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GEOTECHNICAL ENGINEERING REPORT

Vancouver Water Station 14 PFAS Treatment Design

VANCOUVER, WA

July 2024

Shannon & Wilson No: 112510

SHANNON & WILSON

Submitted To: Brown and Caldwell, Inc.
6500 S Macadam Avenue Suite 200
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Attn: Lynn Stephens

Subject: GEOTECHNICAL ENGINEERING REPORT, VANCOUVER WATER STATION 14 PFAS TREATMENT DESIGN, VANCOUVER, WASHINGTON

Shannon & Wilson, Inc. (Shannon & Wilson) prepared this report and participated in this project as a subconsultant to Brown and Caldwell, Inc (Brown and Caldwell). Our scope of services was specified in our Task Order Authorization with Brown and Caldwell dated October 1, 2023. This report presents our geotechnical engineering recommendations and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON

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ACRONYMS

ACRONYMS

ASCE	American Society of Civil Engineering
ASTM	American Society for Testing and Materials
bgs	Below Existing Ground Surface
CSZ	Cascadia Subduction Zone
GAC	Granular Activated Carbon
IBC	International Building Code
MCE	Maximum Considered Earthquake
pci	pounds per cubic inch
pcf	pounds per cubic foot
PFAS	Per- and Polyfluoroalkyl substances
PGA	Peak Ground Acceleration
psf	Pounds Per Square Foot
SPT	Standard Penetration Test
USGS	United States Geological Survey
VFD	Variable Frequency Drive
VWP	Vibrating Wire Piezometer
DCP	Dynamic Pone Penetrometer
Mm/yr	millimeters per year
Mw	moment magnitude
CSZ	Cascadia Subduction Zone
SPT	Standard Penetration Test
CMU	Cement Modular Unit
Mr	Resilient Modulus

1 INTRODUCTION

1.1 Project Understanding and Site Description

We understand the Vancouver Water Station 14 (WS14) PFAS Treatment Design project (the Project) is proposed to address water quality at WS14. In late 2020, per- and polyfluoroalkyl substances (PFAS) were first detected in the groundwater at the station. We understand the Project includes the preliminary design of an onsite PFAS treatment facility at WS14.

The proposed PFAS treatment facility will include three pairs (6 total) of granular activated carbon (GAC) treatment vessels supported on an at-grade structural slab foundation. The at-grade slab will have an approximate footprint of 114 feet by 32 feet and will have a top of slab elevation of 270.5 feet. The tallest point on the proposed GAC vessels will be on the order of 30 feet above ground and accordingly, the proposed GAC vessels are anticipated to have a fundamental period of vibration less than 0.5 seconds.

The proposed Model 12-40 GAC's are 12-feet in diameter and 26 feet and 9 inches in height. Each GAC vessel is specified to have a system operating weight of 385,000 pounds. Backwash waste from the GAC will be sent to an 80,000-gallon backwash tank north of the PFAS treatment facility. An approximate 70-foot by 25-foot infiltration area for backwash waste was originally considered immediately north of the backwash tank; however, this has been removed from the proposed upgrades.

Other site improvements include below grade, precast concrete vaults, and a new paved driveway to allow service trucks to access the new PFAS treatment facility and be able to complete a full turnaround. In addition, a new at-grade restroom/storage facility is proposed north of the backwash tank. The restroom/storage facility will have a finish floor elevation of 270.0 feet and we assume will have wall loads less than 1 kip per lineal foot. The site grading plan calls for a small interlocking block retaining wall (less than 4 feet in height) on the SW corner of the site as shown on Figure 2.

1.2 Site Description

WS14 is located southwest of the intersection of NE 78th Street and NE Andresen Road in Vancouver, Washington. The site currently consists of 3 groundwater wells (Wells #1 through #3) that feed into an aeration tower and subsequent wet well. A single-story treatment building is located in the southwest portion of the site which treats water pumped from the wet well via two VFD driven booster pumps. A paved driveway exists along the western margin of the site providing access to the treatment building and well houses.

A grove of large trees is located in the proposed PFAS treatment facility located east of Well #2 and the existing treatment building. An existing wetland area is north of the proposed PFAS treatment facility.

1.3 Scope of Services

Shannon & Wilson's services were conducted in general accordance with the scope of services specified in our Task Order Authorization with Brown and Caldwell dated October 1, 2023. The scope of services performed for this study consisted of the following tasks:

- Review existing information, including available geologic maps and previous geotechnical reports;
- Complete two borings to depths of 71.5 feet below ground surface (bgs);
- Complete two test pits to depths between 3.0 and 6.6 feet bgs;
- Complete two dynamic cone penetrometer (DCP) tests to support pavement design;
- Complete one infiltration test at a depth of 3.0 feet bgs;
- Conduct laboratory testing following the completion of the subsurface explorations;
- Provide code based seismic ground motion parameters in accordance with 2022 Oregon Structural Code and ASCE 7-16;
- Provide geotechnical recommendations for shallow foundation support of the new PFAS treatment facility, including estimates of allowable bearing capacity, total static and differential settlement;
- Provide recommendations for pavement design;
- Evaluate geologic hazards at the site including liquefaction, lateral spreading, and surface fault rupture;
- Provide construction considerations including subgrade preparation, reuse of on-site materials, and preparation and compaction of site fills and backfill; and
- Prepare this geotechnical engineering report.

2 GEOLOGIC SETTING

The geology of the project area has been extensively mapped (Wells and others, 2018, 2020, O'Connor and others, 2016) as Pleistocene (between 2.6 million to 11,000 years ago) catastrophic flood deposits. Holocene and Pleistocene (2.6 million years ago to present) Eolian deposits are also mapped within the project area.

The Eolian deposits are described as massive sand and silt deposits broadly covering upland areas. Most upland areas in the project vicinity are covered by variable thicknesses of eolian deposits, however, are difficult to distinguish from catastrophic flood deposits (O'Connor and others, 2016).

Pleistocene (between 2.6 million to 11,000 years ago) glacial-outburst flood deposits underlay Eolian deposits. The glacial flood deposits typically consist of sand and gravel sediments. The flood sediments were transported and deposited by as many as 40 or more very large, catastrophic floods that originated in western Montana. During the late stages of the last great ice age, between about 18,000 and 15,000 years ago, a lobe of the continental ice sheet repeatedly blocked and dammed the Clark Fork River in western Montana, which then formed an immense glacial lake called Lake Missoula. The lake grew until its depth was sufficient to buoyantly lift and rupture the ice dam, which allowed the entire massive lake to empty catastrophically. Once the lake had emptied, the ice sheet again gradually dammed the Clark Fork Valley and the lake refilled, leading to 40 or more repetitive outburst floods at intervals of decades (Allen and others, 2009). During each short-lived episode, floodwaters washed across the Idaho panhandle, through the eastern Washington scablands, and through the Columbia River Gorge. Because of constrictions in the drainage path, the floodwaters backed up and formed lakes into which sediments were deposited. One such lake covered parts of Clark County including the project site, and extended south to Eugene, Oregon. The flood deposits can be as thick as 200 feet (O'Connor and others, 2016).

2.1 Seismic Setting

The contemporary tectonics and seismicity of the region are the result of oblique, northeastward subduction at a rate of about 37 millimeters per year (mm/yr) (DeMets and others, 2010) of the Juan de Fuca oceanic plate beneath the North American continental plate (Wells and others, 1998; Wells and Simpson, 2001). This complex tectonic setting produces east-west compressive strain along the Cascadia Subduction Zone (CSZ), as well as northward translation and rotation of the mobile, crustal, Cascadia forearc blocks that span the leading edge of the North American plate (Wells and others, 1998; McCaffrey and others, 2007, 2013). Rotation of the Sierra-Nevada block and expansion of the Basin and Range drive the northward migration and clockwise rotation of the Cascadia forearc blocks (Pezzopane and Weldon, 1993; Wells and others, 1998; Wells and Simpson, 2001). As a result, the southern portion of the forearc, the Oregon Coast block, is impinging on western Washington and northwest Oregon at a rate of about 8 to 12 mm/yr causing crustal shortening in northwest Oregon and western Washington (Wells and others, 1998; Wells and Simpson, 2001; Mazzotti and others, 2002).

The combined effect of margin-normal subduction and margin-parallel shortening produces complex and diverse deformation within the northern edge of the Cascadia forearc and triggers large (greater than magnitude [M] 6), damaging earthquakes from three seismogenic source zones:

- The locked zone of the CSZ fault interface, which produces great mega-thrust earthquakes;
- The deep intraslab portion of the CSZ (i.e., the subducted portion of the Juan de Fuca Plate), the source of Wadati-Benioff zone earthquakes; and
- The overriding North American Plate, where shallow crustal faults rupture.

All three sources potentially produce earthquakes that impact the ground motion hazards at the project site. Offshore, elastic release of strain accumulated in the locked plate interface of the CSZ produces great megathrust earthquakes (greater than M 8.0) occurring at irregular intervals that span from about 100 to more than 1,200 years, with an average recurrence interval of about 300 to 500 years (Atwater and Hemphill-Haley, 1997; Clague, 1997; Goldfinger and others, 2003 and 2012); the most recent rupture occurred in A.D. 1700 (Satake and others, 1996; Atwater and Hemphill-Haley, 1997; Clague, 1997; Yamaguchi and others, 1997; Goldfinger and others, 2003 and 2012).

Onshore, migration and rotation of tectonic blocks produce deformation along shallow faults within the upper part of the crust. At depth, rupture within the subducting slab, referred to as the intraslab, has produced some of the largest recorded earthquakes (M 6.5 to 7) to strike the Pacific Northwest, the northern California Coast, and Western Washington. However, over the past century, intraslab earthquakes have been markedly infrequent in Oregon. The following sections briefly describe the location, characteristics, and seismicity of each of the sources.

2.1.1 Cascadia Subduction Zone: Mega-Thrust Interface Source

CSZ mega-thrust earthquakes originate along the interface between the subducting oceanic plates and the North American plate. Because of the significant uncertainty of the landward extent of a potential rupture surface, estimates of the closest distance between the project and potential rupture surface range from about 65 to 140 horizontal miles. Focal depths for mega-thrust earthquakes are commonly on the order of about 15 to 25 miles. Rupture of the interface could result in earthquakes with moment magnitudes (Mw) on the order of 8.5 to over 9.0, with strong shaking that lasts for several minutes. No large earthquakes have occurred in this zone during historic times (the last 170 years). However, geologic evidence suggests that coastal estuaries have experienced rapid subsidence at various times within the last 2,000 years (Atwater, 1987; Atwater and Hemphill-Haley, 1997) as a result of tectonic movement associated with mega-thrust earthquakes on the CSZ. It appears that ruptures of this zone have occurred at irregular intervals that span from about 100 to more than 1,200 years, with an average recurrence interval of about 300 to 500 years (Atwater and Hemphill-Haley, 1997). Based on historical tsunami records in Japan (Satake and others, 1996) the most recent interplate event on the CSZ was a Mw 9 event on January 26, 1700.

2.1.2 Cascadia Subduction Zone: Intraslab Source

CSZ intraslab earthquakes originate from within the subducting oceanic plates as a result of down-dip tensional forces and bending caused by mineralogical and density changes in the plates at depth. These earthquakes typically occur 28 to 37 miles beneath the surface. The nearest seismogenic intraslab portion of the Juan de Fuca plate is approximately 30 to 60 miles below the Portland area. Ludwin and others (1991) estimate that the maximum Mw from this source zone would be about 7.5. Ground shaking produced by intraplate earthquakes would be less intense and less prolonged in the Portland area than ground motions generated by large subduction zone interface earthquake events. Historic seismicity from this source zone includes the 1949 Mw 6.7 Olympia earthquake, the 1965 Mw 6.7 earthquake between Tacoma and Seattle, and the 2001 Mw 6.8 Nisqually earthquake. While intraslab events have occurred frequently in the Puget Sound area, they are historically rare in Oregon.

2.1.3 Shallow Crustal Source

Shallow crustal earthquakes within the North American Plate have historically occurred in a diffuse pattern within the Pacific Northwest, typically within the upper 4 to 19 miles of the continental crust. Mabey and others (1993) concluded from their analysis of local geologic features that a crustal earthquake of up to Mw 6.5 could occur virtually anywhere in the Portland area. Based on their fault model, Wong and others (2000) determined that an earthquake of up to Mw 6.8 is possible on the Portland Hills Fault, which is mapped within about 2.4 miles of the project site. The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades earthquake at approximate Mw 6.5 to 7.0. Other examples include the 1993 Mw 5.6 Scotts Mill earthquake and the 1993 Mw 6.0 Klamath Falls earthquake.

Faults and associated folds which demonstrate geological evidence of co-seismic surface deformation during the Quaternary time period in Oregon and Washington have been located and characterized by the United States Geological Survey (USGS). The USGS provides approximate fault locations and a detailed summary of available fault information in the USGS Quaternary Fault and Fold Database (2014). The database defines four categories of faults, Class A through D, based on evidence of tectonic movement known or presumed to be associated with large earthquakes during Quaternary time (within the last 2.6 million years). For Class A faults, geologic evidence demonstrates that a tectonic fault exists and that it has likely been active within the Quaternary period. For Class B faults, there is equivocal geologic evidence of Quaternary tectonic deformation, or the fault may not extend deep enough to be considered a source of significant earthquakes. Class C and D faults lack convincing geologic evidence of Quaternary tectonic deformation or have been studied carefully enough to determine that they are not likely to generate significant earthquakes.

According to the USGS Quaternary Fault and Fold Database (USGS, 2024), there are 6 Class A features within approximately 15 miles of the project site. Their names, general locations relative to the site, and the time since their most recent deformation are summarized in Exhibit 2-1. The CSZ itself is approximately 135 miles west of the project site, with an average slip rate of approximately 40 millimeters (1.5 inches) per year.

Exhibit 2-1: USGS Class A Faults Within an Approximate 15-Mile Radius of the Project Site

Fault Name	USGS Fault Number	Approximate Length	Approximate Distance and Direction from Project Site[1]	Slip Rate Category[2]	Time Since Last Deformation3
Lacamas Lake Fault	880	14.9 miles	4.3 miles W	< 0.2 mm/yr	< 750 ka
East Bank Fault	876	18.0 miles	9.4 miles SW	< 0.2 mm/yr	< 750 ka
Portland Hills Fault	877	30.4 miles	10.3 miles SW	< 0.2 mm/yr	< 1.6 Ma
Oatfield Fault	875	18.0 miles	12.7 miles SW	< 0.2 mm/yr	< 1.6 Ma
Grant Butte Fault	878	6.2 miles	13.5 miles SE	< 0.2 mm/yr	< 750 ka
Damascus-Tickle Creek Fault Zone	879	9.9 miles	14.3 miles SE	< 0.2 mm/yr	< 750 ka

NOTES:

1 Approximate distance between the project site and nearest extent of fault mapped at the ground surface.

2 mm = millimeters; yr = year.

3 Ma = “Mega-annum” or million years ago; ka = “Kilo-annum” or one thousand years ago.

3 SUBSURFACE EXPLORATION PROGRAM

Shannon & Wilson explored subsurface conditions at the project site with two geotechnical borings, designated B-1 and B-2, and two test pits, designated TP-1 and TP-2. Borings B-1 and B-2 were completed to a depth of 71.5 feet, between February 15 and February 16, 2024. Test pits TP-1 and TP-2 were completed to depths of 6.6 feet and 3.0 feet, respectively, on February 20, 2024. Infiltration testing was completed in TP-1 at a depth of 3.0 feet. To support pavement design, two dynamic cone penetration (DCP) tests, designated DCP-1, and DCP-2, were completed on February 20, 2024.

Approximate locations of the explorations are shown on the Site and Exploration Plan, Figure 2. Details of drilling, sampling procedures, and our log of the materials encountered are presented in Appendix A, Field Explorations.

4 LABORATORY TESTING

The samples we obtained during field explorations were transported to the Shannon & Wilson laboratory for further examination. Representative samples were selected for a suite of laboratory tests. The testing program included moisture content analyses, particle size analyses, and corrosivity testing. Testing was performed by Shannon & Wilson in accordance with applicable ASTM International standards or subcontracted to an outside laboratory. Results of the laboratory tests and brief descriptions of the test procedures are presented in Appendix B, Laboratory Testing.

4.1 Corrosivity Testing Results

Analytical testing was performed on soil samples collected from the subsurface exploration to evaluate the corrosivity potential of the soil at the project site. The results of the corrosivity testing were graded for potentially corrosive environments to ductile-iron pipes using the 10-point evaluation procedure (AWWA, 2009). The evaluation procedure is based on information from soil resistivity, pH, oxidation-reduction potential, sulfides, and moisture. Based on the 10-point evaluation procedure, the analyzed soils are not considered corrosive to ductile-iron pipes.

The Naval Facilities Engineering Command Design Manual 7.02, Soils and Foundation Design Manuals (NAVFAC, 1986) indicates that as it pertains to foundations, if analysis indicates the sulfate concentration to be more than 0.5 percent in soil, or more than 1200 ppm in groundwater the use of a sulfate resistant cement such as Type V Portland cement should be considered. Measured sulfate concentrations were less than 0.5 percent in the soil and therefore below the criteria for using sulfate resistant cement.

5 SUBSURFACE CONDITIONS

5.1 Geotechnical Units

Shannon & Wilson grouped the materials encountered in our field explorations into a single geotechnical unit, as described below. The soil classifications are presented on the boring log. The materials encountered are described on the boring logs in Appendix A. A partial summary of the geotechnical unit is as follows:

- **Catastrophic Flood Deposits**: Lean Clay with Sand (CL); Sandy Silty to Silt with Sand (ML); Sandy Silt to Silty Sand (ML/SM); Silty Sand (SM); Poorly Graded Sand with Silt to Silty Sand (SP-SM/SM); Poorly Graded Sand with Silt (SP-SM); and Poorly Graded Sand to Poorly Graded Sand with Silt (SP/SP-SM).

The geotechnical unit was grouped based on the engineering properties, geologic origin, and distribution in the subsurface. Contacts between the units may be more gradational than shown in the boring logs. The Standard Penetration Test (SPT) N-values are shown on the boring logs as recorded in the field (uncorrected).

5.2 Groundwater

Shallow perched, groundwater was observed seeping through the side walls of the test pits and ponding at the base of the excavation. Groundwater was not noted on the borings B-1 or B-2 which were advanced using mud rotary drilling techniques. Mud-rotary drilling techniques can make the depth to groundwater difficult and/or not possible to discern during drilling due to the introduction of drilling fluids into the borehole to flush the drill cuttings to the surface. A vibrating wire piezometer (VWP) was installed in boring B-1 to allow for ongoing groundwater level measurements. A summary of groundwater measurements is provided in Exhibit 5-1.

Exhibit 5-1: Depth to Groundwater Summary

Exploration (Method)	Date	Depth of Water (ft)
TP-1	2/20/2024	3.6[a]
TP-2	2/20/2024	2.0[a]
B-1	3/6/2024	52.4[b]
B-1	3/26/2024	51.3[b]
B-1	6/17/2024	51.4

NOTES:

1. Measurement from visual observation of seepage in test pit at time of exploration was conducted.
2. Measurement from vibrating wire piezometer (VWP)

Groundwater levels should be expected to vary with changes in topography and precipitation. Generally, groundwater highs occur at the end of the wet season in late spring or early summer, and groundwater lows occur towards the end of the dry season in the early to mid-fall.

Based on the observations of groundwater in the test pit explorations and measurements collected from VWP, we anticipate a shallow perched layer of groundwater is present in the upper 5 feet of the subsurface at the site.

5.3 Seismic Design Ground Motions

Our seismic hazard evaluation was performed in accordance with the criteria in the 2021 International Building Code (IBC). The 2021 IBC refers to ASCE's Minimum Design Loads for Buildings and Other Structures, 2016 Edition (ASCE 7-16) Chapter 20 for determination of Site Class. As discussed in Section 6.2, potentially liquefiable soils were encountered in the subsurface exploration program. Based on the potentially liquefiable soils encountered in our subsurface exploration, the site class assigned to the site is Site Class F.

As permitted by Chapter 20 of ASCE-16, site response analysis is not required to determine spectral accelerations for sites with potentially liquefiable soils (Site Class F) when the structure has a fundamental period of vibration equal to or less than 0.5 seconds. Provided the structure has a fundamental period of vibration equal to or less than 0.5 seconds, the site classification can be determined using ASCE 7-16 Table 20.3-1 and the site coefficients, F_a and F_v in ASCE 7-16 Tables 11.4-1 and 11.4-2. Our seismic analyses are based on the structure having a fundamental period of vibration of less than 0.5 seconds and a site response analysis is not required. Based on the Standard Penetration Test (SPT) N-values from the boring, the non-liquefied seismic site class for the site is Site Class D.

Based on S_s and S_1 values mapped at the site by ASCE 7-16, Section 11.4.8 states a site response analysis is required for the site; however, we note some exceptions may be valid. As permitted by Exception 1 in Section 11.4.8 of ASCE 7-16 Supplement 3, site response analysis is not required to determine spectral accelerations for a seismic Site Class D when the value of S_{M1}, as determined by ASCE 7-16 Equation 11.4-2, is increased by 50 percent for all applications of S_{M1}. Please note that S_{D1}, as determined by ASCE 7-16 Equation 11.4-4, will then be increased by 50 percent for all applications in ASCE 7-16.

We have provided seismic design ground motion parameters that are based on Exception 1 in Supplement 3 being valid (and that a site response analysis is not required. The provided USGS Code-Based MCE and Design Seismic Parameters for Site Class D are presented in Exhibit 5-2.

Exhibit 5-2: USGS Code-Based MCE and Design Seismic Parameters

Seismic Parameter	Value
Site Class	D
Mapped MCE Peak Ground Acceleration, PGA	0.363 g
PGA Site Coefficient, F_{PGA}	1.237
Peak Ground Acceleration Corrected for Site Effects, PGA_M	0.449 g
Mapped Short Period Spectral Acceleration, S_s	0.806 g
Mapped 1-Second Period Spectral Acceleration, S_1	0.367 g
Short Period Site Coefficient, F_a	1.178
1-Second Period Site Coefficient, F_v	1.933
Short Period Design Spectral Acceleration, S_{DS}	0.633 g
1-Second Period Design Spectral Acceleration, S_{D1}	0.709 g*

NOTES:

Due to the potentially liquefiable soils at the site, the seismic site class per ASCE 7-16 is F; however, the seismic parameters to determine base shear forces can be determined using the non-liquified seismic site class (Site Class D) provided the structure has a fundamental period of vibration less than 0.5 seconds per Chapter 20 of ASCE 7-16. The seismic parameters provided in Exhibit 5-2 are based on Site Class D.

g = acceleration due to gravity

* See Section 11.4.8 of ASCE 7-16 Supplement 3. Value of S_{D1} in Exhibit 5-2 has been increased by 50 percent as required by ASCE 7-16 Supplement 3.

5.4 Liquefaction

5.4.1 General

Soil liquefaction is a phenomenon in which excess pore water pressure of loose to medium dense, saturated, granular soils increases during ground shaking to a level near the initial effective stress. The increased excess pore pressure results in a reduction of soil shear strength. The effects of liquefaction typically include lateral spreading, slope instability, and ground settlement.

Soil behavior under seismic loading is the primary factor in determining the susceptibility of soil to liquefaction. Important factors in evaluating soil behavior are relative density, the fines content (percent of soil by weight smaller than 0.075 millimeters, passing the No. 200 sieve), and the plasticity characteristics of the fines. Relative density can be estimated from SPT N-values collected from the borings for this project.

Settlement may occur in cohesionless soil that undergoes liquefaction and pore pressure development during ground shaking. The settlement is related to densification and rearrangement of particles during ground shaking, as well as volume change as the excess pore pressure dissipates after ground shaking. Seismic ground settlement may not occur uniformly over an area, and differential settlement may impact existing or proposed structures supported by liquefied soil.

5.4.2 Liquefaction Analysis and Liquefaction-Induced Settlement

We evaluated liquefaction potential of the upper 70 feet of the site soils based on the SPT N-values collected from borings B-1 and B-2, results of laboratory testing, and groundwater measurements collected from the VWP installed in boring B-1. These collective data were evaluated according to the methods and procedures recommended by Boulanger and Idriss (2014). In addition, our analysis was performed using a peak ground acceleration (PGA_M) of 0.449g and a mean Moment Magnitude (M_w) of 7.8, which is based on the total seismic source contribution to the site. The mean M_w was determined by using USGS's online Unified Hazard Tool for a 2,475-year return period.

Liquefaction-induced settlement magnitudes were calculated using correlations developed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). Results of our liquefaction-induced settlement analysis indicate that total liquefaction-induced settlement will be up to 1.2 inches with the settlement occurring within a potentially liquefiable zone of soil between depths of 50 and 60 feet bgs. Exhibit 5-3 presents a summary of the liquefaction-induced volumetric strain calculated at boring locations B-1 and B-2.

Exhibit 5-3: Summary of Liquefaction-Induced Settlement at Depth

Boring Location	Approximate Depths of Liquefaction Susceptible Soils (feet)	Approximate Total Liquefaction-Induced Settlement (inch)
B-1	52 to 58	0.8 to 1.2
B-2	65 to 70	Up to 0.3

Ishihara and others (2016) provided an update to the procedures included in Ishihara and Yoshimine (1992) for estimating settlement triggered by liquefaction at the ground surface recommended procedure. In their recommendations, they state that volumetric strain estimated in loose liquefiable layers below a depth of 15 meters (49.2 feet) should not be counted in settlements manifesting at the ground surface. As the depth of the static groundwater table exceeds this depth, liquefaction-induced settlement that manifests at the ground surface is anticipated to be minimal.

5.5 Lateral Spreading

Based on the depth to groundwater and distance from existing free faces at the project site, we consider the risk of lateral spreading impacting the new facility to be low.

5.6 Fault Surface Rupture

As presented in Exhibit 2-1, the closest fault to the site is mapped approximately 4.3 miles to the west. Consequently, it is our opinion that the risk of surface fault rupture at the site is low during a design life of 100 years or less.

6 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

6.1 General

The key geotechnical design considerations for the site include settlement due to long-term loading and the seismic considerations. As discussed in Section 5.4.2, liquefaction-induced ground settlement at the ground surface is expected to be minimal given the distance to the groundwater table (and potentially liquefiable layers) beneath the ground surface.

6.2 Foundation Design – Tanks and Vessels

The proposed six Model 12-40 GAC's are 12-feet in diameter and 26 feet and 9 inches in height. Each GAC vessel will have a system operating weight of 391,000 pounds. We understand all six GAC vessels will be supported on a structural pad. Based on discussion with the structural engineer, we understand the average contact pressure at the base of the structural pad will be less than 3,000 psf.

We do not have information on the configuration of the 80,000-gallon backwash tank; however, we anticipate it will by cylindrical and will likely have a height and diameter on the order of 40 feet and 18 feet, respectively. Based on discussion with the structural engineer, we understand the backwash tank will have an average contact pressure less than 3,000 psf at its base.

Given the relatively high areal pressurebeneath the tanks and vessels, we recommend installing a minimum 36-inch-thick reinforced crushed rock mat beneath the tank and vessel foundations. Details regarding the reinforced crushed rock mat is presented in Section 7.4. For foundations bearing on the reinforced crushed rock mat section, a net allowable bearing pressure of 3,000 psf may be used for design. We further recommend evaluating the tank vessel foundations with a subgrade reaction modulus of 150 pci.

6.2.1 Tank and Vessel Settlement

Based on the above loading configurations and recommended implementation of reinforced crushed rock mats, we anticipate total settlement beneath the vessels and backwash tank will be up to 1 inch. Differential settlements across the vessels and tanks are expected to be up to 50 percent of the estimated total settlement.

6.2.2 Tank and Vessel Lateral Resistance

The nominal lateral sliding resistance at the base of the tank and vessel foundations can calculated by multiplying the vertical load by a coefficient of 0.4 for provided the foundation is underlain by the crushed rock mat described above in Section 6.2 and below in Section 7.4

6.3 Foundation Design – Restrooms/Storage Building

We understand the restroom/storage building will be consructed at-grade and will consist of cement modular unit (CMU) construction. According to ASCE 7-16 Table 12.13-3, single-story structures with concrete or masonry wall systems have differential settlement value limits of 1.2 inches over a distance of 50 feet for risk category IV structures. Wall loads for the restroom building are anticipated to be 1 kip per lineal foot or less.

The bottom of the foundation should be at least 18 inches below the lowest adjacent grade for frost protection. Based on a bottom of footing depth of 18 inches, the wall footings of the restroom/storage building are anticipated to be founded on loose to medium dense silty sand of the Catastrophic Flood Deposits unit. Accordingly, we recommend constructing a minimum 6-inch-thick crushed rock pad beneath the footings to help provide a more uniform surface and protect the subgrade from becoming disturbed during placement of rebar and formwork. The crushed rock pad should be compacted to a well-keyed, firm, and unyielding condition.

While not encountered in our explorations, if undocumented fill is encountered during construction, the undocumented fill should be removed from the foundation subgrade and be replaced with crushed rock. The crushed rock should extend 6 inches horizontally outward from the footing edge for every 1 foot of thickness (i.e., 0.5H:1V outward projection from edge of foundation). The crushed rock should be compacted to not less than 93percent of the maximum dry density, as determined by ASTM D1557.

6.3.1 Allowable Bearing Capacity – Restroom/Storage Building

We recommend an allowable bearing capacity of 2,000 pounds per square foot (psf) be used to design the building foundations provided the minimum 6-inch-thick crushed rock pad is incorporated into the design. This allowable bearing capacity can be increased by up to one-third for short-term loads resulting from wind or seismic forces. We recommend the wall footings have a minimum width of 18 inches. If the wall foundation is considered to be flexible, a subgrade reaction modulus of 125 pounds per cubic inch (pci) may also be used in design.

6.3.2 Building Foundation Settlement

Based on the above allowable bearing capacity of 2,000 psf and with the understanding that the wall loads will be less than 1 kips per lineal foot, we anticipate total settlement beneath the wall footings will be less than 1 inch. Differential settlement is anticipated to be less than 50 percent of the total settlement which will be less that the threshold provided in ASCE 7-16 Table 12.13-3 for single-story structures with concrete or masonry wall systems.

6.3.3 Foundation Lateral Resistance – Restroom/Storage Building

The soil resistance available to withstand lateral foundation loads is a function of the frictional resistance, which can develop on the base of the foundation, and the partial soil passive resistance, which is assumed to be about 50 percent of the full soil passive resistance. The nominal lateral sliding resistance at the base of the footing can calculated by multiplying the vertical load by a coefficient of 0.4 for footings underlain by a minimum of 6 inches of crushed rock.

A nominal partial passive fluid pressure of 200 pcf may be also used for design. Passive resistance should be ignored in the upper 12 inches as construction and post-construction activities often disturb this upper material and it is susceptible to frost penetration.

6.4 Lateral Earth Pressures on Buried Structures and Retaining Walls

6.4.1 Static and Seismic Lateral Earth Pressures

We recommend that the embedded walls of below grade vaults and retaining walls less than 4 feet tall be evaluated as non-yielding walls by applying a static at-rest equivalent fluid earth pressure equal to 56H (where H is the height of the wall retaining soil) which is based on level retained material.

We understand a interlocking block retaining wall will support a backslope with inclinations up to 3.3H:1V. For wall locations with a maximum backslope of 3H:1V, we recommend evaluating the retaining wall with a static at-rest equivalent fluid earth pressure of 73H (where H is the height of the retained soil). These static at-rest earth pressures should be evaluated as a triangular distribution.

Section 1807.2.2 of the 2021 International Building Code (currently adopted by City of Vancouver at the time of this report) requires retaining walls supporting retained material up to 6 feet in height be designed to account for seismic lateral earth pressure. Evaluation of seismic loading should be performed by applying an additional uniform seismic earth pressure of 25H on portions of the wall retaining soil. The additional seismic earth pressure should be evaluated as a uniform distributed pressure increase. This seismic earth pressure should be applied with the static at-rest pressure value. Additional surcharge pressures may be appropriate for evaluation if dead of live loads, such as traffic loads, are anticipated to be present.

6.4.2 Embedded Wall Drainage

Hydrostatic pressures are not included in the equivalent fluid pressures above as the long-term static groundwater is anticipated to be below the base of the embedded portions of the structures; however, we note that perched groundwater conditions can develop within a few feet of the ground surface at the site. To prevent the buildup of hydrostatic pressures on the vaults from perched water, we recommend that any vault structure with a height greater than 4 feet be backfilled with a minimum 12-inch-wide zone of free-draining crushed rock material (i.e., 1 1/2-inch minus with less than 3 percent passing the No 200 sieve). To protect this 12-inch-wide zone of crushed rock, we recommend placing a non-woven geotextile (such as Mirafi 140 N) along the contacts of the crushed rock zone with the adjacent and overlying soil. A drain should be included at the base of the crushed rock zone to reroute collected water to a suitable discharge point.

6.4.3 Surcharge Loading

Surcharge loads, if applicable, should be added to static and seismic lateral earth pressures. If vehicle traffic will be operating near the walls, then typically a 250 psf vertical surcharge load is applied as a lateral uniform pressure equal to 118 psf. If construction traffic surcharge pressures in excess of 250 psf will be applied at the ground surface adjacent to the buried building wall during or after construction, we recommend that those surcharge pressures be modeled as a uniform rectangular pressure acting on the face of the non-yielding wall as 0.47q, where q is the surcharge load as shown in Figure 3. Other surcharge loading conditions should be evaluated, as applicable, with the use of the general equations and diagrams shown on Figure 4.

6.4.4 Lateral Resistance

Lateral resistance of buried vault structures can be obtained through partial passive pressure developed along the buried portions of the structures. Provided the structures are backfilled with crushed rock compacted to at least 95 percent of the maximum dry density

(ASTM D1557), passive resistance on buried foundations can be calculated based on granular backfill material with a friction angle of 34 degrees, a unit weight of 120 pounds per cubic foot (pcf), and the equivalent fluid lateral pressures in Exhibit 6-1. Note, the full passive value is based on lateral movement equal to up to 6 percent of the wall height occurring before the full passive resistance is reached.

Exhibit 6-1: Passive Equivalent Fluid Pressures

Coefficient	Full Passive Value (psf)	Partial Passive Value (psf)
Passive	350H	175H

H = embedded height of structure (feet)

For estimating the lateral resistance contribution from the partial passive pressure, we recommend ignoring the upper 1-foot of adjacent backfill or soil due to potential disturbance resulting from post-construction activities and frost penetration.

6.5 Pavement Design

We understand that the proposed pavement will be subjected to traffic loading from passenger vehicles and occasional chemical trucks during normal operations. The pavement design was performed in accordance with the recommended procedures and guidelines in the 1993 AASHTO Guide for Design of Pavement Structures (AASHTO, 1993). The pavement section thicknesses recommended below can accommodate up to 100,000 Equivalent Single Axle Loads ESALs over a 20-year design life, which correlates to average annual daily traffic of approximately 3 to 7 passes from a 2-axle freight truck. The traffic conditions were discussed with the project team and were confirmed to be a suitable minimum loading condition for the proposed pavement. However, the project team indicated that a more conservative estimate would be 120,000 ESALs over a 20-year design life. The increase from 100,000 to 120,000 ESALs is a relatively small increase in loading for pavement and does not significantly increase the pavement thickness. Although, for comparison we have presented pavement thicknesses for both loading conditions presented in Exhibit 6-2 below.

Based on the results from the DCP testing, we estimated a resilient modulus (Mr) based on an empirical correlation to the DCP penetration. The Mr value provides an indication of the underlying subgrade strength. The Mr value correlated from the DCP testing is estimated to be an average of 3,000 psi. Subgrades with Mr values less than 5,000 psi are generally considered to be poor subgrade conditions for pavement. Other inputs for the AASHTO pavement analysis (i.e., layer coefficients, standard deviation, reliability) are based on the recommended values presented in the design guide.

Exhibit 6-2: Recommended Pavement Thicknesses

Loading Condition	Design Life (years)	AC Thickness (inches)	Base Aggregate Thickness (inches)	Comment
100,000 ESALs	20	4.0	12.0	Minimum Thickness
		5.0	9	Alternate Thickness
120,000 ESALs	20	4.0	13.0	Minimum Thickness
		5.0	10.0	Alternate Thickness

Due to the relatively low subgrade strength measured in our DCPs, we would recommend inclusion of a woven geotextile such as Mirafi RS380i or RS580i or equivalent be included between the subgrade and the base aggregate. A woven geotextile with properties similar to Mirafi RS380i is capable of providing both separation and some reinforcement of the pavement section and will improve the long-term performance of the pavement.

7 CONSTRUCTION CONSIDERATIONS

7.1 Earthwork

Site preparation will include clearing, grubbing, and stripping and excavation during subgrade preparation. Organic material and topsoil should be stripped and removed from all proposed building and pavement areas. Based on our test pit explorations, we anticipate a stripping depth of approximately 6 to 7 inches. Greater depths may be necessary to remove localized zones of organic material. Stripped material should be transported off site for disposal or used as fill in landscaping areas.

We recommend that the primary root systems for trees and other vegetation be completely removed. Trees designated for preservation should be clearly marked prior to site stripping, clearing, and grubbing. Trees and their root balls should be grubbed to the depth of the roots, which would exceed 3 feet bgs. Depending on the methods used to remove the root balls, considerable disturbance of the subgrade could occur during site clearing and grubbing. We recommend that soil disturbed during clearing and grubbing operations be removed and replaced with compacted crushed rock structural fill.

7.2 Subgrade Preparation

As recommended in Section 6.2, a 36-inch-thick reinforced crushed rock mat is recommended to be installed beneath the tank and vessel structural slab foundations a reinforced crushed rock. In Section 6.3, a minimum 6-inch-thick crushed rock pad is recommended to be installed beneath the restroom/storage facility footings. All subgrades beneath these recommended crushed rock sections; as well as the pavement subgrades should be neatly trimmed and carefully prepared. Any deleterious, loose, or softened material should be removed from the subgrade and replaced with crushed rock and any loose subgrade compacted prior to placing crushed rock. All crushed rock beneath foundations and pavement structures should be compacted to not less than 95 percent of the maximum dry density as determined by ASTM D1557.

Prior to the placement of crushed rock or the construction of foundations, structural slabs, or pavements, we recommend proof rolling the subgrade with a 10-to-12-ton roller operating in the static mode to identify any areas of excessive yielding. The proof rolling should be observed by the Geotechnical Engineer of Record or their representative who will evaluate the subgrade. If areas of excessive yielding are identified, the material in the yielding subgrade should be removed and replaced with compacted granular structural fill.

Areas that cannot be accessed with proof rolling compaction equipment should be evaluated by probing by the Geotechnical Engineer of Record or their representative. In addition, probing should be used to evaluate subgrade in pavement areas if the subgrade consists of primarily fine-grained soil that is wet and sensitive. Soft or weak spots should be over-excavated and replaced with imported compacted granular material. We recommend a contingency be placed in the budget for further over-excavation and stabilization of the subgrade as necessary.

We recommend that the footing excavations be observed by the Geotechnical Engineer of Record or their representative prior to placing steel and concrete to evaluate the suitability of the exposed subgrade, and to verify that the recommendations of this report have been followed and that conditions encountered are as anticipated. All deleterious, soft, or unsuitable materials observed by the Geotechnical Engineer should be removed and replaced with crushed rock compacted to not less than 95 percent of the maximum dry density determined by ASTM D1557.

7.3 Wet Weather Construction

Excavation and construction operations may expose the on-site soils that are sensitive to inclement weather conditions. The stability of exposed soils may rapidly deteriorate due to a change in moisture content (i.e., wetting or drying) or the action of heavy or repeated construction traffic. Accordingly, excavations should be adequately protected from the elements and from the action of repetitive or heavy construction loadings. Exposed subgrade should be covered with crushed rock as soon as practical to prevent water from softening the subgrade resulting in the need for additional over-excavation. Exposed subgrade should not be left open overnight during the rainy season which typically extends from mid-October to June. If water accumulates in the excavation for any reason, prior to placing crushed rock, the water should be sump pumped out with an engineered sump system to protect from removal of soils fines during pumping, and subgrade surface reobserved by the Geotechnical Engineer of Record.

7.4 Reinforced Crushed Rock Mat

The subgrade beneath the reinforced crushed rock mat should be prepared as described in Sections 7.1 and 7.2 of this report. A detail showing the reinforced crushed rock mat is presented in Figure 5.

After the subgrade has been approved by the Owner's Representative, we recommend placing a layer of non-woven geotextile conforming to the properties provided in Table 3, Section 9-33.2(1) of the Washington Department of Transportation Standard Specifications, such as Mirafi 180N. The non-woven geotextile layer should be immediately overlain by a geogrid such as Tensar InterAx NX750 or NX850 Geogrid or an approved alternative. An alternative to having separate geotextile layers (i.e., separate non-woven and geogrid materials) overlying the subgrade is to use a geotextile with both separation and reinforcing properties such as Mirafi RS580i. All geotextile and geogrid layers should be placed over the entire surface of the subgrade and joints overlapped and/or tied (in the case of geogrid) in accordance with the manufacturer's recommendations. The geogrid described above should not be placed along uneven surfaces or along localized low areas created by additional over-excavations. As shown on Figure 5, the geogrid should be placed as a horizontal layer.

The backfill placed over and between the geogrid layers should be structural fill material as described in Section 7.5 of this report. Within the crushed rock fill, we recommend placing a layer of geogrid at 12-inch intervals during backfill placement, as shown in Figure 5.

7.5 Structural Fill Placement and Compaction

Unless otherwise specified or shown, we recommend structural fill consisting of free-draining imported crushed rock. The imported crushed rock should be a maximum 3/4-inch particle size and contain less than 7 percent passing the No. 200 sieve based on a washed sieve analysis (ASTM D1140). Sand should not be used for structural fill or backfill.

Unless otherwise noted, structural fill should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557 and to a firm and unyielding condition. The structural fill should be placed in maximum loose lifts of 8 inches prior to applying compactive effort. The moisture content of the backfill material prior to and during compaction should be uniformly distributed throughout the material and should be less than 3 percent wet of optimum.

All new structural fill material placed directly beneath shallow foundations should consist of imported, crushed rock compacted, as described above. Over-excavation should be performed beyond the proposed foundation limits so that the zone that is within a 1H:1V line extending from the exterior of the foundation is encompassed. Each lift of compacted structural fill should be tested by the project's special inspection firm and the results provided to the Geotechnical Engineer for their review, prior to placement of subsequent lifts.

All new structural fill material placed directly beneath shallow foundations should consist of imported, crushed rock compacted, as described above. We do not recommend re-using excavated onsite soils as structural fill.

7.6 Excavation and Groundwater Control

Based on our explorations, we anticipate that the excavations for foundations and vaults can be accomplished using conventional equipment. All excavations should be completed in accordance with applicable Occupational Safety and Health Administration and state regulations.

Perched groundwater may be present within a few feet of the ground surface as evidenced in Test Pit TP-2 at the time of our subsurface exploration. Perched groundwater may also be present at shallower depths. Groundwater levels will fluctuate with the seasons and in response to extreme weather conditions.

If encountered, it may be possible to control perched groundwater by pumping from localized, well-constructed, filtered sumps. If used, this engineered sump system should be operated within a drainage layer material and not directly on exposed native subgrade. We recommend that the contractor be made responsible for the actual means and methods used to control groundwater, including perched water, and for providing stable excavation subgrades. The Contactor should also be responsible for necessary treatment of collected water to standards required for disposal and for the disposal itself.

7.7 Pavement Materials

Crushed surfacing top course and the crushed surfacing base course materials should meet the specifications provided in Section 9-03.9(3) of the WSDOT SS 2024. We recommend that the asphalt concrete consist of class ½-inch with a binder grade of PG64-22. Asphalt should meet the specifications included in Section 5-04 of the WSDOT SS 2024.

8 LIMITATIONS

The analyses, interpretations, conclusions, and recommendations contained in this report are based on site conditions as they presently exist and further assume that the explorations are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the past and current explorations. If future phases of work at the site uncover subsurface conditions different from those encountered in these explorations, we should be advised at once so that we can review these encountered conditions and reconsider our interpretations and conclusions, where necessary. If there is a substantial lapse of time between the submission of this report and the start of future phases of work at the site, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that we review our report to determine the applicability of the conclusions and recommendations.

Within the limitations of scope, schedule, and budget, the analyses, interpretations, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied. These interpretations and conclusions were based on our understanding of the project as described in this report and the site conditions as observed previously and at the time of our current explorations.

This report was prepared for the exclusive use of Brown and Caldwell, its subconsultants, and the City of Vancouver, as covered in this report. During future phases of work involving construction, the data contained in this report should be provided to the contractors for their information, but our report interpretations and conclusions should not be construed as a warranty of subgrade conditions as included in this report.

The scope of our present services did not include environmental assessments or evaluations regarding the presence or absence of wetlands, or hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this site, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered.

Shannon & Wilson, Inc., has prepared and included, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

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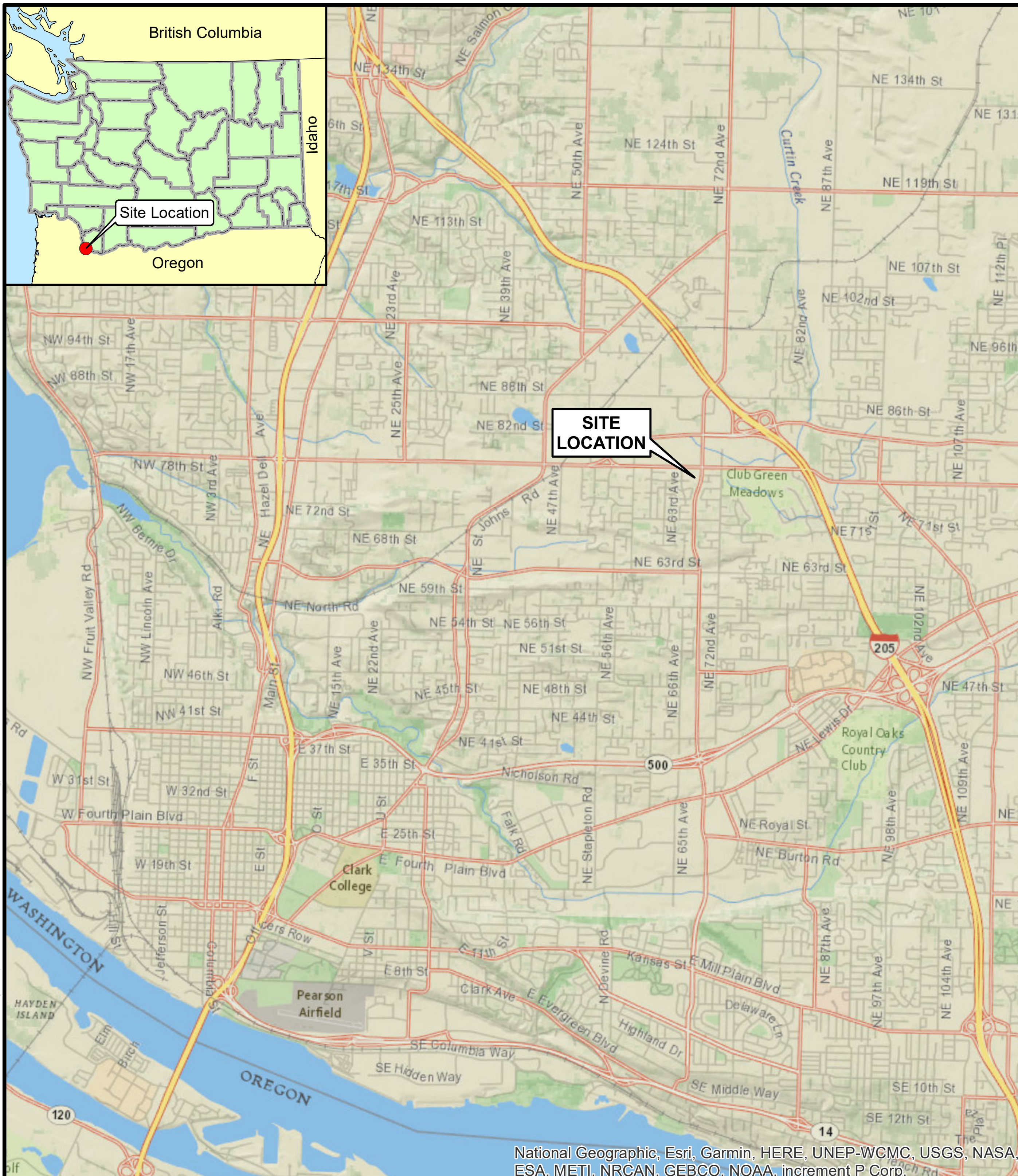

0 0.5 1 2

Scale in Miles

N

Vancouver WS14 PFAS Treatment Design
Vancouver, Washington

VICINITY MAP

July 2024 112510

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FIG. 1

Filename: P:\GIS\PDX\112000s\112510_Vancouver Water\Avmxd\VicinityMap.mxd Date: 2/29/2024 Login: AEH

NOTES
1. Aerial imagery obtained through Google Maps Satellite.
2. Existing contours and features adapted from file 180318-C-00SVV22.dwg, provided by Brown & Caldwell on March 21, 2024.
3. Proposed features adapted from file 180318-C-00SPV22.dwg, provided by Brown & Caldwell on March 21, 2024.

LEGEND
B-2 Designation and Approximate Location of Boring
B-1 Designation and Approximate Location of Boring with Vibrating Wire Piezometer
TP-1 Designation and Approximate Location of Test Pit with Infiltration Test
DCP-1 Designation and Approximate Location of Dynamic Cone Penetrometer (DCP) Test
Proposed Retaining Wall

0 30 60 120
Scale in Feet

Vancouver WS14 PFAS Treatment Design
Vancouver, Washington

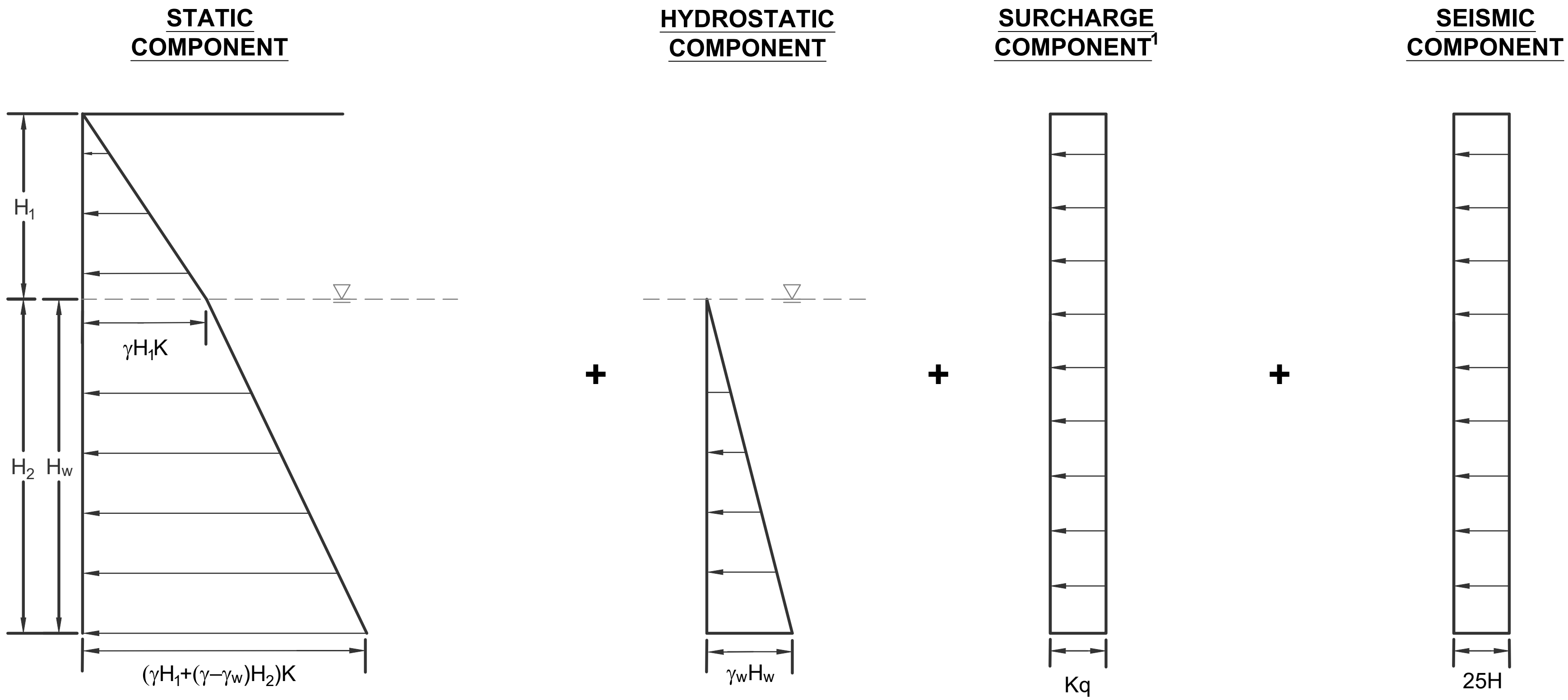
SITE AND EXPLORATION PLAN

July 2024 112510

SHANNON & WILSON **FIG. 2**

FIG. 2

File: P:\PDX\112000s\112510 Vancouver Water\DRAFTING\LateralEarthPressures.dwg Date: 03-28-2024 Author: AEH



NOT TO SCALE

NOTES

1. Surcharge due to construction and vehicle loading should be included and the estimated vertical surcharge from vehicle loading is 250 psf. Additional surcharge pressures for various loading configurations is presented on Figure 4.
2. Groundwater assumed to be below the base of the structure when drainage is provided. If drainage is not provided behind walls, the hydrostatic pressure component and buoyant soil pressures should be used along the full height of the wall.
3. The walls are assumed to have a level back slope behind the wall.

Vancouver WS14 PFAS Treatment Design
Vancouver, Washington

LATERAL EARTH PRESSURE DISTRIBUTION ON PERMANENT EMBEDDED WALLS

July 2024 112510

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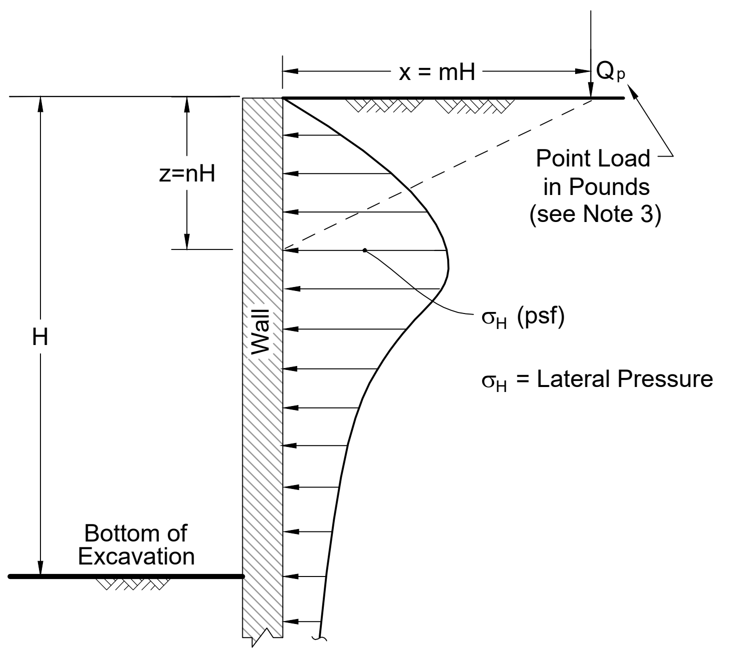
FIG. 3

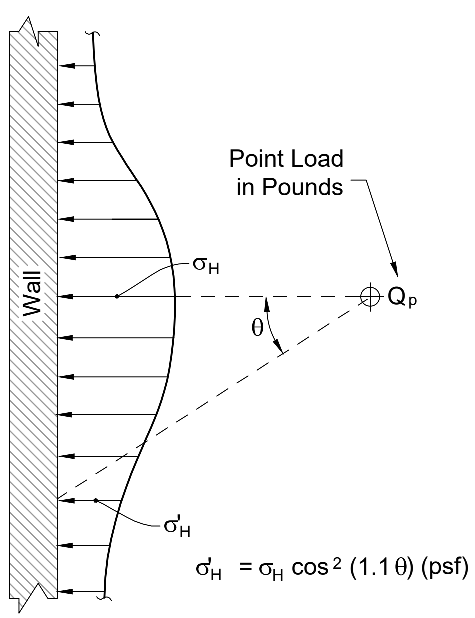
FIG. 3



ELEVATION VIEW

For $m \le 0.4$: $\sigma_H = 0.28 \dfrac{Q_p}{H^2} \dfrac{n^2}{(0.16 + n^2)^3}$ (psf) (see Note 3)

For $m > 0.4$: $\sigma_H = 1.77 \dfrac{Q_p}{H^2} \dfrac{m^2 n^2}{(m^2 + n^2)^3}$ (psf)

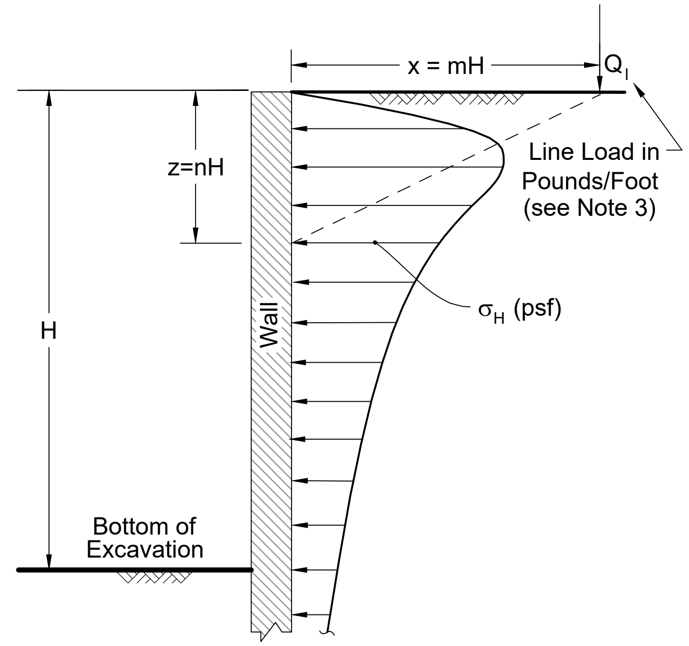


$\sigma'_H = \sigma_H \cos^2 (1.1\,\theta)$ (psf)

PLAN VIEW

A) LATERAL PRESSURE DUE TO POINT LOAD i.e. SMALL ISOLATED FOOTING OR WHEEL LOAD

(NAVFAC DM 7.2, 1986)



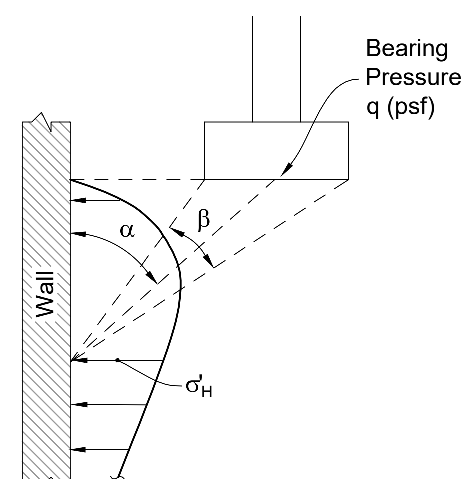
ELEVATION VIEW

For $m \le 0.4$: $\sigma_H = 0.20 \dfrac{Q_l}{H} \dfrac{n}{(0.16 + n^2)^2}$ (psf) (see Note 3)

For $m > 0.4$: $\sigma_H = 1.28 \dfrac{Q_l}{H} \dfrac{m^2 n}{(m^2 + n^2)^2}$ (psf)

B) LATERAL PRESSURE DUE TO LINE LOAD i.e. NARROW CONTINUOUS FOOTING PARALLEL TO WALL

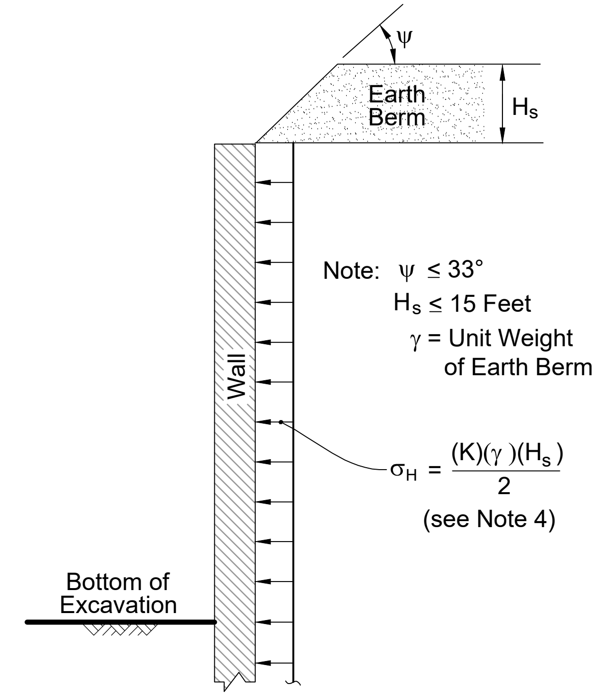
(NAVFAC DM 7.2, 1986)



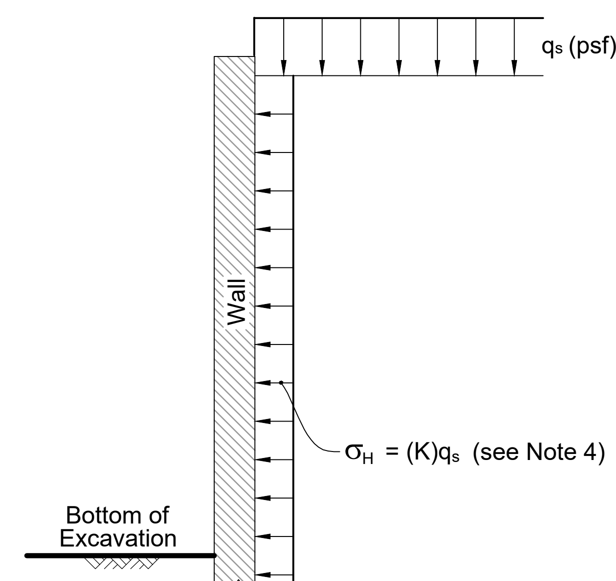
$\sigma_H = \dfrac{2q}{\pi} (\beta - \sin \beta \cos 2\alpha)$ (psf) — β in radians

C) LATERAL PRESSURE DUE TO STRIP LOAD

(derived from Fang, *Foundation Engineering Handbook*, 1991)



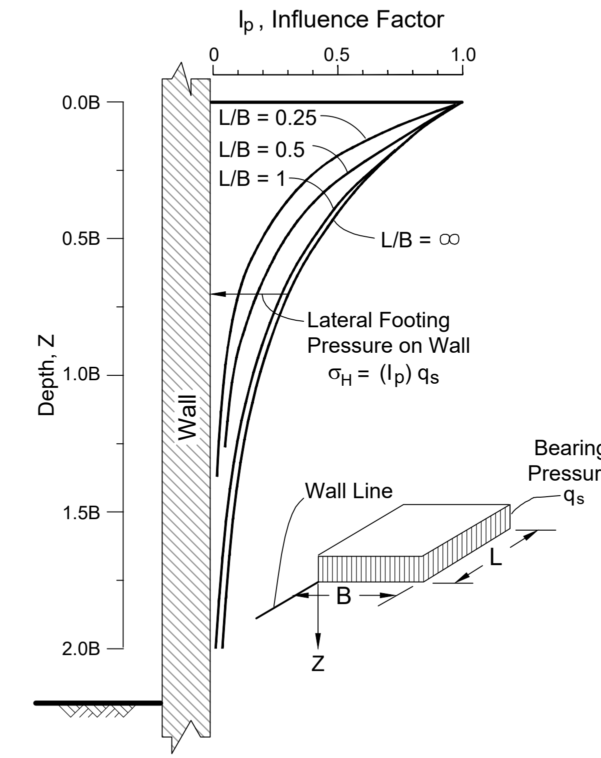
EARTH BERM



UNIFORM SURCHARGE

D) LATERAL PRESSURE DUE TO EARTH BERM OR UNIFORM SURCHARGE

(derived from Poulos and Davis, *Elastic Solutions for Soil and Rock Mechanics*, 1974; and Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, 1967)



E) LATERAL PRESSURE DUE TO ADJACENT FOOTING

(derived from NAVFAC DM 7.2, 1986; and Sandhu, *Earth Pressure on Walls Due to Surcharge*, 1974)

NOTES

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. If point or line loads are close to the back of the wall such that $m \le 0.4$, it may be more appropriate to model the actual load distribution (i.e., Detail E) or use more rigorous analysis methods.
4. See text for recommended K values.

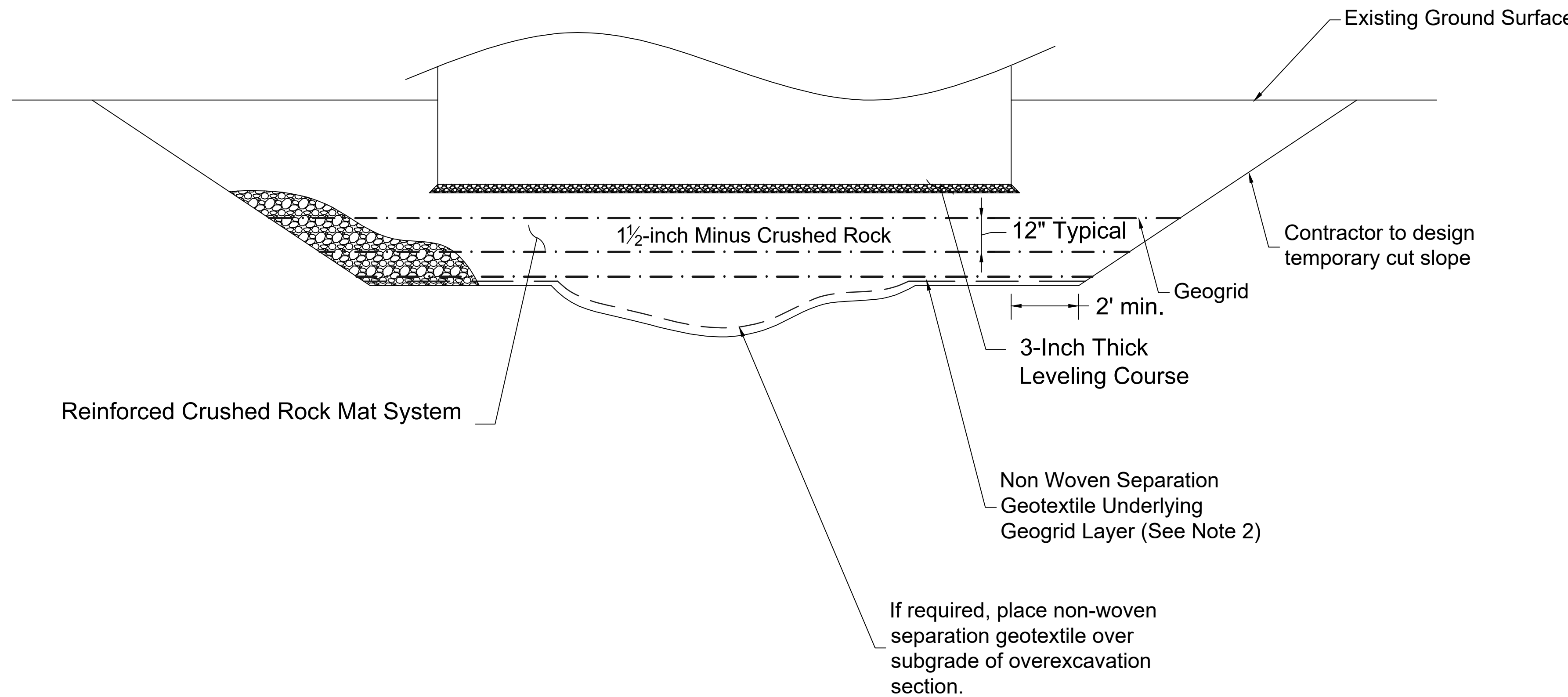
Vancouver WS14 PFAS Treatment Design
Vancouver, Washington

RECOMMENDED SURCHARGE LOADING FOR TEMPORARY AND PERMANENT WALLS

July 2024 | 112510

SHANNON & WILSON | **FIG. 4**

File: P:\PDX\112000s\112510 Vancouver Water\DRAFTING\SURCH TMP-PERM WALL (BIG).dwg Date: 03-28-2024 Author: AEH



NOTES
1. Not Drawn to Scale
2. An alternative to providing separate non-woven separation geotextile and geogrid materials over the subgrade is to provide a material where the two layers are pre-bonded together similar to Tensar RS580si.

Vancouver WS14 PFAS Treatment Design
Vancouver, Washington

TYPICAL REINFORCED CRUSHED ROCK PAD

July 2024 112510

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FIG. 5

File: P:\PDX\112000s\112171 Conveyor Structure\DRAFTING\Reinforced Gravel Mat.dwg Date: 03-28-2024 Author: AEH

APPENDIX A: FIELD EXPLORATIONS

Appendix A

Field Explorations

CONTENTS

Figures

APPENDIX A: FIELD EXPLORATIONS

A.1 GENERAL

The geotechnical field exploration program included two geotechnical borings, designated B-1 and B-2, and two test pits, designated TP-1 and TP-2. A vibrating wire piezometer (VWP) was installed in boring B-1. Infiltration testing was completed in TP-1. Two dynamic cone penetration (DCP) tests were completed designated DCP-1 and DCP-2. Completed exploration locations were measured in the field and are shown on the Site and Exploration Plan, Figure 2. Shannon & Wilson geology and engineering staff were present during the drilling of the geotechnical borings, test pits, and borehole installations to locate the drilling sites, log the materials encountered, and collect soil samples.

This appendix describes the techniques used to advance and sample the borings and presents logs of the materials encountered. DCP test and infiltration test procedures and data are also presented.

A.2 DRILLING

The borings were completed between February 15 and February 16, 2024. The borings were completed to 76.5 feet below ground surface (bgs). The borings were drilled by Holt Services, Inc. out of Vancouver, WA using a track-mounted Mobile B-57 drilling rig.

A.2.1 Disturbed Sampling

Disturbed samples were collected in the borings at 2.5-foot depth intervals, using a standard 2-inch outside diameter (O.D.) split spoon sampler in conjunction with Standard Penetration Testing (SPT). In a Standard Penetration Test, ASTM D1586, the 2-inch O.D. sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance or N-value. The SPT N-value provides a measure of in situ relative density of granular soils such as sand and gravel, and the consistency of cohesive soils such as silt and clay. All disturbed samples were visually identified and described in the field, sealed to retain moisture, and returned to our laboratory for additional examination and testing.

SPT N-values can be significantly affected by several factors, including the efficiency of the hammer used. Automatic hammers generally have higher energy transfer efficiencies than cathead (manual) hammers. For reference, cathead hammers are typically assumed to have an average energy efficiency of 60 percent. All N-values presented in this report are in blows per foot, as counted in the field. No corrections of any kind have been applied.

N-values of zero indicate that the sampler advanced the last 12 inches of the 18-inch sampling interval without a single hammer strike. That is, the weight of the drilling rods or the weight of the drilling rods plus the weight of the hammer (not in motion) was sufficient to advance the sampler.

An SPT was considered to have met "refusal" when 50 blows were required to drive the sampler 6 inches or less. If refusal was encountered in the first 6-inch interval (for example, 50 for 1.5"), the count is reported as 50/1st 1.5". If refusal was encountered in the second 6-inch interval (for example, 48, 50 for 1.5"), the count is reported as 50/1.5". If refusal was encountered in the last 6-inch interval (for example, 39, 48, 50 for 2"), the count is reported as 98/8".

A.3 GEOTECHNICAL TEST PITS

The geotechnical test pits were excavated on February 20, 2024, using a Deere 50G mini excavator equipped with a 2.5-foot wide-smooth bucket provided and operated by the Western States Soil Conservation Inc. out of Hubbard, Oregon. The test pits were excavated to depths between 3.0 to 6.6 feet bgs. A Shannon & Wilson representative was present during the explorations to locate the test pits, observe the excavation, collect soil samples, and log the materials encountered.

A.3.1 Soil Sampling

Disturbed jar samples were collected of cuttings from the excavator bucket or the spoils pile at select intervals determined in the field by the Shannon & Wilson representative on site. The material was examined, classified, and described in the field, sealed to retain moisture, and returned to our in-house laboratory for additional examination and testing.

A.4 MATERIAL DESCRIPTIONS

Soil samples were described and identified visually in the field in general accordance with ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the samples were noted. Once transported to Shannon & Wilson, the SPT and hand grab samples from the test pits were re-examined, and the field descriptions, and identifications were modified where necessary. The specific terminology used is defined in the Soil Description and Log Key, Figure A1.

A.5 DRILL LOGS

The summary logs of the borings are presented in the Logs of Borings, Figures A2 to A3. Material descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The left-hand portions of the logs show individual sample intervals, percent recovery, SPT data, and natural moisture content measurements. Material descriptions and geotechnical unit designations are shown in the center of the boring logs, and the right-hand portions provide a graphic log, miscellaneous comments, and a graphic depicting hole installation and backfill details.

A.6 BOREHOLE INSTALLATIONS

A.6.1 Vibrating Wire Pressure Transducer

A vibrating wire pressure transducer (Geokon model 4500S) was installed in boring B-1 to allow for ongoing groundwater level measurements. A vibrating wire pressure transducer (VWP) measures pressure using a pressure-sensitive diaphragm with a vibrating wire element attached to it. A cable runs from the transducer to the ground surface, where a readout device can be attached. Pressure acting on the outside face of the diaphragm causes it to deflect, which changes the tension of the wire element and the frequency of its vibration. The readout device measures the frequency of the induced vibration, which is converted to pressure, or head of groundwater above the transducer. The vibrating wire pressure transducer was permanently grouted into place in the borehole. The instrument was taped to a measured length of polyvinyl chloride (PVC) pipe to control the depth of installation. Prior to insertion, an initial "zero" reading was taken at "zero" head. With the transducer and PVC pipe in place, the hole was tremie grouted with bentonite cement grout. The cable leading up from the transducer is protected at the surface by a flush-mount monument set in concrete. The VWP installation depth is indicated on the boring log.

A.7 DYNAMIC CONE PENETROMETER TESTS

Pavement subgrade testing was conducted in two locations, designated DCP-1 and DCP-2, using a Dynamic Cone Penetrometer (DCP). The DCP is a device widely used to determine in situ strength properties of base materials and subgrade soils. The four main components of the DCP include the cone, rod, anvil, and hammer. The cone is attached to one end of the DCP rod while the anvil and hammer are attached to the other end. Energy is applied to the cone tip through the rod by dropping the 17.64-pound hammer a distance of 22.6 inches against the anvil. The diameter of the cone is 0.16 inches larger than the rod to ensure that only tip resistance is measured. The number of blows required to advance the cone into the subsurface materials is recorded. The DCP index is the ratio of the depth of penetration to the number of blows of the hammer. This can be correlated to a variety of material properties, including CBR and Resilient Modulus. DCP testing was performed by a Shannon & Wilson geologist. This appendix presents DCP Test Data in Figures A6 and A7.

A.8 INFILTRATION TESTING

In situ infiltration testing was completed using the Alternative Single-Ring Falling Head Infiltration test procedure to support design of stormwater infiltration facilities within the project site area. The tests were conducted in accordance with the City of Vancouver Surface Water Management Design and Construction Requirements and the Clark County Stormwater Manual. At the infiltration test location, a hole was excavated with a Deere 50G mini excavator with a smooth bucket to the desired test depth of 3 feet. In this method, a 6-inch-diameter PVC casing was installed and encased into the bottom of the hole. Nine inches of water was then added to the bottom of the PVC casing to pre-soak the soil for a period of four hours. After the presoak period, water was again added to the hole to return the level to nine inches over the soil surface, and the drawdown was measured as the water level dropped to 3-inches above the top of the soil. This process was repeated to perform as many 6-inch drawdown trials that could be completed in a 1-hour time period. After testing, the test pit was over-excavated to explore the subsurface conditions beneath the tested depth. Infiltration test results are presented in Figure A8.

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[2]	FINE-GRAINED SOILS (50% or more fines)[1]	COARSE-GRAINED SOILS (less than 50% fines)[1]
Major	***Silt, Lean Clay, Elastic Silt,*** or ***Fat Clay***[3]	***Sand*** or ***Gravel***[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: ***Sandy*** or ***Gravelly***[4]	More than 12% fine-grained: ***Silty*** or ***Clayey***[3]
Minor Follows major constituent	15% to 30% coarse-grained: ***with Sand*** or ***with Gravel***[4]	5% to 12% fine-grained: ***with Silt*** or ***with Clay***[3]
	30% or more total coarse-grained *and* lesser coarse-grained constituent is 15% or more: ***with Sand*** or ***with Gravel***[5]	15% or more of a second coarse-grained constituent: ***with Sand*** or ***with Gravel***[5]

[1]All percentages are by weight of total specimen passing a 3-inch sieve.
[2]The order of terms is: *Modifying Major with Minor*.
[3]Determined based on behavior.
[4]Determined based on which constituent comprises a larger percentage.
[5]Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND	
Fine	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.)
Medium	#40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.)
Coarse	#10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL	
Fine	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.)
Coarse	3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

Bentonite Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Inclinometer or Non-perforated Casing
Gravel	Vibrating Wire Piezometer
Perforated or Screened Casing	

PERCENTAGES TERMS [1, 2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1]Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

[2]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Vancouver Water Station 14
Vancouver, Washington

SOIL DESCRIPTION AND LOG KEY

July 2024 112510

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A1
Sheet 1 of 3

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW	Well-Graded Gravel; Well-Graded Gravel with Sand
			GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM	Silty Gravel; Silty Gravel with Sand
			GC	Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW	Well-Graded Sand; Well-Graded Sand with Gravel
			SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM	Silty Sand; Silty Sand with Gravel
			SC	Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat or other highly organic soils (see ASTM D4427)

FILL	Placed by humans, both engineered and nonengineered. May include various soil materials and debris.		The Fill graphic symbol is combined with the soil graphic that best represents the observed material

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the *CL-ML* area of the plasticity chart.
2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.
3. The soil graphics above represent the various USCS identifications (i.e., *GP*, *SM*, etc.) and may be augmented with additional symbology to represent differences within USCS designations. *Sandy Silt (ML)*, for example, may be accompanied by the *ML* soil graphic with sand grains added. Non-USCS materials may be represented by other graphic symbols; see log for descriptions.

Vancouver Water Station 14
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SOIL DESCRIPTION AND LOG KEY

July 2024 112510

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FIG. A1
Sheet 2 of 3

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASITICTY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4%
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10%
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20%
High	It take considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20%

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
approx.	Approximate/Approximately
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

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SOIL DESCRIPTION AND LOG KEY

July 2024 112510

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FIG. A1
Sheet 3 of 3

2013 BORING CLASS3 112510.GPJ SW2013LIBRARYPDX.GLB SWNEW.GDT 3/7/24

Total Depth:	71.5 ft.	Northing:	~ 132,640 ft.	Drilling Method:	Mud Rotary
Top Elevation:	~ 269 ft.	Easting:	~ 1,102,972 ft.	Drilling Company:	Holt Services, Inc.
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	Mobile B-57 Track Rig #13
Horiz. Datum:	WA83-SF	Offset:	~	Other Comments:	

Hole Diam.:	5" in.
Rod Type:	NWJ
Hammer Type:	Automatic

SOIL DESCRIPTION

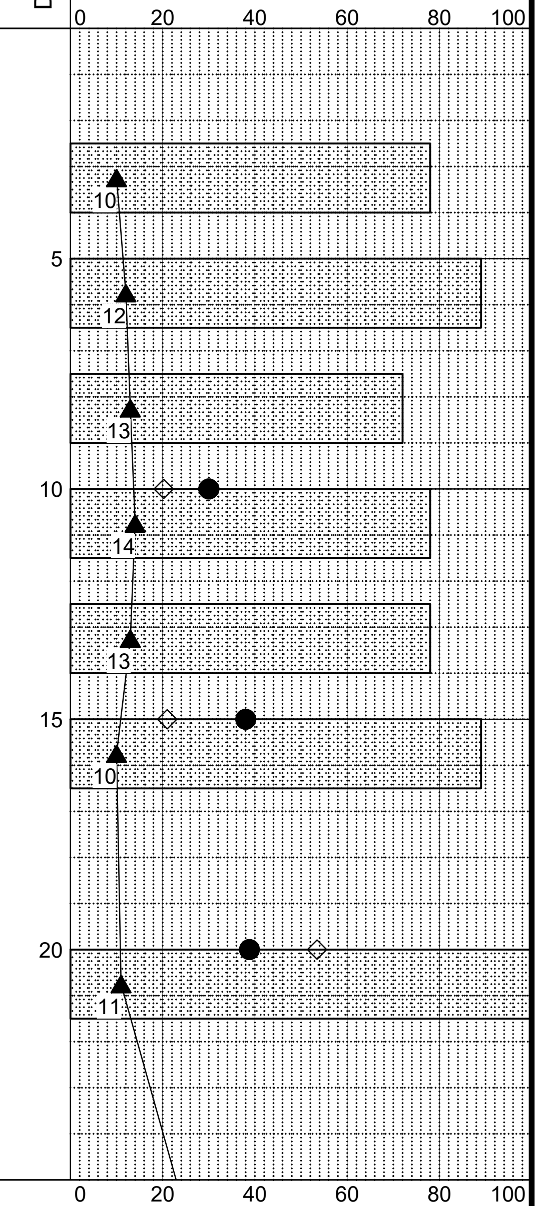
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples
Topsoil: 6-inches thick	268.5 / 0.5	
Loose to medium dense/stiff, brown and red-brown, *Sandy Silt (ML)*; moist; fine to coarse sand; nonplastic to low plasticity; trace organics; micaceous; iron oxidation and staining.		S-1
CATASTROPHIC FLOOD DEPOSITS	264.5 / 4.5	
Medium dense, brown, *Silty Sand (SM)*; moist; interbedded with *Poorly Graded Sand with Silt (SP-SM)*; fine to coarse sand; nonplastic fines; trace organics and rootlets; micaceous; iron oxidation and staining.		S-2, S-3
Medium dense, brown, *Silty Sand (SP-SM)*; moist to wet; interbedded with *Poorly Graded Sand with Silt (SP-SM)*; fine to medium sand; nonplastic fines; micaceous; iron oxidation and staining.	259.5 / 9.5	S-4, S-5, S-6
Medium dense, brown, *Sandy Silt (ML)*; moist; contains a 3" thick interbed of *Silt (ML)* to *Lean Clay (CL)*; fine to coarse sand; nonplastic; micaceous.	251.0 / 18.0	S-7
Medium dense, brown, *Poorly Graded Sand with Silt to Silty Sand (SP-SM/SM)*; moist; fine to medium sand; nonplastic fines; micaceous.	246.0 / 23.0	

CONTINUED NEXT SHEET

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

Standard Penetration Test

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Vancouver Water Station 14
Vancouver, Washington

LOG OF BORING B-1

July 2024

112510

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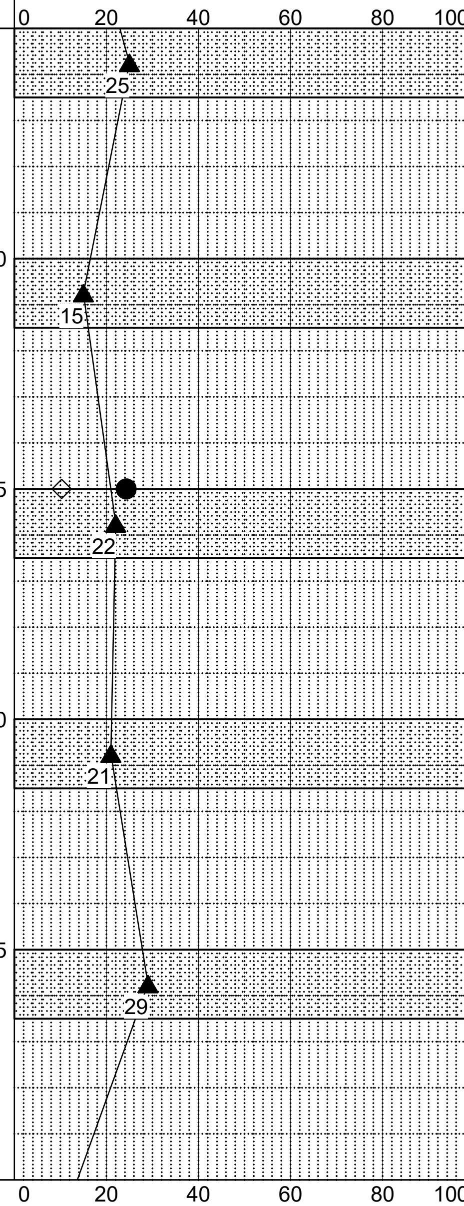
FIG. A2
Sheet 1 of 3

REV 2

Total Depth:	71.5 ft.	Northing:	~ 132,640 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	5" in.
Top Elevation:	~ 269 ft.	Easting:	~ 1,102,972 ft.	Drilling Company:	Holt Services, Inc.	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	Mobile B-57 Track Rig #13	Hammer Type:	Automatic
Horiz. Datum:	WA83-SF	Offset:	~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Continued: Medium dense, brown, Poorly Graded Sand with Silt to Silty Sand (SP-SM/SM); moist; fine to medium sand; nonplastic fines; micaceous. **CATASTROPHIC FLOOD DEPOSITS**			S-8 S-9		30
Medium dense, brown, *Poorly Graded Sand to Poorly Graded Sand with Silt (SP/SP-SM)*; moist; fine to medium sand; nonplastic fines; micaceous.	*236.0* 33.0		S-10 S-11 S-12		35 40 45
Medium dense, brown, *Silty Sand (SM)*; moist; fine to medium sand; nonplastic fines; micaecous.	*221.0* 48.0				

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



CONTINUED NEXT SHEET

LEGEND

Standard Penetration Test

Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├───┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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Vancouver, Washington

LOG OF BORING B-1

July 2024 112510

SHANNON & WILSON

FIG. A2
Sheet 2 of 3

REV 2

Total Depth: 71.5 ft.	Northing: ~ 132,640 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	5" in.
Top Elevation: ~ 269 ft.	Easting: ~ 1,102,972 ft.	Drilling Company:	Holt Services, Inc.	Rod Type:	NWJ
Vert. Datum: NAVD88	Station: ~	Drill Rig Equipment:	Mobile B-57 Track Rig #13	Hammer Type:	Automatic
Horiz. Datum: WA83-SF	Offset: ~	Other Comments:			

SOIL DESCRIPTION

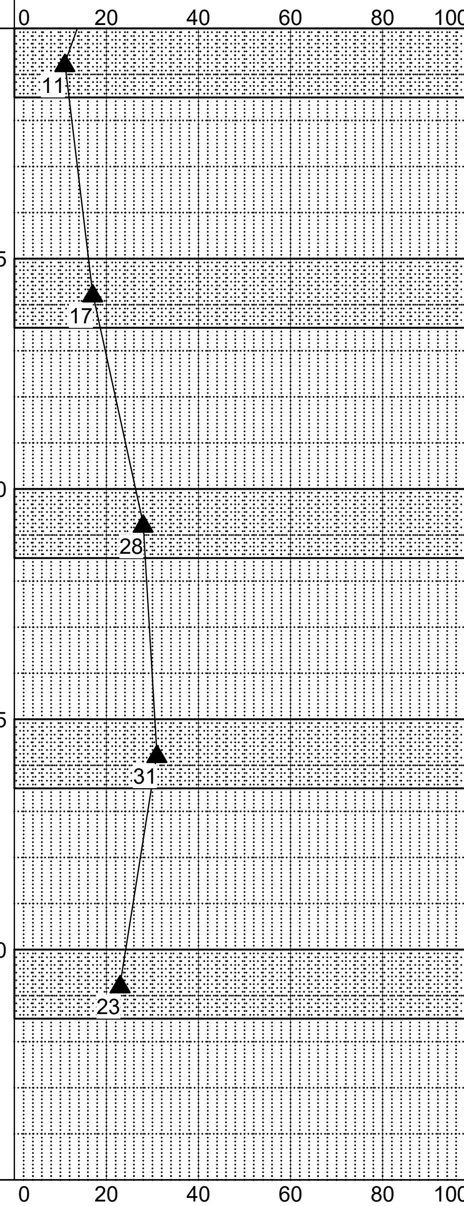
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)	Samples
Continued: Medium dense, brown, Silty Sand (SM); moist; fine to medium sand; nonplastic fines; micaecous. **CATASTROPHIC FLOOD DEPOSITS**		S-13, S-14
Medium dense, brown, *Sandy Silt to Silty Sand (ML/SM)*; moist; fine sand; nonplastic.	*211.0* 58.0	S-15
Hard, brown, *Lean Clay with Sand (CL)*; moist; fine sand; medium plasticity; micaceous; iron oxidation and staining.	*206.0* 63.0	S-16
Very stiff, brown, *Sandy Silt to Silt with Sand (ML)*; moist; interbedded with *Silty Sand (SM)*; fine sand; low plasticity; micaceous; iron oxidation and staining. *Geokon Vibrating Wire Piezometer (Serial # 2331100) installed 70 feet below ground surface*	*201.0* 68.0	S-17
Completed: February 15, 2024	*197.5* 71.5	

Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



LEGEND

Standard Penetration Test

Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Vancouver Water Station 14
Vancouver, Washington

LOG OF BORING B-1

July 2024 | 112510

SHANNON & WILSON

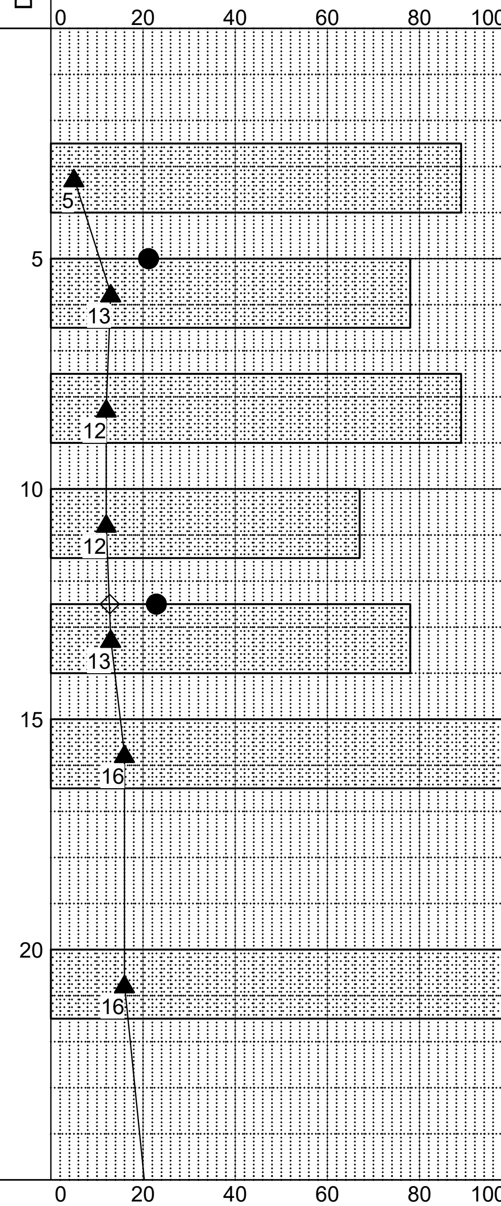
FIG. A2
Sheet 3 of 3

REV 2

Total Depth: 71.5 ft.	Northing: ~ 132,554 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	5" in.
Top Elevation: ~ 274 ft.	Easting: ~ 1,102,822 ft.	Drilling Company:	Holt Services, Inc.	Rod Type:	NWJ
Vert. Datum: NAVD88	Station: ~	Drill Rig Equipment:	Mobile B-57 Track Rig #13	Hammer Type:	Automatic
Horiz. Datum: WA83-SF	Offset: ~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Samples
Topsoil: 6-inches thick	273.1 0.5	
Loose to medium dense, brown, *Silty Sand (SM)*; moist; fine to coarse sand; nonplastic fines; trace organics; micaceous; slight iron oxidation and staining. **CATASTROPHIC FLOOD DEPOSITS**		S-1, S-2
Medium dense, brown, *Poorly Graded Sand with Silt to Silty Sand (SP-SM/SM)*; moist; trace fine to coarse, subrounded gravel; fine to coarse sand; nonplastic fines; micaceous; slight iron oxidation and staining.	266.6 7.0	S-3, S-4
Medium dense, brown, *Poorly Graded Sand with Silt (SP-SM)*; moist; fine to medium sand; nonplastic fines; micaceous.	261.6 12.0	S-5, S-6, S-7
Medium dense, brown, *Silty Sand (SM)*; moist; interbedded with *Poorly Graded Sand with Silt (SP-SM)*.	250.6 23.0	

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

Standard Penetration Test

Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──┤ Liquid Limit

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Vancouver Water Station 14
Vancouver, Washington

LOG OF BORING B-2

July 2024 112510

SHANNON & WILSON

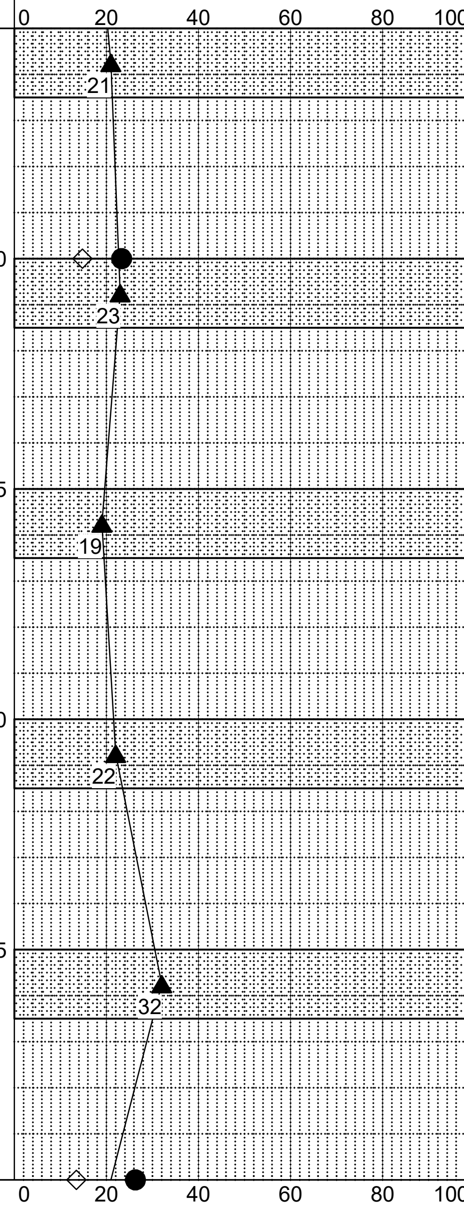
FIG. A3
Sheet 1 of 3

REV 2

Total Depth: 71.5 ft.	Northing: ~ 132,554 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	5" in.
Top Elevation: ~ 274 ft.	Easting: ~ 1,102,822 ft.	Drilling Company:	Holt Services, Inc.	Rod Type:	NWJ
Vert. Datum: NAVD88	Station: ~	Drill Rig Equipment:	Mobile B-57 Track Rig #13	Hammer Type:	Automatic
Horiz. Datum: WA83-SF	Offset: ~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Continued: Medium dense, brown, Silty Sand (SM); moist; interbedded with *Poorly Graded Sand with Silt (SP-SM)*; fine to medium sand; nonplastic fines; micaceous.			S-8		
CATASTROPHIC FLOOD DEPOSITS Medium dense, brown, *Poorly Graded Sand with Silt (SP-SM)*; moist; fine to medium sand.	*245.6* 28.0		S-9		30
Medium dense, brown, *Silty Sand (SM)*; moist; fine sand; nonplastic fines; micaceous.	*240.6* 33.0		S-10		35
Medium dense to dense, brown, *Poorly Graded Sand to Poorly Graded Sand with Silt (SP/SP-SM)*; moist; fine to medium sand; nonplastic fines; micaceous.	*235.6* 38.0		S-11 S-12		40 45
Medium dense, brown, *Silty Sand (SM)*; moist; fine sand; nonplastic fines; micaceous.	*225.6* 48.0				

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

Standard Penetration Test

Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ⊢—⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Vancouver Water Station 14
Vancouver, Washington

LOG OF BORING B-2

July 2024 112510

SHANNON & WILSON

FIG. A3
Sheet 2 of 3

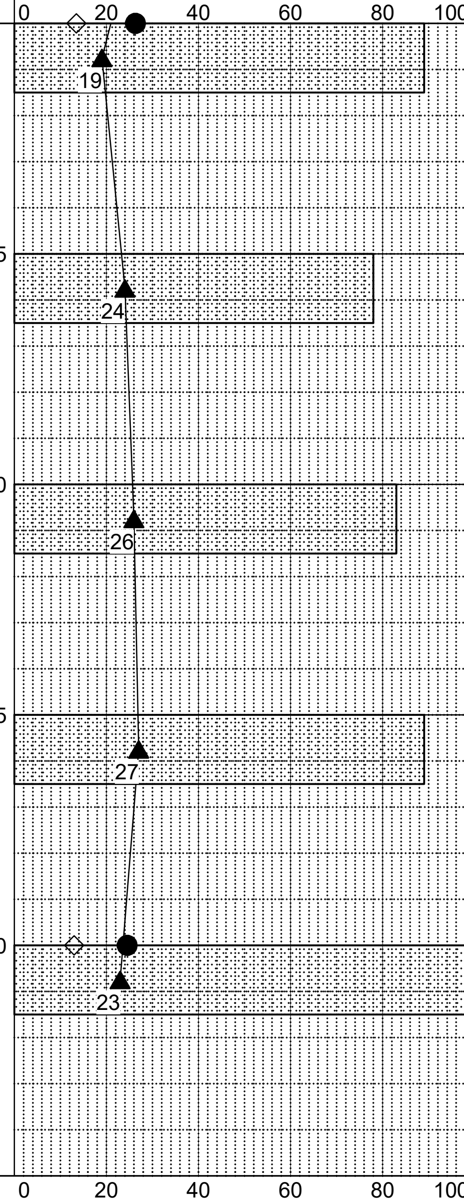
REV 2

Total Depth: 71.5 ft.	Northing: ~ 132,554 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	5" in.
Top Elevation: ~ 274 ft.	Easting: ~ 1,102,822 ft.	Drilling Company:	Holt Services, Inc.	Rod Type:	NWJ
Vert. Datum: NAVD88	Station: ~	Drill Rig Equipment:	Mobile B-57 Track Rig #13	Hammer Type:	Automatic
Horiz. Datum: WA83-SF	Offset: ~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Medium dense, brown, Silty Sand (SM); moist; fine sand; nonplastic fines; micaceous. **CATASTROPHIC FLOOD DEPOSITS**			S-13		
Medium dense, brown, *Poorly Graded Sand with Silt (SP-SM)*; moist; fine to medium sand; nonplastic fines; micaceous.	*220.6* 53.0		S-14 S-15 S-16 S-17		55 60 65 70
Completed: February 16, 2024	*202.1* 71.5				

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ⊢—⊣ Liquid Limit

LEGEND

Standard Penetration Test

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Vancouver Water Station 14
Vancouver, Washington

LOG OF BORING B-2

July 2024 112510

SHANNON & WILSON

FIG. A3
Sheet 3 of 3

REV 2

MASTER_LOG_E 112510.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 3/7/24 *Log: DSJ Rev: VCB Typ: DSJ*

Coordinates: N: ~ 132,860 ft. E: ~ 1,103,001 ft.
Elevation: ~ 260 ft.

SOIL PROFILE DESCRIPTION	Depth (ft)	Symbol	Samples	Type	Ground Water	NOTES
Brown, *Silt with Sand (ML)*; moist; fine sand; nonplastit to low plasticity; trace organics and rootlets; micaceous. **TOPSOIL**	0.6		G-1	G		
Brown, *Silty Sand (SM)*; moist to wet; trace fine subangular gravel; fine to coarse sand; nonplastic fines; slight iron oxidation and staining; micaceous; groundwater started entering the test pit at a rapid pace at approximately 3.6' bgs. **CATASTROPHIC FLOOD DEPOSITS** *Water began entering the test pit at approximately 3.6' bgs*	6.6		G-2, G-3, G-4	G, G, G	Seepage	
Completed: February 19, 2024						

TEST PIT PHOTOS

NOTES

1. The description in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
2. Refer to Soil Classification and Log Key for explanation of "Symbols" and Definitions.
3. Group symbol is based on visual-manual identification.
4. Where possible, a 1/2-inch-diameter, steel T-bar probe was used to estimate the density of soil.

LEGEND

- Seepage
- G Grab Sample

Vancouver Water Station 14
Vancouver, Washington

LOG OF TEST PIT TP-1

July 2024 112510

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FIG. A4

FIG. A4

REV 2

Coordinates: N: ~ 132,732 ft. E: ~ 1,103,005 ft.
Elevation: ~ 262 ft.

SOIL PROFILE DESCRIPTION	Depth (ft)	Symbol	Samples	Type	Ground Water	NOTES
Brown, *Silt with Sand (ML)*; moist; trace fine subangular gravel; fine to medium sand; nonplastic to low plasticity; trace organics and rootlets; slight iron oxides; micaceous. **TOPSOIL**	0.5		G-1	G		
Brown, *Silty Sand to Silty Sand with Gravel (SM)*; moist to wet; trace fine subangular gravel; fine to coarse sand; nonplastic fines; slight iron oxidation and staining; micaceous. *Water began entering the test pit at approximately 2' bgs* **CATASTROPHIC FLOOD DEPOSITS**	3.0		G-2 G-3	G G	Seepage	
Completed: February 19, 2024						

TEST PIT PHOTOS

NOTES
1. The description in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
2. Refer to Soil Classification and Log Key for explanation of "Symbols" and Definitions.
3. Group symbol is based on visual-manual identification.
4. Where possible, a 1/2-inch-diameter, steel T-bar probe was used to estimate the density of soil.

LEGEND
- Seepage
- G Grab Sample

Vancouver Water Station 14
Vancouver, Washington

LOG OF TEST PIT TP-2

July 2024 112510

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FIG. A5

FIG. A5

REV 2

DCP TEST DATA

Project: *Vancouver Water Station 14 - 112510* **Date:** *20-Feb-24*
Location: *DCP-1* **Soil Type(s):** *Silty SAND*

Hammer
- ◯ 10.1 lbs.
- ◉ 17.6 lbs.
- ◯ Both hammers used

- ◯ CH
- ◯ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
2	161	1
2	538	1
2	751	1
2	912	1
		1
		1
		1
		1
		1
		1
		1
		1
		1
		1
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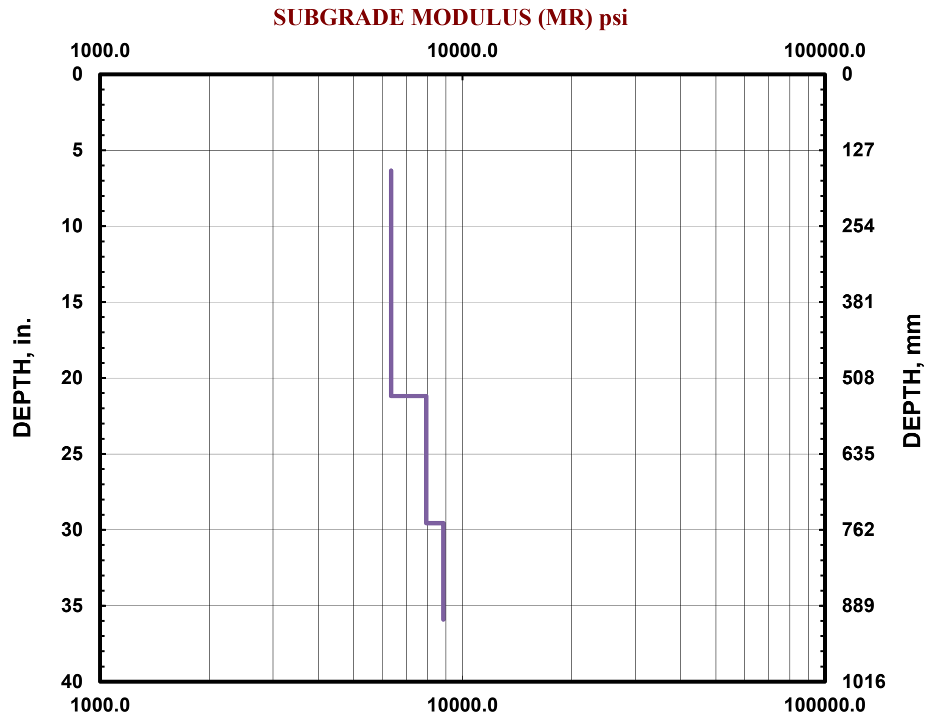
		1
		1
		1
		1
		1



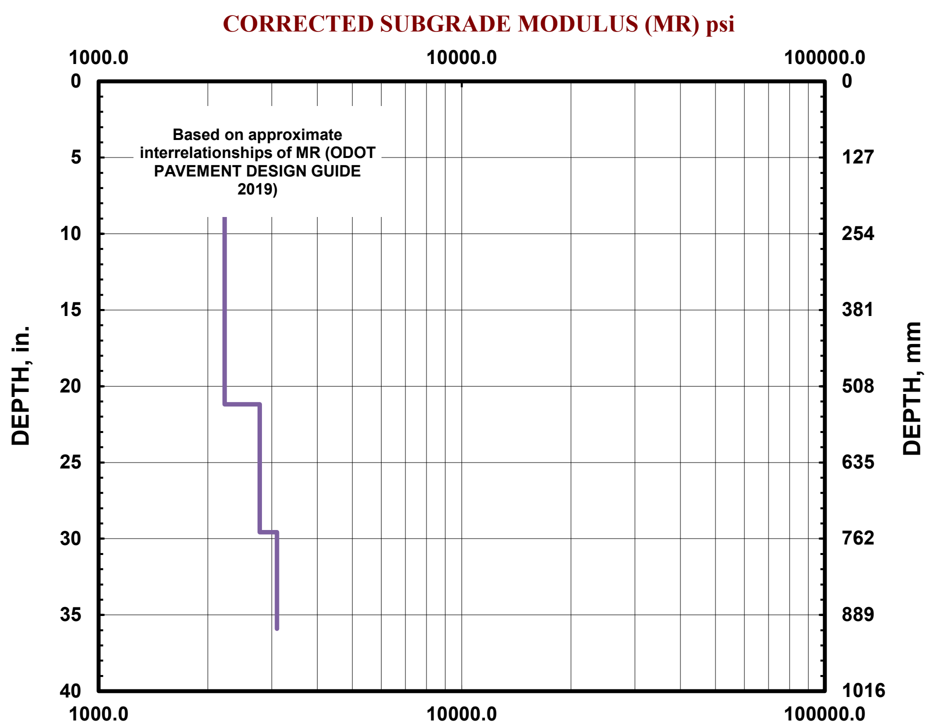


FIG. A6

DCP TEST DATA

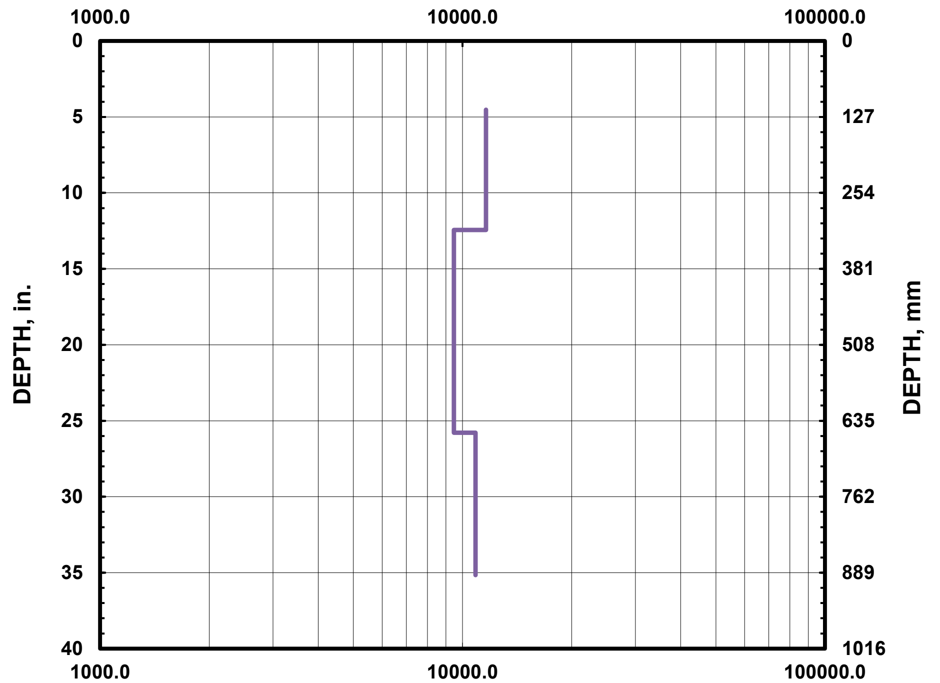
Project: *Vancouver Water Station 14 - 112510*
Date: *20-Feb-24*
Location: *DCP-2*
Soil Type(s): *Silty SAND with gravel*

Hammer
- ◯ 10.1 lbs.
- ◉ 17.6 lbs.
- ◯ Both hammers used

- ◯ CH
- ◯ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
5	115	1
5	316	1
5	655	1
5	893	1
		1
		1
		1
		1
		1
		1
		1
		1
		1
		1
		1
		1
		1
		1
		1
		1

SUBGRADE MODULUS (MR) psi



CORRECTED SUBGRADE MODULUS (MR) psi

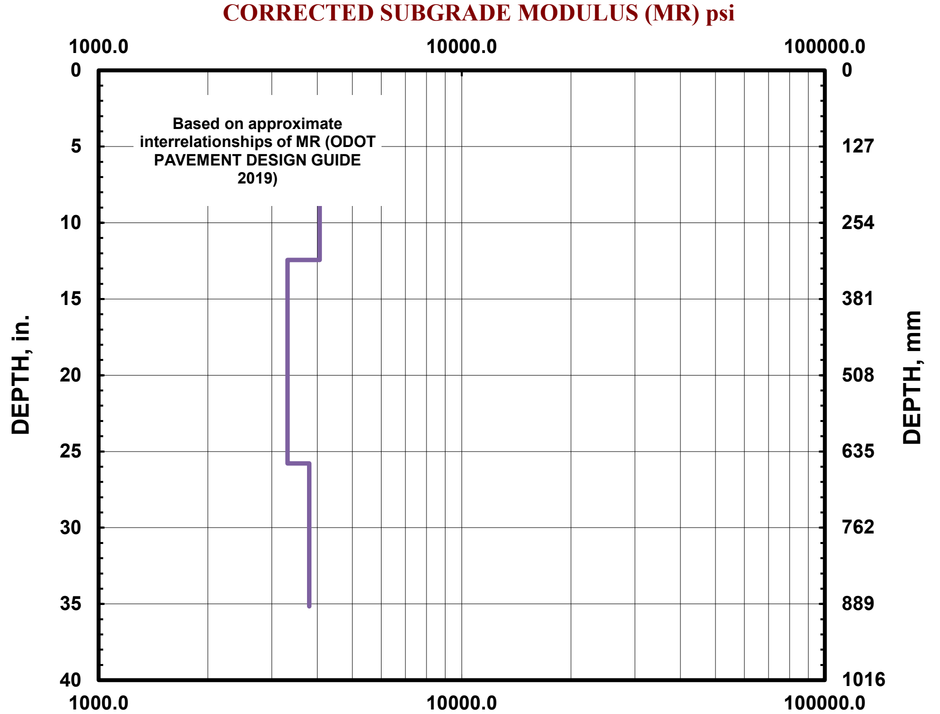


FIG. A7

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Location: TP-1 Vancouver Water Station 14	**Date:** 2/20/2024 **Job Number:** 112510	**Infiltration Test Number:** TP-1
Depth to bottom of hole: 3.00 ft	**Dimension of Pipe:** 3.5'X6.0"	**Test Method:** Alternative Single-Ring Falling Head Infiltration Test

Tester's Name: Nick Berry
Tester's Company: Shannon & Wilson, Inc.

Depth (feet)	Soil Texture:
0-0.5	Organic Topsoil
0.5-3.0	Silty Sand

Presaturation Start Time: 0806
Presaturation End Time: 1206

Time	Time Interval (minutes)	Measurement (feet)	Head (feet)	Drop in Water Level (feet)	Infiltration rate (inches per hour)	Remarks
12:06	--	2.42	1.08	--	--	Average Rate Over Trial = 0.53 in/hr
12:16	10	2.44	1.06	0.02	1.73	
12:26	10	2.44	1.06	0.00	0.00	
12:36	10	2.45	1.05	0.01	0.72	
12:46	10	2.45	1.05	0.00	0.00	
12:56	10	2.46	1.04	0.01	0.72	
13:06	10	2.46	1.04	0.00	0.00	

Vancouver Water Station 14
Vancouver, WA

INFILTRATION TEST RESULTS TP-1

July 2024 112510

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A8

APPENDIX B: LABORATORY TESTING

Appendix B

Laboratory Testing

CONTENTS

Attachments

Laboratory Test Summary Report from Cooper Testing Laboratory, Inc., dated February 29, 2024.

APPENDIX B: LABORATORY TESTING

B.1 GENERAL

Soil samples obtained during the field explorations were described and identified in the field in general accordance with the Practice for Description and Identification of Soils (Visual-Manual Procedure), ASTM D2488. The Specific terminology used is defined in the Soil Description and Log Key, Figure A1, Appendix A. The physical characteristics of the collected samples were noted, and field descriptions and identifications were modified, as necessary, in accordance with the terminology presented in Appendix A, Figure A1.

During the review, select soil samples were collected for further testing. The material descriptions and identifications were refined/revised, as necessary, based on the results of the laboratory tests. The soil testing program included moisture content testing, particle size analyses, and corrosivity testing. Results of the soil testing are presented in the Log of Borings in Appendix A, and in the Laboratory Test Summary Report from Cooper Testing Laboratory, Inc., dated February 29, 2024. All laboratory tests were performed in accordance with applicable ASTM International (ASTM) standards.

B.2 SOIL TESTING

B.2.1 Moisture (Natural Water) Content

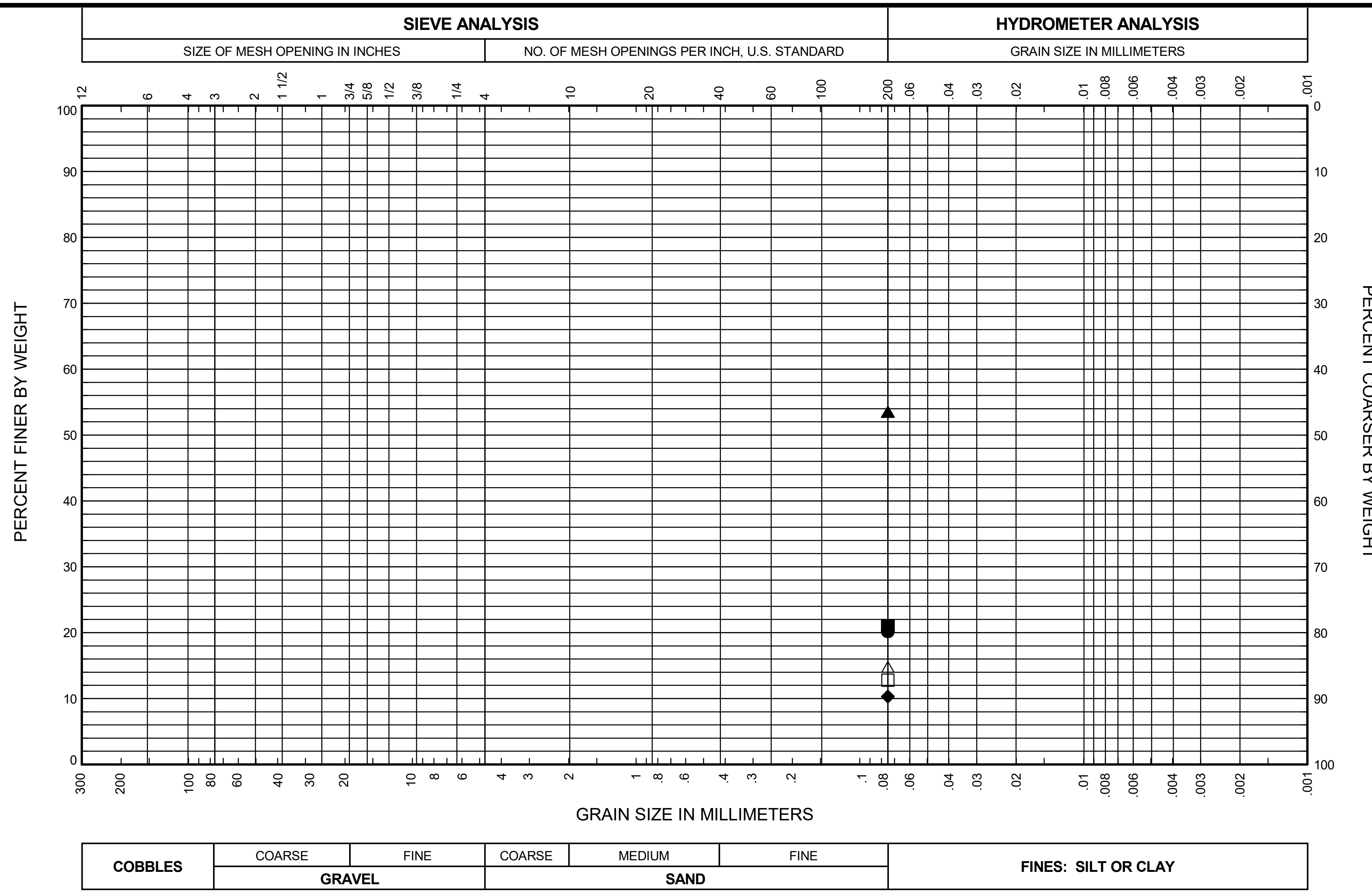
Natural moisture content determinations were performed in accordance with ASTM D2216, on selected soil samples. The natural moisture content is a measure of the amount of moisture in the soil at the time of exploration. It is defined as the ratio of the weight of water to the dry weight of the soil, expressed as a percentage. The results of moisture content determinations are presented on the Logs of Borings in Appendix A.

B.2.1 Particle-size Analyses

Particle-size analyses were conducted on select samples to determine their grain-size distributions. Grain size distributions were determined in accordance with ASTM D6913 D1140, and D422 as applicable. For all samples, a wet sieve analysis was performed to determine the percentage (by weight) of each sample passing the No. 200 (0.075 mm) sieve. Particle size analyses were performed by Shannon & Wilson. The percentage of each sample passing the No. 200 sieve is presented on the Logs of Borings in Appendix A.

B.2.2 Corrosivity Testing

Analytical testing was performed on specimens to evaluate the corrosivity potential of the soil at the site. The corrosivity test suite included redox potential, soil conductivity (resistivity), soluble chloride and sulfate concentration, sulfide tests, and soil pH. Analytical testing was performed by Cooper Testing Laboratory, Inc., of Palo Alto, California. In total, two samples ranging from 2.5 to 6.5 feet below ground surface were tested. The results of the corrosivity testing are attached to the end of this appendix.



BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	Cc	Cu	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-1, S-4	10.0	*SM*	*Silty Sand*			-	-	20.2	30	
■ B-1, S-6	15.0	*SM*	*Silty Sand*			-	-	21.0	38	
▲ B-1, S-7	20.0	*ML*	*Sandy Silt*			-	-	53.5	39	
◆ B-1, S-10	35.0	*SP-SM*	*Poorly Graded Sand with Silt*			-	-	10.3	24	
○ B-2, S-2	5.0	*SM*	*Silty Sand*			-	-	21.1	21	
□ B-2, S-5	12.5	*SP-SM*	*Poorly Graded Sand with Silt*			-	-	12.8	23	
△ B-2, S-9	30.0	*SP-SM*	*Poorly Graded Sand with Silt*			-	-	14.8	23	

Vancouver PFAS #14
Vancouver, Washington

GRAIN SIZE DISTRIBUTION

July 2024 112510

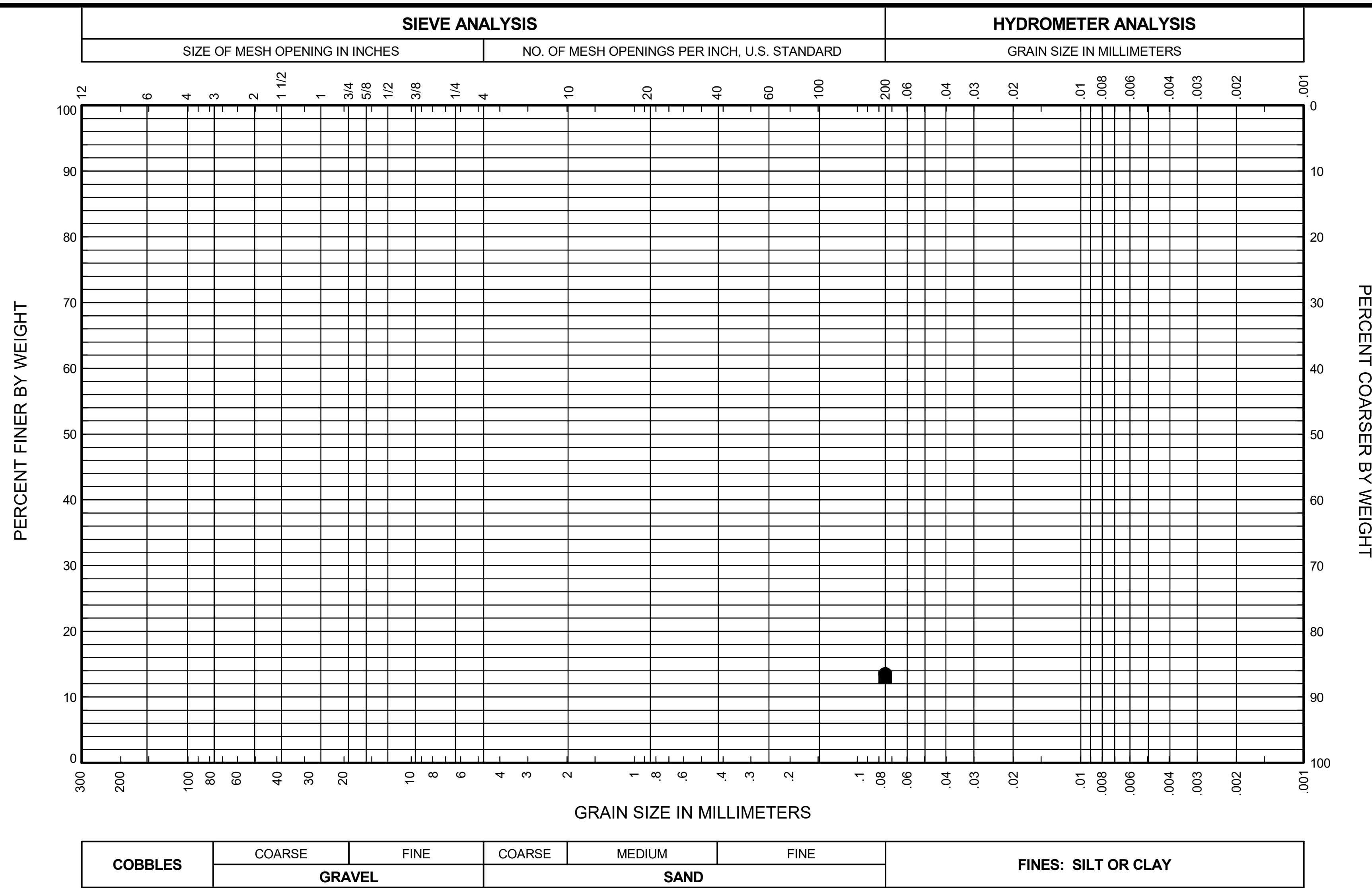
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B1
Sheet 1 of 2

NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

GSA_INTERNAL 112510.GPJ SHAN_WIL.GDT 3/7/24

FIG. B1



BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	Cc	Cu	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-2, S-13	50.0	*SP-SM*	*Poorly Graded Sand with Silt*			-	-	13.5	26	
■ B-2, S-17	70.0	*SP-SM*	*Poorly Graded Sand with Silt*			-	-	13.0	25	

Vancouver PFAS #14
Vancouver, Washington

GRAIN SIZE DISTRIBUTION

June 2024

112510

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B1
Sheet 2 of 2

NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

GSA_INTERNAL 112510.GPJ SHAN_WIL.GDT 3/7/24

FIG. B1

Corrosivity Tests Summary

COOPER TESTING LABORATORY

CTL # 645-171 **Date:** 2/29/2024 **Tested By:** PJ **Checked:** PJ
Client: Shannon & Wilson **Project:** Vancouver PFAS #14 **Proj. No:** 112510
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
B-1	S-2	5.0-6.5	7,025	-	-	3	23	0.0023	5.7	481	19	-	23.4	Brown Clayey SAND
B-2	S-1	2.5-4.0	9,584	-	-	<2	15	0.0015	6.1	498	19	-	24.0	Brown CLAY w/ Sand

Important Information

About Your Geotechnical Engineering Report

IMPORTANT INFORMATION

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the Geoprofessional Business Association (https://www.geoprofessional.org)