WWHM2012 calculations included in Section 5.5, the total volume required for the infiltration facility is 6,720 cubic feet. For further design details and WWHM calculations please refer to Section 5.5 of this report.

DEVELOPED DRAINAGE BASIN



DEVELOPED BASIN

TOTAL AREA:
- 44,090 SF (1.02 AC)

IMPERVIOUS AREA:
- TOTAL = 26,376 SF (0.61 AC)
  - ROOF = 10,411 SF (0.24 AC)
  - ROADS = 15,965 SF (0.37 AC)

PERVIOUS AREA (LAWN A/B):
- 17,714 SF (0.41 AC)

No.	Date	By	Ckd.	Appr.	Revision

Title: DEVELOPED DRAINAGE BASIN

For: MILT SMITH & ASSOCIATES, CONSTRUCTORS, INC
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BELLEVUE, WA 98004
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Scale: Horizontal 1"=20' Vertical N/A

Designed TRA
Drawn TRA
Checked TRA
Approved BJT
Date 3/7/23

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Job Number 22104

Sheet 1 of 1

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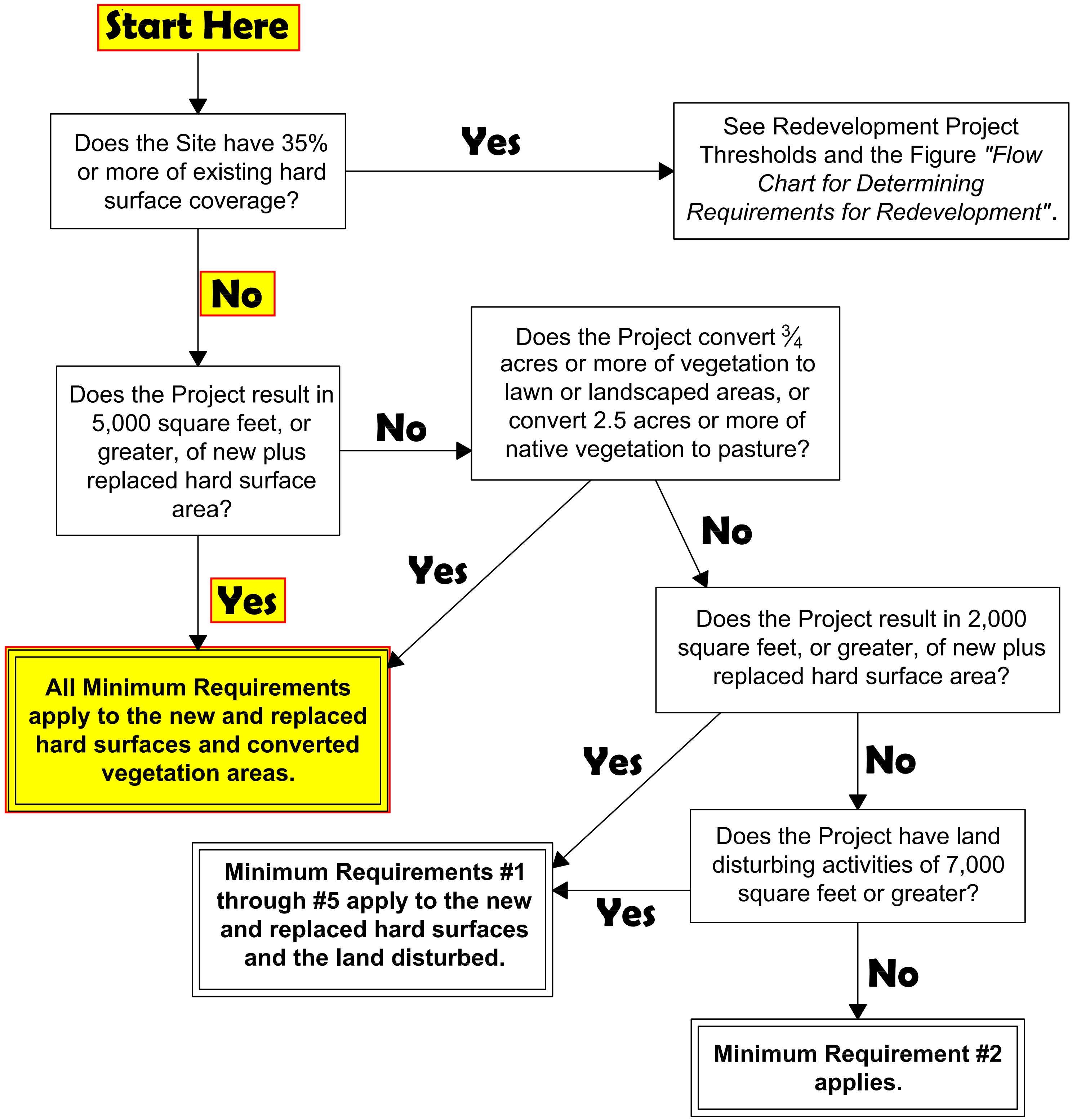
5.3 Performance Standards and Goals

The project is subject to the provisions of the City of Monroe's design standards and guidelines, as well as the 2019 Stormwater Management Manual for Western Washington. This report, along with the accompanying plans, are intended to satisfy the Stormwater Site plan preparation requirements outlined in the regulatory documents listed above. See Figure 5.3.1 for the flowchart determining the requirements for this new development project.

Hydrologic modeling was performed using the 2012 Western Washington Hydrology Model hence forth referred to as WWHM2012. WWHM2012 is a locally calibrated continuous simulation model developed by the Washington State Department of Ecology. The model evaluates several decades of hydrologic data to derive peak flow rate and duration information. Please reference, Section 5.5 of this report for further information.

Water quality treatment will be provided via a Modular Wetland System (or approved equal). Please reference Section 5.6 of this report for further information.

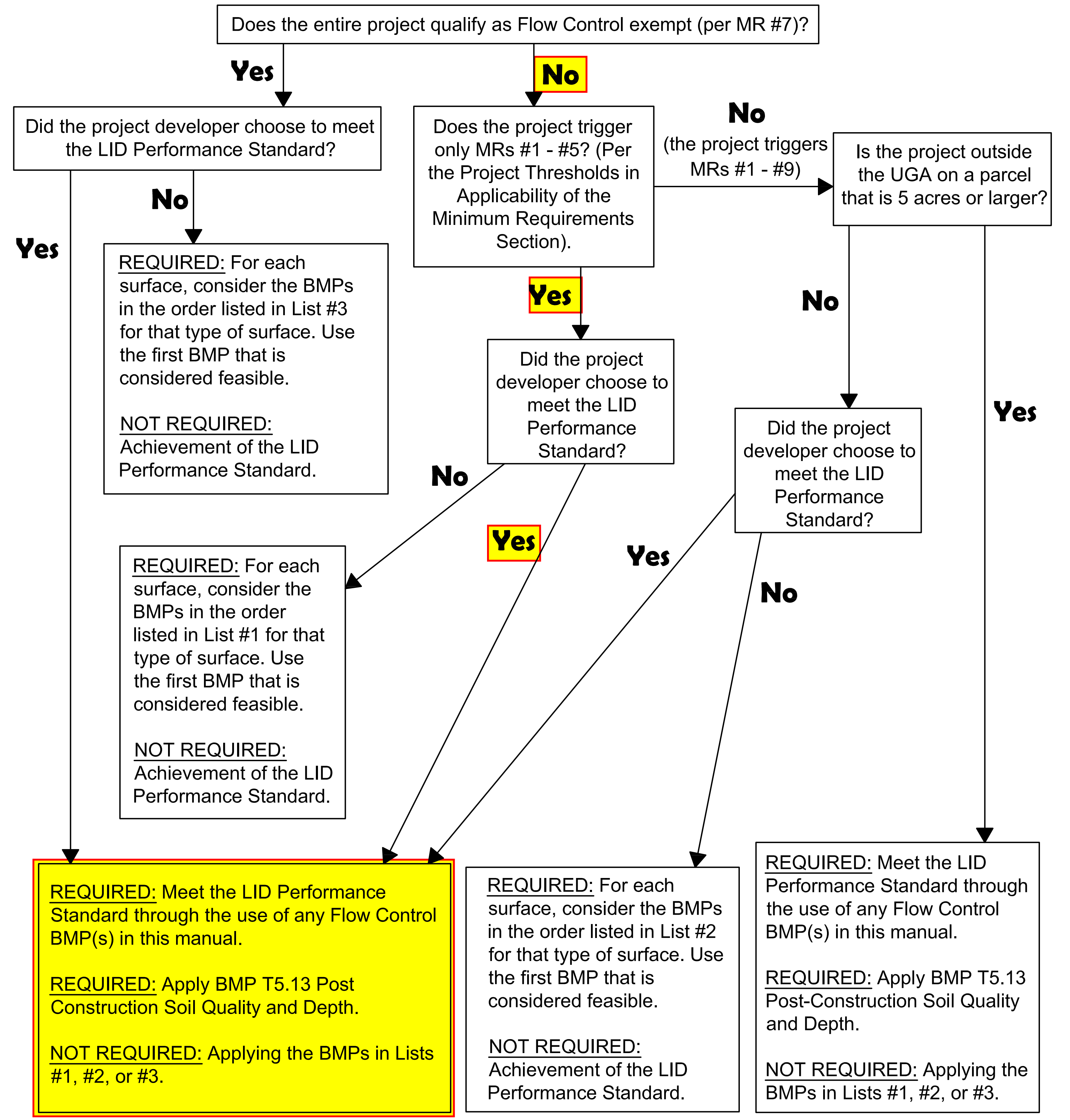
This project has opted to meet the LID performance standard which requires that the drainage facility match the predeveloped durations from 8% of the 2-yr to the 50% 2-yr storm. See the completed flow chart for determining the Minimum Requirements #5 included as figure 5.3.2 in this section. By using an infiltration facility the LID performance standard is met. All soil in the lawn and landscaped areas for the site will be amended to meet the post-Construction Soil Quality and Depth requirement. Each individual rooftop area will have a perforated stub-out connection.



DEPARTMENT OF
ECOLOGY
State of Washington

Flow Chart for Determining Requirements for New Development

Revised March 2019



DEPARTMENT OF ECOLOGY
State of Washington

Flow Chart for Determining MR #5 Requirements

Revised March 2019

5.4 Low Impact Development Features

This project has opted to meet the LID performance standard which requires that the drainage facility match the predeveloped durations from 8% of the 2-yr to the 50% 2-yr storm. See the completed flow chart for determining the Minimum Requirements #5 included as Figure 5.3.2 in this section. By using an infiltration facility the LID performance standard is met.

All soil in the lawn and landscaped areas for the site will be amended to meet BMP T5.13 Post-Construction Soil Quality and Depth. Additionally, each all rooftop areas will have a Perforated Stub-Out Connection in compliance with BMP T5.10C.

5.5 Flow Control System

Surface runoff from the site will be routed to the detention vault located in northern center of the site. A detailed breakdown of the basin areas can be located in Section 5.2 of this report. This facility has been sized using the WWHM2012 stormwater program. The infiltration system is designed to infiltrate the full 100-yr storm. The volume required for the detention vault is 6,720 cubic feet. These design calculations are provided within this section of the report.

WWHM2012

PROJECT REPORT

INFILTRATION FACILITY

General Model Information

Project Name:	22104-Developed Basin
Site Name:	
Site Address:	
City:	
Report Date:	2/27/2023
Gage:	Everett
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.200
Version Date:	2019/09/13
Version:	4.2.17

POC Thresholds

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

Landuse Basin Data
Predeveloped Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Forest, Flat	1.02
Pervious Total	1.02
Impervious Land Use	acre
Impervious Total	0
Basin Total	1.02

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Lawn, Flat	0.41
Pervious Total	0.41
Impervious Land Use	acre
ROADS FLAT	0.61
Impervious Total	0.61
Basin Total	1.02

Element Flows To:

Surface	Interflow	Groundwater
Vault 1	Vault 1	

Routing Elements
Predeveloped Routing

Mitigated Routing

Vault 1

Width: 24 ft.
Length: 80 ft.
Depth: 4 ft.
Infiltration On
Infiltration rate: 2
Infiltration safety factor: 1
Total Volume Infiltrated (ac-ft.): 162.388
Total Volume Through Riser (ac-ft.): 0
Total Volume Through Facility (ac-ft.): 162.388
Percent Infiltrated: 100
Total Precip Applied to Facility: 0
Total Evap From Facility: 0
Discharge Structure
Riser Height: 3.5 ft.
Riser Diameter: 18 in.
Element Flows To:
Outlet 1 Outlet 2

REQUIRED VOLUME
= 6,720 CF

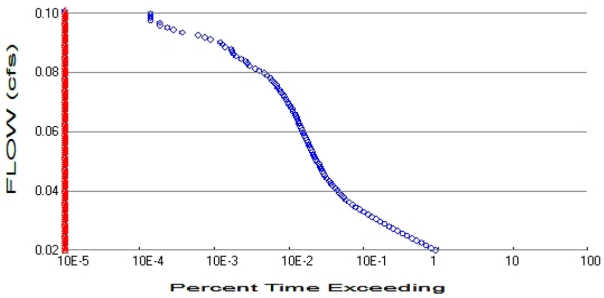
Vault Hydraulic Table

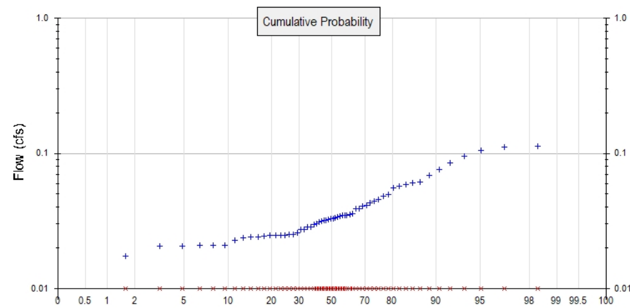
Stage(feet)	Area(ac.)	Volume(ac-ft.)	Discharge(cfs)	Infilt(cfs)
0.0000	0.044	0.000	0.000	0.000
0.0444	0.044	0.002	0.000	0.088
0.0889	0.044	0.003	0.000	0.088
0.1333	0.044	0.005	0.000	0.088
0.1778	0.044	0.007	0.000	0.088
0.2222	0.044	0.009	0.000	0.088
0.2667	0.044	0.011	0.000	0.088
0.3111	0.044	0.013	0.000	0.088
0.3556	0.044	0.015	0.000	0.088
0.4000	0.044	0.017	0.000	0.088
0.4444	0.044	0.019	0.000	0.088
0.4889	0.044	0.021	0.000	0.088
0.5333	0.044	0.023	0.000	0.088
0.5778	0.044	0.025	0.000	0.088
0.6222	0.044	0.027	0.000	0.088
0.6667	0.044	0.029	0.000	0.088
0.7111	0.044	0.031	0.000	0.088
0.7556	0.044	0.033	0.000	0.088
0.8000	0.044	0.035	0.000	0.088
0.8444	0.044	0.037	0.000	0.088
0.8889	0.044	0.039	0.000	0.088
0.9333	0.044	0.041	0.000	0.088
0.9778	0.044	0.043	0.000	0.088
1.0222	0.044	0.045	0.000	0.088
1.0667	0.044	0.047	0.000	0.088
1.1111	0.044	0.049	0.000	0.088
1.1556	0.044	0.050	0.000	0.088
1.2000	0.044	0.052	0.000	0.088
1.2444	0.044	0.054	0.000	0.088
1.2889	0.044	0.056	0.000	0.088
1.3333	0.044	0.058	0.000	0.088
1.3778	0.044	0.060	0.000	0.088

1.4222	0.044	0.062	0.000	0.088
1.4667	0.044	0.064	0.000	0.088
1.5111	0.044	0.066	0.000	0.088
1.5556	0.044	0.068	0.000	0.088
1.6000	0.044	0.070	0.000	0.088
1.6444	0.044	0.072	0.000	0.088
1.6889	0.044	0.074	0.000	0.088
1.7333	0.044	0.076	0.000	0.088
1.7778	0.044	0.078	0.000	0.088
1.8222	0.044	0.080	0.000	0.088
1.8667	0.044	0.082	0.000	0.088
1.9111	0.044	0.084	0.000	0.088
1.9556	0.044	0.086	0.000	0.088
2.0000	0.044	0.088	0.000	0.088
2.0444	0.044	0.090	0.000	0.088
2.0889	0.044	0.092	0.000	0.088
2.1333	0.044	0.094	0.000	0.088
2.1778	0.044	0.096	0.000	0.088
2.2222	0.044	0.097	0.000	0.088
2.2667	0.044	0.099	0.000	0.088
2.3111	0.044	0.101	0.000	0.088
2.3556	0.044	0.103	0.000	0.088
2.4000	0.044	0.105	0.000	0.088
2.4444	0.044	0.107	0.000	0.088
2.4889	0.044	0.109	0.000	0.088
2.5333	0.044	0.111	0.000	0.088
2.5778	0.044	0.113	0.000	0.088
2.6222	0.044	0.115	0.000	0.088
2.6667	0.044	0.117	0.000	0.088
2.7111	0.044	0.119	0.000	0.088
2.7556	0.044	0.121	0.000	0.088
2.8000	0.044	0.123	0.000	0.088
2.8444	0.044	0.125	0.000	0.088
2.8889	0.044	0.127	0.000	0.088
2.9333	0.044	0.129	0.000	0.088
2.9778	0.044	0.131	0.000	0.088
3.0222	0.044	0.133	0.000	0.088
3.0667	0.044	0.135	0.000	0.088
3.1111	0.044	0.137	0.000	0.088
3.1556	0.044	0.139	0.000	0.088
3.2000	0.044	0.141	0.000	0.088
3.2444	0.044	0.143	0.000	0.088
3.2889	0.044	0.145	0.000	0.088
3.3333	0.044	0.146	0.000	0.088
3.3778	0.044	0.148	0.000	0.088
3.4222	0.044	0.150	0.000	0.088
3.4667	0.044	0.152	0.000	0.088
3.5111	0.044	0.154	0.018	0.088
3.5556	0.044	0.156	0.208	0.088
3.6000	0.044	0.158	0.502	0.088
3.6444	0.044	0.160	0.869	0.088
3.6889	0.044	0.162	1.291	0.088
3.7333	0.044	0.164	1.756	0.088
3.7778	0.044	0.166	2.248	0.088
3.8222	0.044	0.168	2.755	0.088
3.8667	0.044	0.170	3.261	0.088
3.9111	0.044	0.172	3.752	0.088
3.9556	0.044	0.174	4.216	0.088

4.0000	0.044	0.176	4.639	0.088
4.0444	0.044	0.178	5.012	0.088
4.0889	0.000	0.000	5.330	0.000

Analysis Results
POC 1





+ Predeveloped   x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 1.02
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0.41
Total Impervious Area: 0.61

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	0.034272
5 year	0.052575
10 year	0.06669
25 year	0.086886
50 year	0.103705
100 year	0.122099

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)
2 year	0
5 year	0
10 year	0
25 year	0
50 year	0
100 year	0

Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

Year	Predeveloped	Mitigated
1949	0.034	0.000
1950	0.035	0.000
1951	0.031	0.000
1952	0.025	0.000
1953	0.021	0.000
1954	0.112	0.000
1955	0.044	0.000
1956	0.039	0.000
1957	0.048	0.000
1958	0.035	0.000

1959	0.035	0.000
1960	0.032	0.000
1961	0.061	0.000
1962	0.030	0.000
1963	0.050	0.000
1964	0.036	0.000
1965	0.030	0.000
1966	0.017	0.000
1967	0.035	0.000
1968	0.043	0.000
1969	0.104	0.000
1970	0.025	0.000
1971	0.039	0.000
1972	0.029	0.000
1973	0.027	0.000
1974	0.059	0.000
1975	0.024	0.000
1976	0.025	0.000
1977	0.021	0.000
1978	0.025	0.000
1979	0.069	0.000
1980	0.032	0.000
1981	0.025	0.000
1982	0.033	0.000
1983	0.056	0.000
1984	0.034	0.000
1985	0.041	0.000
1986	0.096	0.000
1987	0.046	0.000
1988	0.024	0.000
1989	0.024	0.000
1990	0.032	0.000
1991	0.033	0.000
1992	0.025	0.000
1993	0.021	0.000
1994	0.023	0.000
1995	0.033	0.000
1996	0.057	0.000
1997	0.114	0.000
1998	0.021	0.000
1999	0.027	0.000
2000	0.020	0.000
2001	0.008	0.000
2002	0.031	0.000
2003	0.024	0.000
2004	0.041	0.000
2005	0.029	0.000
2006	0.076	0.000
2007	0.060	0.000
2008	0.084	0.000
2009	0.026	0.000

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.1135	0.0000
2	0.1120	0.0000
3	0.1045	0.0000

4	0.0957	0.0000
5	0.0844	0.0000
6	0.0760	0.0000
7	0.0686	0.0000
8	0.0609	0.0000
9	0.0601	0.0000
10	0.0588	0.0000
11	0.0571	0.0000
12	0.0557	0.0000
13	0.0495	0.0000
14	0.0482	0.0000
15	0.0457	0.0000
16	0.0441	0.0000
17	0.0430	0.0000
18	0.0410	0.0000
19	0.0407	0.0000
20	0.0389	0.0000
21	0.0389	0.0000
22	0.0356	0.0000
23	0.0353	0.0000
24	0.0350	0.0000
25	0.0349	0.0000
26	0.0346	0.0000
27	0.0342	0.0000
28	0.0336	0.0000
29	0.0335	0.0000
30	0.0329	0.0000
31	0.0327	0.0000
32	0.0322	0.0000
33	0.0321	0.0000
34	0.0320	0.0000
35	0.0313	0.0000
36	0.0311	0.0000
37	0.0301	0.0000
38	0.0298	0.0000
39	0.0287	0.0000
40	0.0285	0.0000
41	0.0273	0.0000
42	0.0272	0.0000
43	0.0257	0.0000
44	0.0252	0.0000
45	0.0251	0.0000
46	0.0247	0.0000
47	0.0246	0.0000
48	0.0246	0.0000
49	0.0246	0.0000
50	0.0244	0.0000
51	0.0241	0.0000
52	0.0239	0.0000
53	0.0237	0.0000
54	0.0228	0.0000
55	0.0209	0.0000
56	0.0208	0.0000
57	0.0208	0.0000
58	0.0207	0.0000
59	0.0205	0.0000
60	0.0174	0.0000
61	0.0082	0.0000

Duration Flows
The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.0171	19622	0	0	Pass
0.0180	16987	0	0	Pass
0.0189	14675	0	0	Pass
0.0198	12741	0	0	Pass
0.0206	10930	0	0	Pass
0.0215	9445	0	0	Pass
0.0224	8168	0	0	Pass
0.0233	7080	0	0	Pass
0.0241	6126	0	0	Pass
0.0250	5313	0	0	Pass
0.0259	4654	0	0	Pass
0.0268	4066	0	0	Pass
0.0276	3555	0	0	Pass
0.0285	3136	0	0	Pass
0.0294	2761	0	0	Pass
0.0303	2447	0	0	Pass
0.0311	2145	0	0	Pass
0.0320	1892	0	0	Pass
0.0329	1657	0	0	Pass
0.0338	1508	0	0	Pass
0.0346	1370	0	0	Pass
0.0355	1251	0	0	Pass
0.0364	1154	0	0	Pass
0.0372	1069	0	0	Pass
0.0381	1009	0	0	Pass
0.0390	950	0	0	Pass
0.0399	888	0	0	Pass
0.0407	825	0	0	Pass
0.0416	777	0	0	Pass
0.0425	733	0	0	Pass
0.0434	687	0	0	Pass
0.0442	648	0	0	Pass
0.0451	622	0	0	Pass
0.0460	602	0	0	Pass
0.0469	583	0	0	Pass
0.0477	561	0	0	Pass
0.0486	538	0	0	Pass
0.0495	506	0	0	Pass
0.0504	487	0	0	Pass
0.0512	473	0	0	Pass
0.0521	457	0	0	Pass
0.0530	440	0	0	Pass
0.0539	424	0	0	Pass
0.0547	409	0	0	Pass
0.0556	394	0	0	Pass
0.0565	380	0	0	Pass
0.0574	368	0	0	Pass
0.0582	353	0	0	Pass
0.0591	341	0	0	Pass
0.0600	333	0	0	Pass
0.0609	322	0	0	Pass
0.0617	313	0	0	Pass
0.0626	302	0	0	Pass

0.0635	293	0	0	Pass
0.0644	284	0	0	Pass
0.0652	276	0	0	Pass
0.0661	265	0	0	Pass
0.0670	257	0	0	Pass
0.0679	241	0	0	Pass
0.0687	234	0	0	Pass
0.0696	224	0	0	Pass
0.0705	212	0	0	Pass
0.0714	205	0	0	Pass
0.0722	195	0	0	Pass
0.0731	187	0	0	Pass
0.0740	177	0	0	Pass
0.0748	166	0	0	Pass
0.0757	160	0	0	Pass
0.0766	150	0	0	Pass
0.0775	146	0	0	Pass
0.0783	135	0	0	Pass
0.0792	128	0	0	Pass
0.0801	120	0	0	Pass
0.0810	111	0	0	Pass
0.0818	99	0	0	Pass
0.0827	85	0	0	Pass
0.0836	75	0	0	Pass
0.0845	63	0	0	Pass
0.0853	59	0	0	Pass
0.0862	56	0	0	Pass
0.0871	49	0	0	Pass
0.0880	42	0	0	Pass
0.0888	39	0	0	Pass
0.0897	37	0	0	Pass
0.0906	36	0	0	Pass
0.0915	30	0	0	Pass
0.0923	28	0	0	Pass
0.0932	26	0	0	Pass
0.0941	19	0	0	Pass
0.0950	16	0	0	Pass
0.0958	13	0	0	Pass
0.0967	8	0	0	Pass
0.0976	6	0	0	Pass
0.0985	5	0	0	Pass
0.0993	4	0	0	Pass
0.1002	4	0	0	Pass
0.1011	3	0	0	Pass
0.1020	3	0	0	Pass
0.1028	3	0	0	Pass
0.1037	3	0	0	Pass

Water Quality

Water Quality BMP Flow and Volume for POC #1

On-line facility volume:	0.086 acre-feet
On-line facility target flow:	0.1133 cfs.
Adjusted for 15 min:	0.1133 cfs.
Off-line facility target flow:	0.0638 cfs.
Adjusted for 15 min:	0.0638 cfs.

LID Report

LID Technique	Used for Treatment ?	Total Volume Needs Treatment (ac-ft)	Volume Through Facility (ac-ft)	Infiltration Volume (ac-ft)	Cumulative Volume Infiltration Credit	Percent Volume Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Vault 1 POC	☐	147.77			☐	100.00			
Total Volume Infiltrated		147.77	0.00	0.00		100.00	0.00	0%	No Treat. Credit
Compliance with LID Standard 8% of 2-yr to 50% of 2-yr									Duration Analysis Result = Passed

Model Default Modifications

Total of 0 changes have been made.

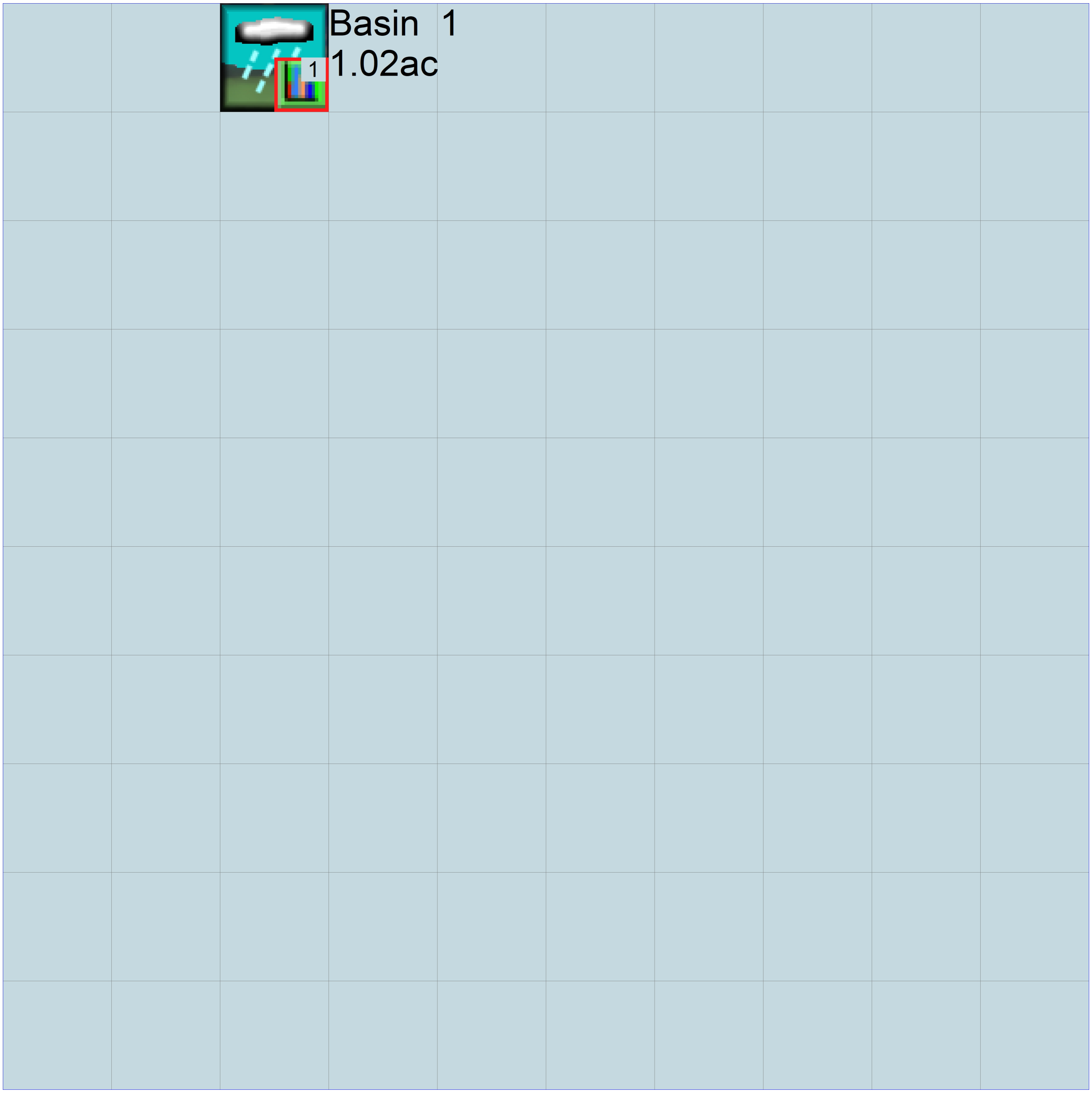
PERLND Changes

No PERLND changes have been made.

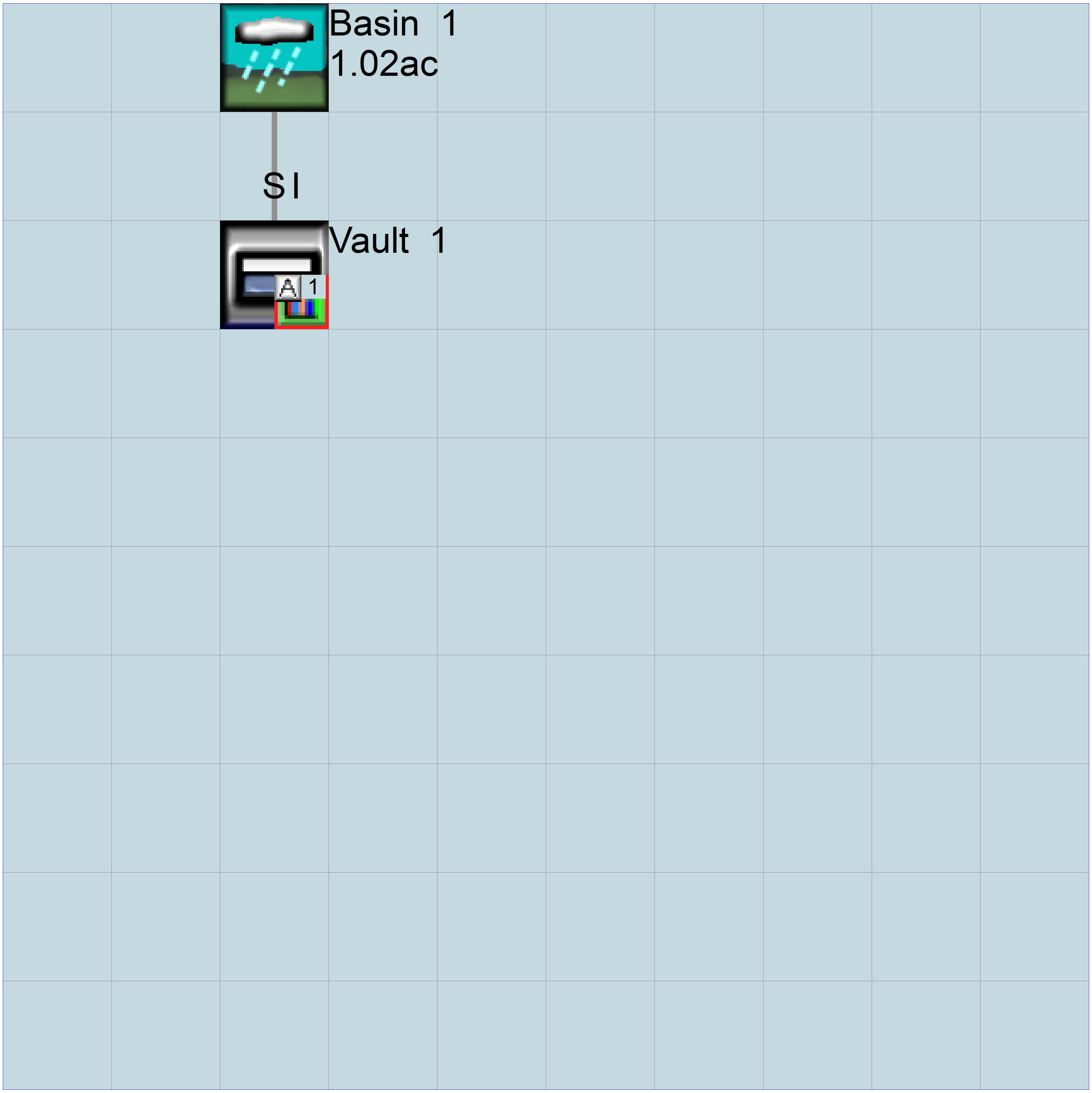
IMPLND Changes

No IMPLND changes have been made.

Appendix
Predeveloped Schematic

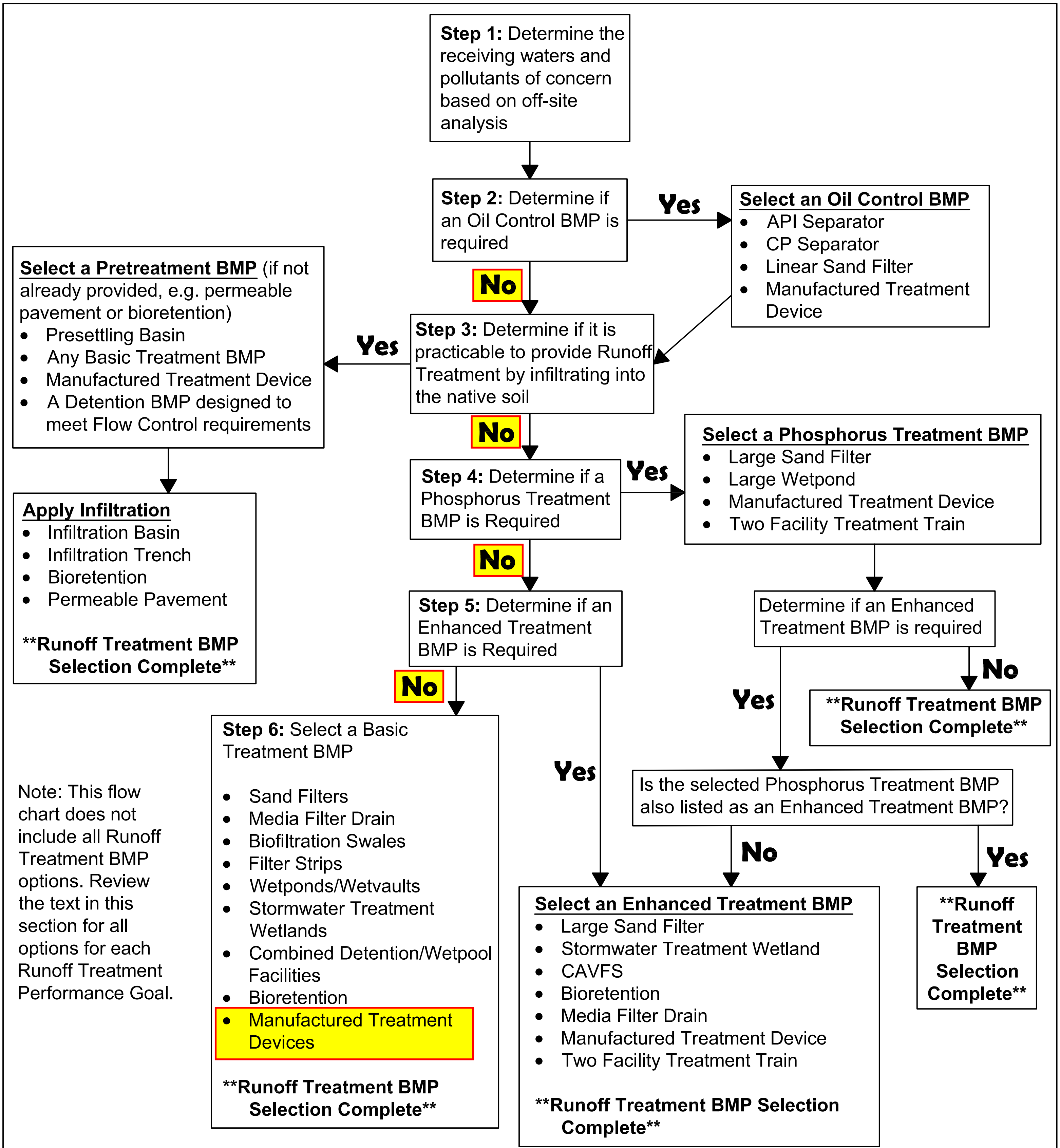


Mitigated Schematic



5.6 Water Quality System

Runoff treatment for the drainage basin will be provided by a Modular Wetland System (or approved equal). The treatment vault will be located immediately preceding the infiltration vault; therefore, it will be designed to treat 91% of the 2-yr storm volume which is equal to 3,746 cubic feet. The final design of the Water Quality facility will be provided during final engineering.



DEPARTMENT OF ECOLOGY
State of Washington

Runoff Treatment BMP Selection Flow Chart

Revised January 2019

Tab 6.0

6.0 CONSTRUCTION STORMWATER POLLUTION PREVENTION PLAN

A complete analysis of the 12 Stormwater Pollution Prevention Plan (SWPPP) elements will be included during final engineering.

Tab 7.0

7.0 SPECIAL REPORTS AND STUDIES

This section contains the following information:

7.1 Geotechnical Engineering Study – Proposed Residential Development Vacant Lot South of 523 Park Street Monroe, Washington dated February 22, 2023

GEOTECH
CONSULTANTS, INC.

2401 10th Ave E
Seattle, Washington 98102
(425) 747-5618

February 22, 2023

JN 22256

Milt Smith and Associates Construction, Inc.
500 – 108th Avenue N.E., #2400
Bellevue, Washington 98004

Attention : Jason Smith
via email: jason.smith@kidder.com

Subject: **Geotechnical Engineering Study**
Proposed Residential Development
Vacant Lot South of 523 Park Street
Monroe, Washington

Greetings:

This report presents our geotechnical engineering findings and recommendations related to the planned development of the vacant property located immediately to the south of the existing residence addressed 523 Park Street. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide design considerations for foundations, retaining walls, subsurface drainage, and on-site stormwater infiltration. This work by your acceptance of our **Contract for Professional Services**.

We were provided with a Site Plan prepared by Baylis Architects. Based on this information, and our discussions with the project team, the existing residence (#523) will remain. The large parcel to the south of this residence will be developed with two detached residential structures. The western building will contain townhomes, and the eastern building will have flats. The buildings will be located on the northern two-thirds of the site. The northwestern portion of the parcel will contain a storm water facility beneath a large paved area. The entrance to the project will be from Park Street near the northwest corner of the site. In the southeastern corner of the property, fill will be placed above the sloping and lower existing grade to create additional parking. The southwestern portion of the site may also be filled to create open space close to the elevation of the development on the northern portion of the site. A retaining wall or terraced walls will have to be constructed along the low sides of this filled area.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site in the southeastern portion of Monroe. Park Street dead-ends at the northwestern corner of the site. A gravel-surfaced driveway extends into the northwestern portion of the site to provide access to the existing residence addressed 523 Park Street. This house, which was vacant at the time of our field work, is located on a separate parcel from the planned new development. The project site is rectangular in shape, and is covered primarily with underbrush and blackberry vines. There are trees of various types and ages scattered around the site. Most of these are younger deciduous trees, with occasional larger trees interspersed. In the northwestern corner of the subject site, along the south side of the driveway, is a shed and some temporary awnings. To the south of the shed is a pile of old floor tiles and bags of aluminum cans, and we observed an old car covered by blackberry vines in the approximate center of the site.

The ground surface on the northern approximately two-thirds of the site is relatively flat, with some slight undulations. The southern approximately one-third of the site slopes downward toward the adjacent southern property. The grade change in this portion of the site is no more than approximately 8 feet. This sloped area has been modified by previous grading, based both on our observations and on the results of the test pits.

The neighboring properties are developed primarily with multi-family buildings. There is a single-family home to the west of the vacant house (#523), north of the existing driveway. Apartments are located on the properties to the northeast, east, and south of the site. The apartment buildings to the east are relatively close to the common property line. Immediately to the south is a paved driveway located in the right-of-way for Terrace Street. This driveway provides access within the adjacent southern apartment complex. The buildings within that complex are located at least 30 feet to the south of the site.

Based on utility maps from the City of Monroe, a water main currently extends through the western edge of the site, within old Park Street right-of-way, and then turns eastward just outside the site's southern property line.

SUBSURFACE

The subsurface conditions on the property have been explored by excavating twelve test pits at the approximate locations shown on the Site Exploration Plan, Plate 2. These test pits were conducted on June 7, 2022 using a mini excavator and on August 17, 2022 using a large trackhoe. A geotechnical engineer from our firm observed the excavation process and logged the condition encountered in the test pits. Logs of the test pits are included as Plate 3 through 6.

Soil Conditions

Planned Northern Development Area: Test Pits 3 – 5, 7 – 9, and 12 were conducted on the northern two-thirds of the property, which is essentially flat. With the exception of Test Pits 8 and 9, uppermost soil layer underlying the topsoil in this area consisted of loose, weathered silt extending to a depth of 2.5 to 3.5 feet.

In Test Pits 8 and 9, we observed a layer of fill underlying a thin topsoil layer. This fill varied in thickness from one foot in Test Pit 9 to 6.5 feet in Test Pit 8. The fill was loose, and contained organics and roots. Underlying the fill in these test pits was the loose, weathered silt encountered in the other explorations.

Underlying the weathered silt, all of the test pits conducted on this portion of the property found coarse-grained soil consisting of very gravelly sand that contained numerous cobbles. This soil has been deposited by fast-flowing water, and is in a medium-dense to dense condition. In some of the test pits, it was compact enough to be difficult to excavate with the mini trackhoe. The coarse-grained soils were encountered to the maximum 15-foot depth reached in Test Pit 12.

<u>Southern One-Third of Site:</u> This portion of the site, which slopes downward toward the south, was explored by Test Pits 1, 2, 10, and 11. All of these explorations found variable uncontrolled fill beneath a thin topsoil layer. This fill contained varying amounts of debris consisting of small to large chunks of concrete and scattered rubbish (wood, plastic, carpet, etc.). The fill extended to depths of 4 feet (Test Pit 10) to 8 feet (Test Pit 11). The slope in the southern portion of the site was apparently created by this past fill placement.

Underlying the fill, the test pits revealed occasional old topsoil, beneath which was loose, unconsolidated alluvial soils consisting of sandy silt, silt, and silty sand that contained small organics and rootlets. This soil was deposited by slow-moving water, resulting in its loose condition. Test Pits 10 and 11 extended to depths of 15 to 17 feet, without encountering dense or coarse-grained soils.

<u>Groundwater Conditions</u>

The test pits were excavated during the summer months.

Groundwater was encountered only in Test Pit 10. This seepage occurred between 12 and 13 feet, and came from a thin layer of sand trapped between low permeability silt soils. In wet weather, we expect that at least localized subsurface water could be encountered on top of the silty alluvial soils on the south portion of the property.

On the northern portion of the site, groundwater was not observed in the test pits. However, in Test Pit 12, which was the deepest, the soil became wet below a depth of 14 feet. This indicates that the recent high groundwater level is near this depth.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. The relative densities and moisture descriptions indicated on the test pit logs are interpretive descriptions based on the conditions observed during excavation.

<u>CONCLUSIONS AND RECOMMENDATIONS</u>

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The test pits conducted for this study encountered competent, coarse-grained soils on the northern approximately two-thirds of the site. This soil is well-suited to support the planned buildings; conventional foundations can be used. It will be necessary to remove the fill (if present), topsoil, and loose, weathered silt from within the foundation areas. This includes any thickened slab areas. Considering the depth to the bearing soils, the use of thickened slabs to support walls, doors, etc.

may be problematic, as it is difficult to ensure the bearing soil remains adequately compacted following the installation of buried utilities within the slab areas. Once the coarse-grained soils are reached, they should be recompacted using a hoe-pack. If the bearing soils are below the planned bottom-of-footing elevations, structural fill consisting of compacted, coarse-grained soil could be placed to reach the footing grades. Requirements for overexcavation widths and compaction of structural fill beneath footings and slabs are discussed below in ***Conventional Foundations*** and ***General Earthwork and Structural Fill***. It will be important that the project geotechnical engineer verifies that suitable bearing soils have been encountered before placing any structural fill beneath foundations.

The southern approximately one-third of the site is underlain by uncontrolled fill and unconsolidated alluvium. These soils are not suitable to support structures or settlement-sensitive elements, such as rigid retaining walls. This has been considered by the project team in preparing the site development layout. All structures will be located within in the area of competent soils. Fill will be placed to raise the grades on this southern portion of the site, requiring retaining walls along the downslope edges. As discussed previously, these retaining wall(s) would best be constructed using geogrid-reinforced modular block walls, which are flexible and can better tolerate settlement. A formal design for these walls will need to be completed once the site layout and final grades have been determined. The following are some considerations for the filled walls to include in project planning:

1. The lowermost walls can be placed over the existing fill soils, but should be supported on a minimum approximately 2-foot-thick layer of coarse-grained structural fill to provide proper support.
2. The fill placed behind the walls will need to consist of coarse-grained, compacted structural fill that is free-draining. The coarse-grained soil found under the northern portion of the site would be suitable for this purpose.
3. The walls will have to be designed to support surcharge loads, such as from upper tiered walls and vehicles.
4. A guardrail may need to be installed around the edges of the parking area, and the lateral loads from such a guardrail would need to be incorporated into the wall design.
5. Underground utilities, such as pipes, catchbasins, etc. cannot be located within the geogrid-reinforced zone behind the walls.
6. The walls will have to be designed for seismic conditions.
7. It would be prudent to construct the walls and complete the backfilling several months before installing utilities or pavements. This will allow the underlying loose soils to compress under the weight of the new fill.

Based on the results of our explorations, there are no geologically critical areas (landslide hazards, steep slopes, seismic hazards, or erosion hazards) within the planned development area. The slopes on, and around, the site are less than 10 feet in height and are manmade. No mitigation measures, such as buffers, necessary to address geologically critical areas for this project.

The existing fill, topsoil, and weathered silt are not reusable as compacted fill in structural areas (under foundations, slabs, pavements, etc.), as they cannot be adequately recompacted to avoid excessive post-construction settlement. It should be possible to reuse the underlying coarse-grained soils as structural fill under footings, floor slabs and other on-grade elements, such as sidewalks. The maximum thickness of compacted fill lifts will vary with the compaction equipment used, and the type of soil. However, in general, loose lift thicknesses for compacted fill should not exceed 12 inches. The pavement areas should be underlain by at least 12 inches of compacted outwash soil or imported granular fill placed over firm soils. It would be prudent to install the rock-covered construction entrance and staging areas where pavements will be placed, in order to provide a well-compacted pavement subgrade.

Our assessment of the potential for using subsurface infiltration on the site to dispose of runoff from impervious surfaces is discussed below in ***Subsurface Infiltration Design Information***.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered. Silty runoff cannot be allowed to flow off the site. We anticipate that a silt fence will be needed around the downslope sides of any cleared areas. Existing ground cover should be left in place wherever possible to minimize the amount of exposed soil. Rocked staging areas and construction access roads should be provided to reduce the amount of soil or mud carried off the property by trucks and equipment. Wherever possible, the access roads should follow the alignment of planned pavements. Trucks should not be allowed to drive off of the rock-covered areas. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Following clearing or rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface. On most construction projects, it is necessary to periodically maintain or modify temporary erosion control measures to address specific site and weather conditions.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking and bathing. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The designer or architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a build up of excessive water vapor within the planned structure.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

We recommend including this report, in its entirety, in the project contract documents. This report should also be provided to any future property owners so they will be aware of our findings and recommendations.

SEISMIC CONSIDERATIONS

In accordance with the 2018 International Building Code (IBC), within the planned development area on the northern two-thirds of the property, the site class within 100 feet of the ground surface is best represented by Site Class Type D (stiff soil). As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.17g and 0.41g, respectively.

The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period).

While the fine grained alluvial soils found on the southern portion of the site are potentially liquefiable below the seasonal water table, the coarse-grained soils that will underlie the planned building areas on the north portion of the site are not liquefiable.

CONVENTIONAL FOUNDATIONS

We recommend that continuous and individual spread footings have minimum widths of 16 and 24 inches, respectively. Exterior footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface for protection against frost and erosion. The local building codes should be reviewed to determine if different footing widths or embedment depths are required. Footing subgrades must be cleaned of loose or disturbed soil prior to pouring concrete. Depending upon site and equipment constraints, this may require removing the disturbed soil by hand.

The native silt soils are susceptible to softening under foot traffic in wet conditions. If the foundation excavation occurs during the wet season, or if seepage is encountered, it would be prudent to protect the excavated bearing surfaces with several inches of clean crushed rock.

An allowable bearing pressure of 2,500 pounds per square foot (psf) is appropriate for footings supported on medium-dense to dense, coarse-grained soils, or on properly-placed structural fil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil will be about one-inch, with differential settlements on the order of one-half-inch in a distance of 30 feet along a continuous footing with a uniform load.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level, well-compacted fill. We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.45
Passive Earth Pressure	300 pcf

Where: pcf is Pounds per Cubic Foot, and Passive Earth Pressure is computed using the Equivalent Fluid Density.

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend maintaining a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate values.

SLABS-ON-GRADE

The building floors can be constructed as slabs-on-grade atop competent native soil, or on structural fill. Existing fill and topsoil must first be removed. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill.

Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. This can affect moisture-sensitive flooring, cause imperfections or damage to the slab, or simply allow excessive water vapor into the space above the slab. All interior slabs-on-grade should be underlain by a capillary break drainage layer consisting of a minimum 4-inch thickness of clean gravel or crushed rock that has a fines content (percent passing the No. 200 sieve) of less than 3 percent and a sand content (percent passing the No. 4 sieve) of no more than 10 percent. Pea gravel or crushed rock are typically used for this layer.

As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI also notes that vapor *retarders* such as 6-mil plastic sheeting have been used in the past, but are now recommending a minimum 10-mil thickness for better durability and long term performance. A vapor retarder is defined as a material with a permeance of less than 0.3 perms, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where vapor retarders are used under slabs, their edges should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection.

If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.01 perms when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

FOUNDATION AND RETAINING WALLS

Retaining or foundation walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain <u>level</u> backfill:

PARAMETER	VALUE
Lateral Earth Pressure *	40 pcf
Passive Earth Pressure	300 pcf
Coefficient of Friction	0.45
Soil Unit Weight	135 pcf

Where: pcf is Pounds per Cubic Foot, and Lateral and Passive Earth Pressures are computed using the Equivalent Fluid Pressures.

*** For a restrained wall, such as a vault wall, that cannot deflect at least 0.002 times its height, an at-rest lateral earth pressure of 55 pcf should be used. This applies only to walls with level backfill, and does not include any surcharges, such as from vehicles or outriggers.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above lateral fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired.

The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized the wall and reinforcing design for a distance of 1.5 times the wall height from corners or bends in the walls, or from other points of restraint. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

Per IBC Section 1803.5.12, a seismic surcharge load need only be considered in the design of walls over 6 feet in height. A seismic surcharge load would be imposed by adding a uniform lateral pressure to the above-recommended lateral pressure. The recommended seismic surcharge pressure for this project is 8**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. The later section entitled ***Drainage Considerations*** should also be reviewed for recommendations related to subsurface drainage behind foundation and retaining walls.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. Also, subsurface drainage systems are not intended to handle large volumes of water from surface runoff. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls at one to 2 percent to reduce the potential for surface water to percolate into the backfill.

Water percolating through pervious surfaces (pavers, gravel, permeable pavement, etc.) must also be prevented from flowing toward walls or into the backfill zone. Foundation drainage and waterproofing systems are not intended to handle large volumes of infiltrated water. The compacted subgrade below pervious surfaces and any associated drainage layer should therefore be sloped away. Alternatively, a membrane and subsurface collection system could be provided below a pervious surface.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The recommended wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled ***General Earthwork and Structural Fill*** contains additional recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing, and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a buildup of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact an experienced envelope consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

DRAINAGE CONSIDERATIONS

Footing drains should be used where: (1) Crawl spaces or basements will be below a structure; (2) A slab is below the outside grade; or, (3) The outside grade does not slope downward from a building. Drains should also be placed at the base of all earth-retaining walls. Footing drains are not needed if these conditions do not apply.

Footing drains should consist of 4-inch-diameter perforated pipes surrounded by at least 4 inches of 1-inch-minus, washed rock that is encircled with non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the bottom of a slab floor or the level of a crawl space. The discharge pipe for subsurface drains should be sloped for flow to the outlet point. Roof and surface water drains must not discharge into the foundation drain system. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains. A typical footing drain detail is attached to the end of this report.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing even a few inches of free draining gravel underneath the vapor retarder limits the potential for seepage to build up on top of the vapor retarder.

No near-surface groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to a building should slope away at least 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls. A discussion of grading and drainage related to pervious surfaces near walls and structures is contained in the ***Foundation and Retaining Walls*** section.

SUBSURFACE INFILTRATION DESIGN INFORMATION

The test pits conducted on the northern portion of the site found highly-permeable, coarse-grained soils below a layer of weathered silt that has a low permeability. The coarse-grained soil is suitable for infiltration of stormwater. We completed a grain size analysis of a representative sample of the coarse-grained soil encountered retrieved from the test pits. A copy of the grain size test results is attached.

The coarse-grained soil would best be described as a sand in accordance with the *Textural Triangle U.S.D.A*. The fines (silt and clay) content of the soil is less than 5 percent.

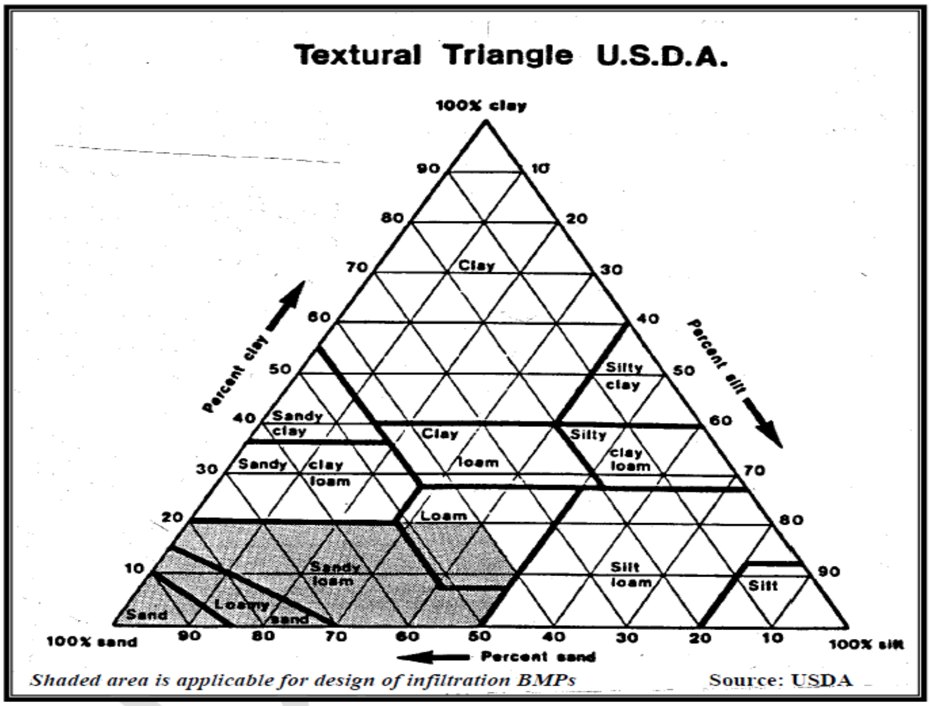


Figure A.1. USDA Textural Triangle.

Table A.1. Recommended Infiltration Rates Based on USDA Soil Textural Classification.

	Short-Term Infiltration Rate (in./hr)[1]	Correction Factor CF	Estimated Design (Long-term) Infiltration Rate (in./hr)
Clean sandy gravels and gravelly sands (i.e., 90% of the total soil sample is retained in the US #10 sieve)	20	2	10
Sand	8	4	2
Loamy Sand	2	4	0.5
Sandy Loam	1	4	0.25
Loam	0.5	4	0.13

Source: Stormwater Management Manual for Western Washington (Ecology 2005).
[1] From WEF/ASCE (1998).

Based on the observations made during the test pit excavations, a Long-Term Design Infiltration Rate of 2 inches/hour is conservative for the highly-permeable coarse-grained soils. The actual Long-Term Rate likely is closer to 10 inches/hour.

Formal infiltration testing should not be necessary to verify the recommended 2 inches/hour design infiltration rate.

All infiltration facilities in the area explored should be excavated through the topsoil, heavily-weathered soil, until the coarse-grained soil is exposed.

As with any infiltration system, the long-term performance will be adversely affected by silt and debris carried along with runoff. In order to prolong the effective life of an infiltration system, it is important to regularly clean and maintain the gutters, downspouts, and/or catch basins that feed into the infiltration system.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. It is important that existing foundations be removed before site development. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, behind permanent retaining or foundation walls, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction. The following table presents recommended relative compactions for structural fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

The potential for reuse of on-site soils as structural fill is discussed in the ***General*** section. Structural fill that will be placed in wet weather should consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the test pits are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of Milt Smith and Associates Construction and their representatives, for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with our understanding of current local standards of practice, and within the scope of our services. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

During the construction phase, we will provide geotechnical observation and testing services when requested by you or your representatives. Please be aware that we can only document site work we actually observe. It is still the responsibility of your contractor or on-site construction team to verify that our recommendations are being followed, whether we are present at the site or not.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plate 3 - 6	Test Pit Logs
Plate 7	Grain Size Analysis
Plate 8	Typical Footing Drain

We appreciate the opportunity to be of service on this project. Please contact us if you have any questions, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

2/22/2023

Marc R. McGinnis, P.E.
Principal

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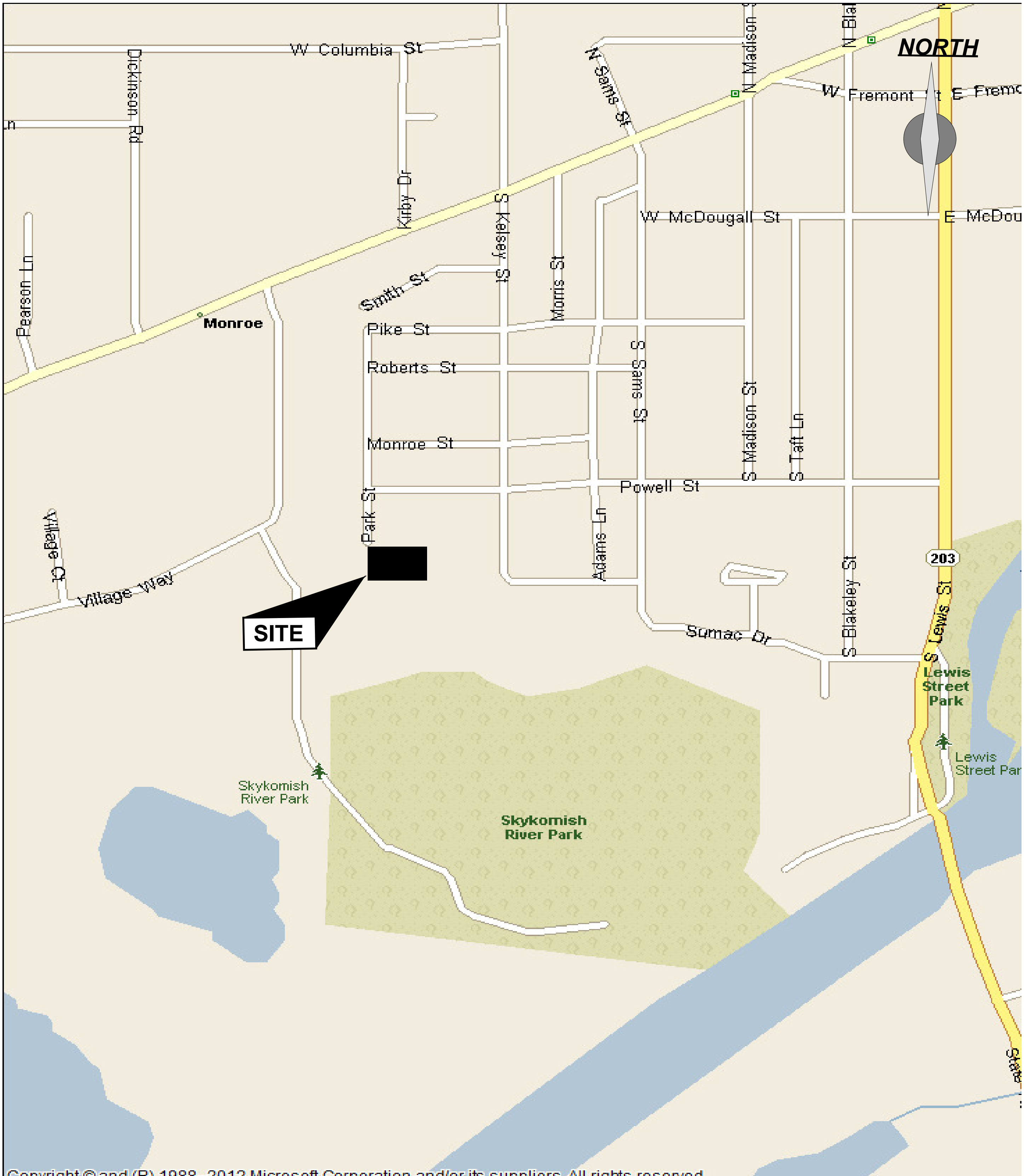
MRM:kg



(Source: Microsoft MapPoint, 2013)

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VICINITY MAP

523 Park Street
Monroe, Washington

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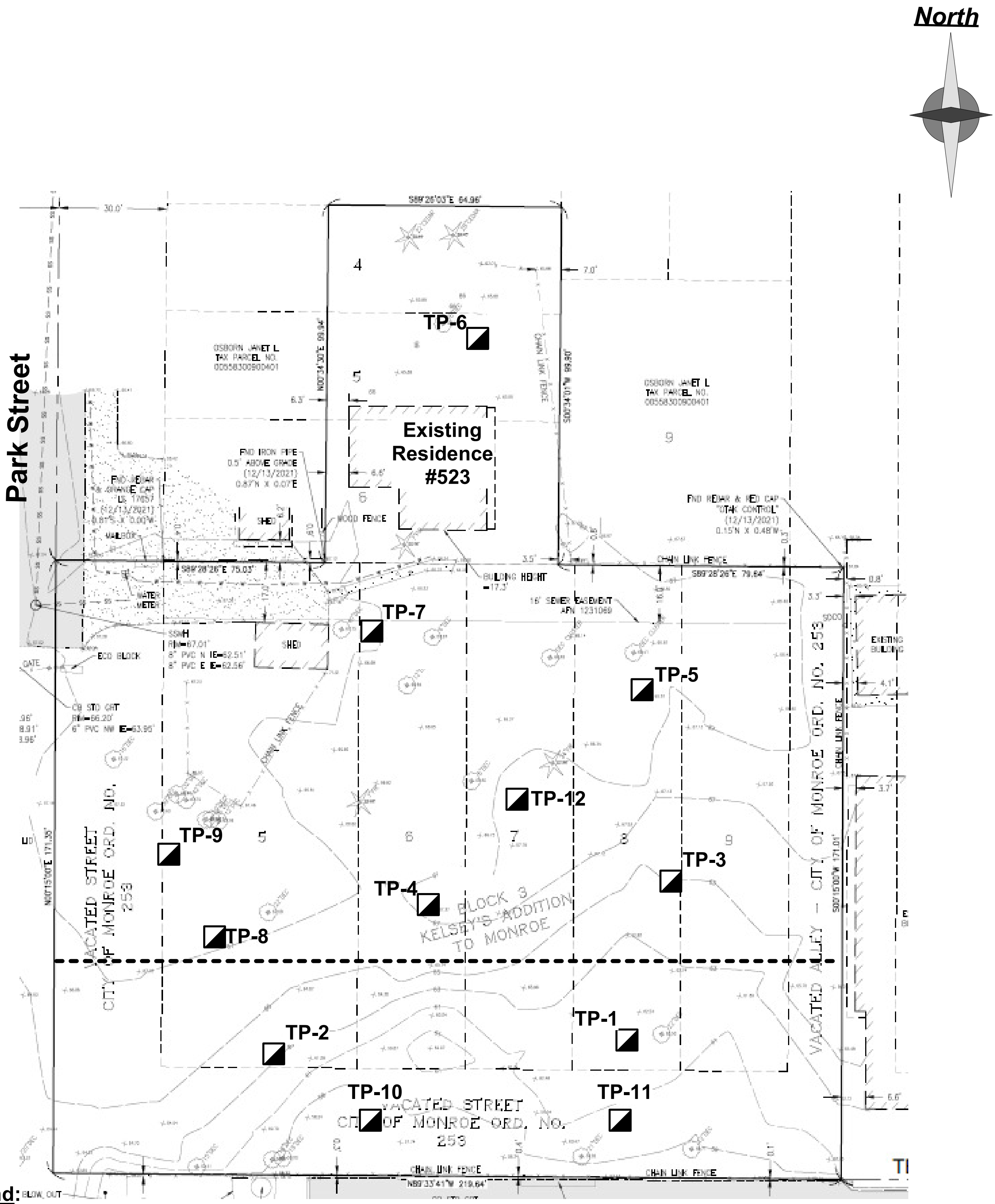

Legend:

Test Pit Location

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SITE EXPLORATION PLAN

523 Park Street
Monroe, Washington

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TEST PIT 1

Depth (feet)	Soil Description
0 – 0.5	Topsoil
0.5 – 7.5	Brown, gravelly, silty SAND with wood debris and numerous chunks of concrete up to 2-feet in size, fine-grained, very moist, medium-dense (FILL)
7.5 – 9.0	Orangish-brown, slightly gravelly, very silty SAND and sandy SILT, moist, loose (Alluvium)

Test Pit was terminated at a depth of 9.0 feet on June 23, 2022.
No groundwater seepage was observed.

TEST PIT 2

Depth (feet)	Soil Description
0 – 0.5	Topsoil
0.5 – 6.0	Brown, slightly gravelly, silty SAND with some small chunks of concrete, fine-grained, very moist, medium-dense (FILL)
6.0 – 9.5	Brownish-gray, silty SAND with pieces of wood, plastic, and carpet, fine-grained, moist, loose (FILL)
9.5 – 10.5	Orangish-brown, sandy SILT, non-plastic, very moist, loose (Alluvium)
10.5 – 12.0	Tan SILT with fine rootlets, non-plastic, very moist, loose (Alluvium)

Test Pit was terminated at a depth of 12.0 feet on June 23, 2022.
No groundwater seepage was observed.

TEST PIT 3

Depth (feet)	Soil Description
0 – 0.5	Topsoil
0.5 – 2.5	Orangish-brown, slightly gravelly, sandy SILT, non-plastic, very moist, loose
2.5 – 9.0	Brown, very gravelly SAND with cobbles, coarse-grained, moist, dense -becomes grayish-brown in color at 5 feet

Test Pit was terminated at a depth of 9.0 feet on June 23, 2022.
No groundwater seepage was observed.

TEST PIT 4

Depth (feet)	Soil Description
0 – 0.5	Topsoil
0.5 – 3.5	Orangish-brown, slightly gravelly, sandy SILT, non-plastic, very moist, loose -becomes tan at 2 feet
3.5 – 9.5	Brown, very gravelly SAND with cobbles, coarse-grained, moist, dense

Test Pit was terminated at a depth of 9.5 feet on June 23, 2022.
No groundwater seepage was observed.

TEST PIT 5

Depth (feet)	Soil Description
0 – 0.5	Topsoil
0.5 – 3.0	Orangish-brown, slightly gravelly, sandy SILT, non-plastic, very moist, loose -becomes tan at 2 feet
3.0 – 9.5	Brown, very gravelly SAND with cobbles, coarse-grained, moist, dense -becomes grayish-brown at 5.5 feet

Test Pit was terminated at a depth of 9.5 feet on June 23, 2022.
No groundwater seepage was observed.

TEST PIT 6

Depth (feet)	Soil Description
0 – 0.75	Topsoil
0.5 – 3.0	Orangish-brown, slightly gravelly, sandy SILT, non-plastic, very moist, loose
3.0 – 9.5	Brown, very gravelly SAND with cobbles, coarse-grained, moist, dense -becomes grayish-brown at 6 feet

Test Pit was terminated at a depth of 9.5 feet on June 23, 2022.
No groundwater seepage was observed.

TEST PIT 7

Depth (feet)	Soil Description
0 – 0.75	Topsoil
0.5 – 3.5	Orangish-brown, slightly gravelly, sandy SILT, non-plastic, very moist, loose
3.5 – 5.0	Brown, very gravelly SAND with cobbles, coarse-grained, moist, dense

Test Pit was terminated at a depth of 5.0 feet on June 23, 2022.
No groundwater seepage was observed.

TEST PIT 8

Depth (feet)	Soil Description
0 – 0.5	Topsoil
0.5 – 6.5	Brown, slightly gravelly, silty SAND with roots, fine-grained, damp, loose (FILL) -layer of crushed rock between 3 and 4 feet
6.5 – 7.5	Orangish-brown, slightly silty, gravelly SAND, fine- to coarse-grained, damp, loose
7.5 – 10	Brown, very gravelly SAND with cobbles, medium- to coarse-grained, damp, dense

Test Pit was terminated at a depth of 10.0 feet on August 17, 2022.
No groundwater seepage was observed.

TEST PIT 9

Depth (feet)	Soil Description
0 – 1.0	Brown, slightly gravelly, silty SAND with roots, fine-grained, damp, loose (FILL)
1.0 – 2.0	Topsoil
2.0 – 3.0	Orangish-brown, slightly silty, gravelly SAND, fine- to coarse-grained, damp, loose
3 – 5.0	Brown, very gravelly SAND with cobbles, medium- to coarse-grained, damp, dense

Test Pit was terminated at a depth of 5.0 feet on August 17, 2022.
No groundwater seepage was observed.

TEST PIT 10

Depth (feet)	Soil Description
0 – 0.5	Topsoil
0.5 – 4.0	Brown, slightly gravelly, silty SAND with some small chunks of concrete, fine-grained, very moist, medium-dense (FILL)
4.0 – 5.0	Old Topsoil
5.0 – 11.0	Orangish-brown, sandy SILT, non-plastic, very moist, loose (Alluvium) -becomes brownish-gray at 7 feet
11.0 – 13.0	Gray, oxidized SAND, medium- to fine-grained, very moist, loose -wet between 12 and 13 feet
13.0 – 15.0	Dark gray SILT with organics and fine rootlets, non-plastic, very moist, loose (Alluvium)

Test Pit was terminated at a depth of 15.0 feet on August 17, 2022.
Perched groundwater seepage was observed between 12 and 13 feet.

TEST PIT 11

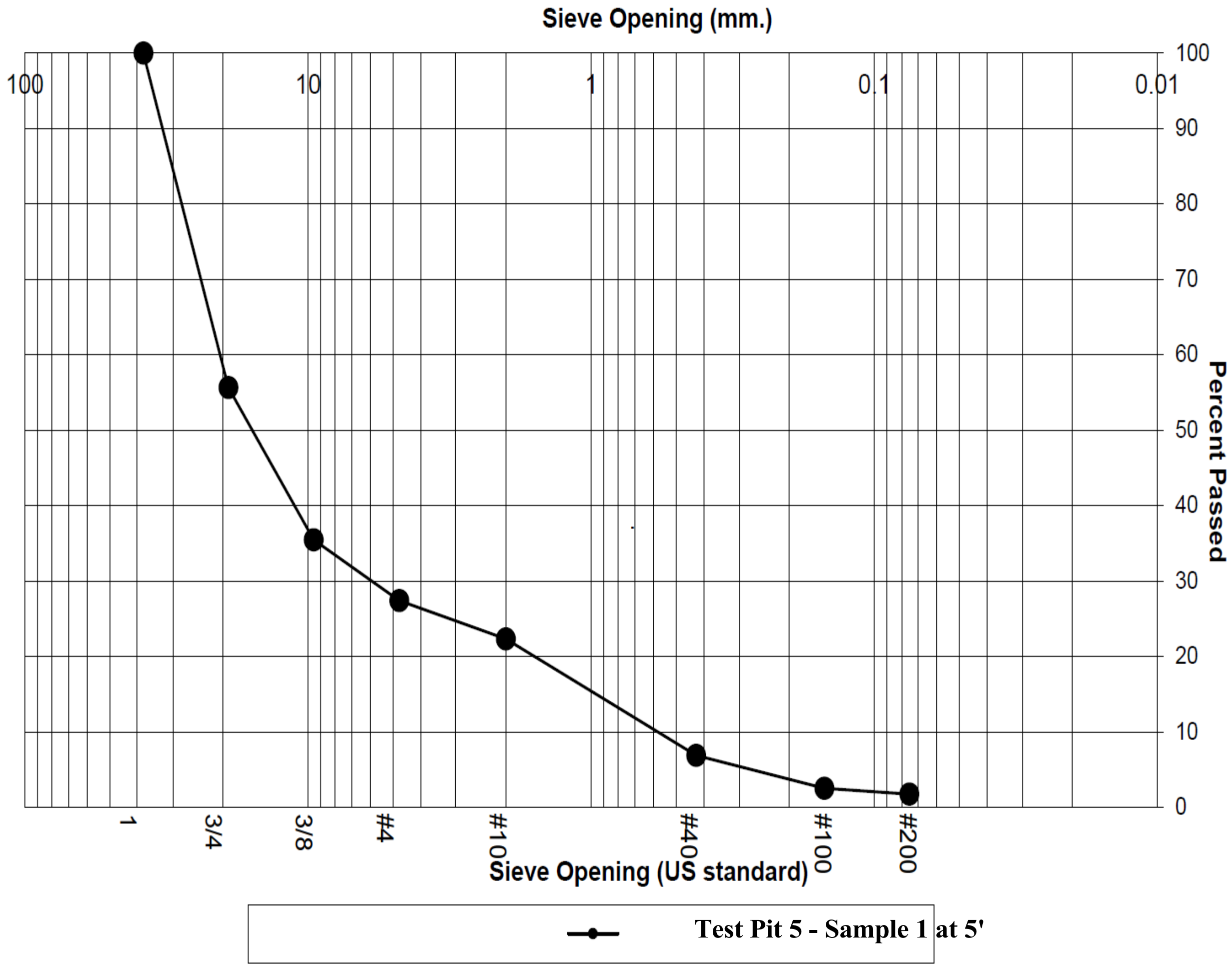
Depth (feet)	Soil Description
0 – 0.5	Topsoil
0.5 – 8.0	Brown, slightly gravelly, silty SAND with some small chunks of concrete, fine-grained, very moist, medium-dense (FILL)
8.0 – 9.0	Old Topsoil
9.0 – 17.0	Tan, sandy SILT with organics and fine rootlets, non-plastic, very moist, loose (Alluvium) -becomes brownish-gray at 14 feet -heavily oxidized from 15 to 16 feet

Test Pit was terminated at a depth of 17.0 feet on August 17, 2022.
No groundwater seepage was observed.

TEST PIT 12

Depth (feet)	Soil Description
0 – 1.0	Topsoil
1.0 – 4.0	Orangish-brown, slightly gravelly, silty SAND, fine-grained, damp, loose
4.0 – 15.0	Brown, very gravelly SAND with cobbles, medium- to coarse-grained, damp, dense -becomes gray at 8.0 feet -becomes very moist at 11.0 feet -becomes wet at 14.0 feet (Seasonal Groundwater level?)

Test Pit was terminated at a depth of 15.0 feet on August 17, 2022.
No groundwater seepage was observed.



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GRAIN SIZE ANALYSIS

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Monroe, Washington

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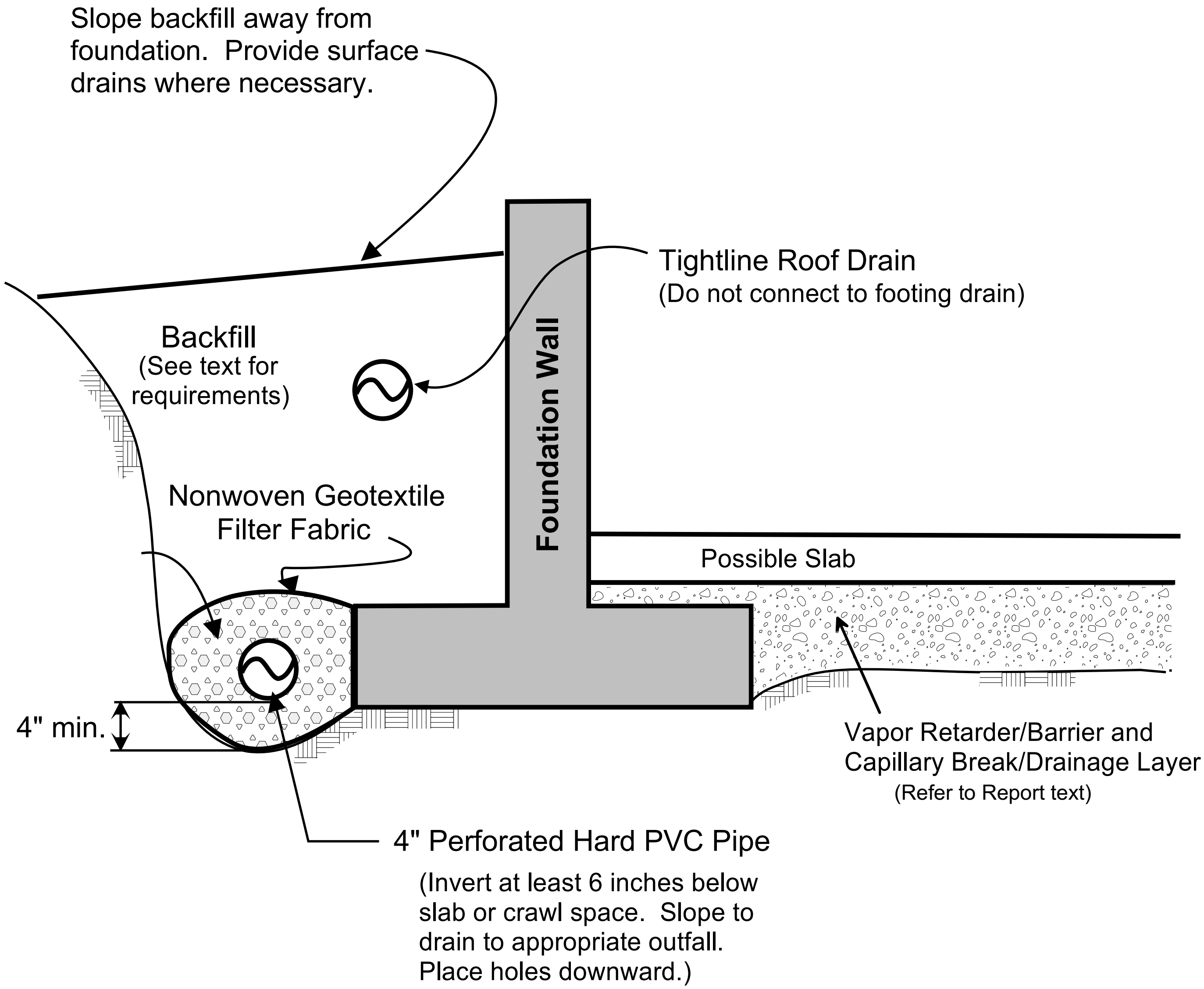

NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

(2) Refer to report text for additional drainage, waterproofing, and slab considerations.

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FOOTING DRAIN DETAIL

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Monroe, Washington

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Tab 8.0

8.0 OTHER PERMITS

This section will be completed during final engineering.

Tab 9.0

9.0 OPERATIONS AND MAINTENANCE MANUAL

This section will be completed during final engineering.

Tab 10.0

10.0 BOND QUANTITIES WORKSHEET

This section will be completed during final engineering.