

**4.** Geotechnical Soil Analysis Report (Includes Geologic Hazard Study)













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November 17, 2023

HSR Development 500 E. Broadway, Suite 120 Vancouver, Washington 98660

Attn: Steven Waugh

# **Geotechnical Site Investigation**

Mills Property Camas, Washington

Columbia West Project: HSR-1-01-1

Columbia West is pleased to present this geotechnical site investigation report for the Mills Property in Camas, Washington. Our services were conducted in accordance with our proposal dated August 14, 2023.

We appreciate the opportunity to work on the project. Please contact us if you have any questions regarding this document.

Sincerely,

Columbia West

Daniel Lehto, PE, GE Principal Engineer

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Attachments

Document ID: Mills Property Geotechnical Report.docx



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#### **EXECUTIVE SUMMARY**

This executive summary presents the primary geotechnical considerations associated with the proposed Mills Property project located in Camas, Washington. Our conclusions and recommendations are based upon the subsurface information presented in this report and proposed development information provided by the design team. Detailed discussion of the geotechnical considerations summarized here is presented in respective sections of the report.

- The site is characterized by pinnacled basalt and basalt outcrops. Thin soil profiles were observed in some areas. Based on subsurface exploration, infiltration of concentrated stormwater is infeasible due to the presence of shallow bedrock and clayey residual soils.
- Although site reconnaissance and Clark County GIS Mapping indicate the presence of slopes greater than 15 percent in the southern boundary of the parcel, site slopes do not meet the definition of a landslide hazard according to Camas Municipal Code, Section 16.59.
- Excavator refusal was encountered in all test pits in the proposed development area at depths ranging from 0 to 10 Below Ground Surface (BGS). Excavations at the site will require rock excavation techniques to install necessary elements. Though not specifically encountered, seeps and springs within the basalt formation have been observed on surrounding sites and have been associated with regional rain events. Therefore, drainage design may need to be altered during the course of construction as seeps and springs become evident over time.



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# GEOTECHNICAL SITE INVESTIGATION MILLS PROPERTY CAMAS, WASHINGTON

#### 1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by HSR Development to conduct a geotechnical site investigation for the proposed Mills Property project located in Camas, Washington. The purpose of the investigation was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. This report summarizes the investigation and provides field assessment documentation and laboratory analytical test reports. This report is subject to the limitations expressed in Section 11.0, *Conclusion and Limitations*, and Appendix C.

#### 1.1 GENERAL SITE INFORMATION

As indicated on Figures 1 and 2, the subject site is located north of Lacamas Lake with an address of 313 SE Leadbetter Road in Camas, Washington. The site is accessed via SE 252<sup>nd</sup> Avenue, as there is no access to the site directly from SE Leadbetter Road. The site is comprised of tax parcel 177885000 totaling approximately 21 acres. The approximate latitude and longitude are N 45° 37′ 09″ and W 122° 24′ 57″, and the legal description is a portion of the NE ¼ of Section 34, T2N, R3E, Willamette Meridian. The regulatory jurisdictional agency is the City of Camas.

#### 1.2 PROJECT UNDERSTANDING

Based on design team correspondence, proposed development is likely to include construction of a residential subdivision. A layout of the planned infrastructure was not available at the time of this writing. Proposed development will likely include paved neighborhood access roads, essential underground utilities, and stormwater management facilities.

We anticipate maximum loads for the residential buildings will be less than 20 kips per column and 3 kips per foot for perimeter footings. Allowable total and differential static settlement tolerances for the structures are 1 inch and 0.5 inch over a 50-foot span, respectively. We also anticipate that proposed structures will be Risk Category II with a fundamental period less than 0.5 second. We should be contacted to revise our recommendations if the assumptions stated above are incorrect.

#### 2.0 SCOPE OF SERVICES

Columbia West's scope of services was outlined in a proposal dated August 14, 2023. In accordance with our proposal, we performed the following geotechnical services:

- Reviewed information available in our files from previous geological and geotechnical studies conducted at and in the vicinity of the site.
- Reviewed preliminary site plans and structural information provided by the design team.
- Conducted subsurface exploration at the site, to include:
  - Excavated 13 test pits to depths ranging from 4 to 10 feet BGS due to excavator refusal.
     Two shovel pits were also explored in an area proposed for the off-site sewer alignment.
- Collected disturbed soil samples from test pits for laboratory analysis.
- Classified and logged observed soil and groundwater conditions.
- Prepared this geotechnical site investigation report for the proposed development, which includes:



- Summary of soil index properties, regional geology, soil conditions, and observed groundwater conditions.
- Summary of geologic and seismic literature research used to evaluate relevant seismic risks, including locations of faults, earthquake magnitudes, and seismic factors from the 2018 IBC and ASCE 7-16
- o Summary of City of Camas Code Chapter 16.59 Geologically Hazardous Areas research
- Liquefaction potential
- o Fill- and load-induced settlement potential
- o Geotechnical design and construction recommendations for:
  - Shallow foundations
  - Lateral earth pressures
  - Site preparation and grading, organic stripping, fill placement and compaction, over-excavation, and construction monitoring and testing.
  - Structural fill materials, onsite soil suitability, and import aggregate specifications.
  - Utility trench excavation and backfill.
  - Drainage and management of groundwater conditions
  - Asphaltic concrete pavement construction for access roads and parking lots
  - Seismic design parameters in accordance with ASCE 7-16

#### 3.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

The subject site lies within the Willamette Valley/Puget Sound Lowland, a wide physiographic depression flanked by the mountainous Coast Range on the west and the Cascade Range on the east. Inclined or uplifted structural zones within the Willamette Valley/Puget Sound Lowland constitute highland areas and depressed structural zones form sediment-filled basins. The site is located in the central-eastern portion of the Portland/Vancouver Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide.

According to the Geologic Map of the Camas Quadrangle, Clark County, Washington, and Multnomah County, Oregon (US Geological Survey, Scientific Investigations Map 3017, 2008), near-surface geology is expected to primarily consist of upper-Oligocene basaltic-andesite of Elkhorn Mountain (Tbem). The Elkhorn Mountain Formation is comprised of a series of lava flows and flow breccia consisting of dark grey basalt and andesite.

The Web Soil Survey (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2022 Website) identifies surface soils as primarily Lauren very gravelly loam, Olympic Stoney Clay Loam, Vader Silt Loam, and a small pocket of Lauren loam along the north property boundary. Although soil conditions may vary from the broad USDA descriptions, Lauren soils are generally coarse-textured sands and gravels with moderate permeability, moderate shear strength, and low shrink-swell potential. Olympic and Vader series soils are more fine textured silt and clay with variable amounts of gravel. Olympic and Vader soils are typically fine-textured, poorly drained soils that develop over bedrock and exhibit slow permeability and have low shear strength.

#### 4.0 REGIONAL SEISMOLOGY

Recent research and subsurface mapping investigations within the Pacific Northwest appear to suggest the historic potential risk for a large earthquake event with strong localized ground movement may be underestimated. Past earthquakes in the Pacific Northwest appear to have caused



landslides and ground subsidence, in addition to severe flooding near coastal areas. Earthquakes may also induce soil liquefaction, which occurs when elevated horizontal ground acceleration and velocity cause soil particles to interact as a fluid as opposed to a solid. Liquefaction of soil can result in lateral spreading and temporary loss of bearing capacity and shear strength. Liquefaction is discussed later in Section 7.0, *Geologic Hazards* 

Three scenario earthquakes are possible with the local seismic setting. Two of the possible earthquake sources are associated with the Cascadia Subduction Zone (CSZ), and the third event is a shallow, local crustal earthquake that could occur in the North American Plate. The three earthquake scenarios are discussed below.

#### Cascadia Subduction Zone

The Cascadia Subduction Zone is a potential source of strong earthquake activity in the Portland/Vancouver Basin. This phenomenon is the result of the earth's large tectonic plate movement. Geologic evidence indicates that volcanic ocean floor activity along the Juan de Fuca ridge in the Pacific Ocean causes the Juan de Fuca Plate to perpetually move east and subduct under the North American Continental Plate. The subduction zone results in historic volcanic and potential earthquake activity in proximity to the plate interface, believed to lie approximately 20 to 50 miles west of the general location of the Oregon and Washington coast (Geomatrix Consultants, 1995).

Evidence suggests that this subduction zone has generated eight great earthquakes in the last 4,000 years, with the most recent event occurring approximately 300 years ago (Weaver and Shedlock, 1991).

Two types of subduction zone earthquakes are possible and considered in this report:

- 1 An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is capable of generating earthquakes with a moment magnitude of 9.0.
- 2 A deep intraplate earthquake on the seismogenic part of the subducting Juan de Fuca Plate. These events typically occur at depths of between 30 and 60 km. This source is capable of generating an event with a moment magnitude of up to 8.0.

#### **Crustal Events**

There are at least six major known fault zones in the vicinity of the site that may be capable of generating potentially destructive horizontal accelerations. These fault zones are described briefly in Table 1.

**Table 1. Faults Within the Site Vicinity** 

Fault Name	Proximity to Site (km) per USGS	Mapped Length (km) per USGS
Beaverton fault zone	26	15
Helvetia fault zone	24	7
Oatfield fault zone	18	29
Portland Hills fault zone	13	49
East Bank fault	13	29
Lacamas Lake fault	1	24



#### 5.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

Subsurface conditions were explored by excavating thirteen test pits (TP-1 through TP-13) using a track-mounted excavator at the approximate locations shown on Figure 2. The test pits were excavated on October 13, 2023 to a maximum depth of 10 feet BGS. Two supplemental excavations (S-1 and S-2) were explored by shovel in an area proposed for sanitary sewer alignment which will connect the subdivision sewer to SE Leadbetter Road. No soil logs were produced for explorations S-1 and S-2, as bedrock was encountered within a few inches of the surface.

Subsurface conditions were logged in accordance with the Unified Soil Classification System (USCS). Disturbed soil samples were collected at representative depth intervals. Test pit logs are presented in Appendix A. Soil descriptions and classification information are also provided in Appendix A. Analytical laboratory test results are presented in Appendix B.

#### 5.1 SURFACE INVESTIGATION AND SITE DESCRIPTION

The site is located north of Lacamas Lake with address 313 SE Leadbetter Road in Camas, Washington and consists of tax parcel 177885000 which totals approximately 21 acres. The site is accessed via SE 252<sup>nd</sup> Avenue and is bound by a dairy farm to the north, wooded private and public land to the west and south, and residential acreage parcels to the east. The site is currently undeveloped and heavily wooded. Site terrain is undulating and characterized by grades of 0 to 15 percent with some steeper areas existing where basalt outcrops create short slopes of up to 25 percent. A gravel access road has been constructed into the site and remnant overgrown logging trails were observed in a few locations.

#### 5.2 SUBSURFACE CONDITIONS

The test pits were excavated through up to 10 inches of forest duff and organic topsoil zone. Underlying the surface vegetation, residual soils and basalt bedrock were encountered. Subsurface lithology may generally be described by the soil units identified in the following text.

#### 5.2.1 Silty and Clayey Gravel Residual Soils

Underlying the surface vegetation, medium dense to dense silty SAND with gravel, clayey GRAVEL or SILT with minor gravel was observed to a maximum depth of 10 feet BGS. The moisture content of the residual soils ranged from 20 to 41 percent at the time of exploration. Atterberg limits analysis indicates a range of plasticity index from 8 to 15, indicating minor to moderate plasticity.

#### **5.2.2** Weathered to Bright Basalt Bedrock

Underlying the residual soils, dense to very dense weathered basalt or intact basalt was encountered and caused excavator refusal at depths ranging from ground surface to 10 feet BGS. Digging was extremely slow, and the formation will likely require blasting to move significant quantities of the material.

#### 5.2.3 Groundwater

Groundwater was not observed in the test pits at the time of exploration. However, our experience on adjacent sites underlain by the Elkhorn Mountain Basalt has shown that significant seeps and springs may manifest during rain events, sometimes several days after a rain. The formation may have connectivity to a more regional aquifer.



#### 6.0 INFILTRATION INFEASIBILITY

Infiltration testing was not conducted due to the minor amount of clayey gravel soil encountered at the stie. Due to the relatively impermeable nature of the basalt bedrock and residual soils, infiltration of concentrated stormwater onsite is not recommended, as this would likely result in uncontrolled runoff as the water travels along the undulated surface of the bedrock and finds new places to discharge.

#### 7.0 GEOLOGIC HAZARDS

Camas Municipal Code, Section 16.59 defines geologic hazard requirements for proposed development in areas subject to City of Camas jurisdiction. Three potential geologic hazards have been identified: (1) erosion hazard areas, (2) landslide hazard areas, and (3) seismic hazard areas. Hazard mapping provided by Clark County Maps Online indicates potential landslide hazard areas (slopes greater than 15 percent) in a few areas, primarily in the central portion of the property.

Columbia West conducted a geologic hazard review to assess whether these hazards are present at the subject property proposed for development, and if so, to provide mitigation recommendations. The geologic hazard review was based upon physical and visual reconnaissance, subsurface exploration, laboratory analysis of collected soil samples, and review of maps and other published technical literature. The results of the geologic hazard review are discussed in the following sections.

#### 7.1 EROSION HAZARDS

Camas Municipal Code, Section 16.59.020.A defines an erosion hazard as areas where slope grades meet or exceed 40 percent. Based upon review of slope grade mapping published by Clark County Maps Online, maximum slope grades of 15 to 25 percent are mapped at the site. Therefore, site slopes do not meet the definition of an erosion hazard according to Camas Municipal Code. However, implementation of proper erosion control BMPs are recommended for the site during construction and in the finished grade condition.

#### 7.2 LANDSLIDE HAZARDS

Columbia West conducted a review of available mapping, *Clark County GIS data*, and site reconnaissance to evaluate the potential presence of a landslide hazard on or near the subject site.

#### 7.2.1 Geologic Literature Review

Columbia West reviewed *Slope Stability, Clark County, Washington* (Fiksdal, 1975) to assess site slope characteristics. The Fiksdal report identifies four levels of potential instability within Clark County: (1) stable areas - no slides or unstable slopes, (2) areas of potential instability because of underlying geologic conditions and physical characteristics associated with steepness, (3) areas of historical or still active landslides, and (4) older landslide debris. The site is mapped as (1) stable - no slides or unstable slopes.

Columbia West also reviewed the *Geologic Map of the Camas Quadrangle, Clark County, Washington, and Multnomah County, Oregon (US Geological Survey, Scientific Investigations Map 3017, 2008)* which indicates that no active landslides or historic landslide deposits are mapped at the subject site or in the surrounding vicinity.



# 7.2.1 Slope Reconnaissance and Slope Stability Assessment

To observe geomorphic conditions, Columbia West personnel conducted visual and physical reconnaissance of slopes on the property. Subsurface native soils at the locations observed generally consisted of clayey gravel or silty SAND with gravel. Soil horizons appeared firm and well developed. Shallow and surficial basalt bedrock was observed throughout the site.

Site reconnaissance and review of topographic mapping published by *Clark County Maps Online* indicates that site topography is undulated and characterized by pinnacled basalt flows. Short slopes between 15 and 25 percent are present in a few areas throughout the site. Slopes appear planar with no observed evidence of instability. Most bedrock encountered was slightly to moderately weathered or intact. There was no observed direct evidence of large-scale, mass slope movements or historic landslides. No landslide debris was observed within explored site soils and groundwater seeps or springs within the face of the slopes were not observed.

Camas Municipal Code defines a landslide hazard as slopes mapped by Fiksdal as 'areas of potential instability' or areas meeting all three of the following characteristics: 1) slopes steeper than 15 percent; 2) hillsides intersecting geologic contacts with permeable sediment overlying low permeability sediment or bedrock, and; 3) any springs or groundwater seepage. The above-mentioned criteria were not observed during our field investigation or site research. Based upon the results of slope reconnaissance, subsurface exploration, and site research, in Columbia West's opinion slopes on the subject site do not meet the definition of a landslide hazard according to Camas Municipal Code.

#### 7.3 SEISMIC HAZARD AREAS

Seismic hazards include areas subject to severe risk of earthquake-induced damage. Damage may occur due to soil liquefaction, dynamic settlement, ground shaking amplification, or surface faulting rupture. These seismic hazards are discussed below.

#### 7.3.1 Soil Liquefaction and Dynamic Settlement

According to the Liquefaction Susceptibility Map of Clark County, Washington (Washington State Department of Natural Resources, 2004), the site is mapped as very low susceptibility for liquefaction due to basalt bedrock. Liquefaction, defined as the transformation of the behavior of a granular material from a solid to a liquid due to increased pore-water pressure and reduced effective stress, may occur when granular materials quickly compact under cyclic stresses caused by a seismic event. The effects of liquefaction may include immediate ground settlement, lateral spreading, and differential compaction.

Soils most susceptible to liquefaction are recent geologic deposits, such as river and floodplain sediments. These soils are generally saturated, cohesionless, loose to medium dense sands within 50 feet of ground surface. Potentially liquefiable soils located above the existing, historic, or expected ground water levels do not generally pose a liquefaction hazard. It is important to note that changes in perched ground water elevation may occur due to project development or other factors not observed at the time of investigation.

Based upon the results of subsurface exploration, literature review, and laboratory analysis, the above-mentioned criteria were not observed during the geotechnical site investigation. Therefore, the potential for soil liquefaction is considered to be very low.



#### 7.3.2 Ground Shaking Amplification

Review of the Site Class Map of Clark County, Washington (Washington State Department of Natural Resources, 2004), indicates that site soils may be represented by Site Class B as defined in 2018 IBC Section 1613.3.2. A designation of Site Class B indicates that minimal amplification of seismic energy may occur during a seismic event due to subsurface conditions. This designation does not represent a geologic hazard in Columbia West's opinion and will not prohibit development if properly accounted for during the design process. Additional seismic information is presented in Section 8.2, Seismic Design Considerations.

#### 7.3.3 Fault Rupture

Because there are no known geologic seismic faults within the site boundaries, fault rupture is unlikely.

#### 8.0 DESIGN RECOMMENDATIONS

The geotechnical site investigation suggests the proposed development is generally compatible with surface and subsurface soils, provided the recommendations presented in this report are incorporated in design and implemented during construction. Design and construction recommendations are presented in the following sections.

#### 8.1 Shallow Foundation Support

Proposed residential structures may be supported by conventional spread footings bearing on firm native soil or engineered structural fill.

Any loose or disturbed soil should be improved or removed and replaced with structural fill. If footing subgrade soils are above their optimum moisture content, we recommend that a minimum of 6 inches of compacted aggregate be placed over exposed subgrade soils. The aggregate pad should extend 6 inches beyond the edge of the foundations and consist of imported granular material as described in Section 9.6.1, *Structural Fill*. Columbia West should observe exposed subgrade conditions prior to placement of crushed aggregate to verify adequate subgrade support.

#### 8.1.1 Footing Dimensions and Bearing Capacity

Continuous perimeter wall and isolated spread footings should have minimum width dimensions of 18 and 24 inches, respectively. The base of exterior footings should bear at least 18 inches below the lowest adjacent exterior grade. The base of interior footings should bear at least 12 inches below the base of the floor.

Footings bearing on subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,500 psf. As the allowable bearing pressure is a net bearing pressure, the weight of the footing and associated backfill may be ignored when calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by 50 percent for transient lateral forces such as seismic or wind.

#### 8.1.2 Shallow Foundation Settlement

Foundation settlement is a significant structural design consideration. Provided subgrade soils are prepared as described above and in Section 9.1, *Site Preparation and Grading*, we anticipate that post-construction static foundation settlement will be less than approximately 1 inch. Differential settlement between comparably loaded foundations is not expected to exceed approximately 0.5



inch over a distance of 50 feet. The risk of differential settlement may increase where structural elements are founded upon both bedrock and soil beneath the same linear footing. Therefore, Columbia West recommends foundations be constructed on a uniform material whenever possible.

#### 8.1.3 Resistance to Sliding

Lateral foundation loads can be resisted by passive earth pressure on the sides of the footing and by friction at the base of the footings. Recommended passive earth pressure for footings confined by native soil or engineered structural fill is 250 pcf. The upper 6 inches of soil should be neglected when calculating passive pressure resistance. Adjacent floor slabs and pavement, if present, should also be neglected from the analysis. The recommended passive pressure resistance assumes that a minimum horizontal clearance of 10 feet is maintained between the footing face and adjacent downgradient slopes.

The estimated coefficient of friction between in situ native soil or engineered structural fill and in-place poured concrete is 0.35. The estimated coefficient of friction between compacted crushed aggregate and in-place poured concrete is 0.45.

#### 8.1.4 Subgrade Observation

Upon completion of stripping and prior to the placement of structural fill or pavement improvements, exposed subgrade soil should be evaluated by proof rolling with a fully-loaded dump truck or similar heavy, rubber tire construction equipment. When the subgrade is too wet for proof rolling, a foundation probe may be used to identify areas of soft, loose, or unsuitable soil. Subgrade evaluation should be performed by Columbia West. If soft or yielding subgrade areas are identified during evaluation, we recommend the subgrade be over-excavated and backfilled with compacted imported granular fill.

#### 8.1.5 Floor Slabs

Floor slabs can be supported on firm, competent, native soil or engineered structural fill prepared as described in this report. Disturbed soils and unsuitable fills in proposed slab locations, if encountered, should be removed and replaced with structural fill.

To provide a capillary break, slabs should be underlain by at least 6 inches of compacted crushed aggregate that contains less than 5 percent by weight passing the No. 200 Sieve. Geotextile may be used below the crushed aggregate layer to increase subgrade support. Recommendations for floor slab base aggregate and subgrade geotextile are discussed in Section 9.6, *Materials*.

Floor slabs with maximum floor load of 150 psf may be designed assuming a modulus of subgrade reaction, k, of 125 pci.

#### 8.2 SEISMIC DESIGN CONSIDERATIONS

Seismic design for proposed structures is prescribed by ASCE 7-16. Based on literature review and results of subsurface exploration conducted by Columbia West, site soils meet the criteria for Site Class B. Seismic design parameters for Site Class B are presented in Table 2.



Table 2. ASCE 7-16 Seismic Design Parameters<sup>1</sup>

	Short Period	1 Second Period
MCE Spectral Acceleration	0.795	0.348
Site Class	E	3
Site Coefficient	Fa = 1.0	Fv = 1.0
Adjusted Spectral Response Acceleration	S <sub>MS</sub> = 0.795	S <sub>M1</sub> = 0.348
Design Spectral Response Acceleration	S <sub>DS</sub> = 0.53	S <sub>D1</sub> = 0.232

<sup>1.</sup> The structural engineer should evaluate ASCE 7-16 code requirements and exceptions to determine if these parameters are valid for design.

As discussed in Section 7.3, Seismic Hazards Area, liquefaction and lateral spreading are not design considerations for the site.

#### 8.3 RETAINING STRUCTURES

Lateral earth pressures should be considered during design of retaining walls and below-grade structures. Hydrostatic pressure and additional surcharge loading should also be considered. Wall foundation construction and bearing capacity should adhere to specifications provided previously in Section 8.1, Shallow Foundation Support.

Permanent retaining walls that are not restrained from rotation and are retaining undisturbed native soil should be designed for active earth pressures using an equivalent fluid pressure of 39 pcf. Walls retaining undisturbed native soils that are restrained from rotation should be designed for an at-rest equivalent fluid pressure of 64 pcf. For walls with imported well-drained granular backfill meeting WSDOT 9.03.12(2), an equivalent fluid pressure of 34 pcf is applicable for active and 60 pcf for at rest is applicable.

The recommended earth pressures assume a maximum wall height of 10 feet with level backfill. These values also assume that adequate drainage is provided behind retaining walls to prevent hydrostatic pressures from developing. Lateral earth pressures induced by surcharge loads may be estimated using the criteria presented on Figure 3.

Seismic forces may be calculated by superimposing a uniform lateral force of 10H<sup>2</sup> pounds per lineal foot of wall, where H is the total wall height in feet. The force should be applied as a distributed load with the resultant located at 0.6H from the base of the wall.

#### 8.3.1 Wall Drainage and Backfill

A minimum 6-inch-diameter, perforated collector pipe should be placed at the base of retaining walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of finished grade. The drain rock and geotextile drainage fabric should meet the specifications provided in Section 9.6, *Materials*. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into



stormwater drainage systems, unless measures are taken to prevent backflow into the drainage system of the wall.

Backfill material placed behind the walls and extending a horizontal distance of ½ H, where H is the height of the retaining wall, should consist of select granular material placed and compacted as described in Section 9.6.1, *Structural Fill*.

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be delayed at least four weeks after placement of wall backfill, unless survey data indicates that settlement is complete prior to that time.

#### 8.4 PAVEMENT RECOMMENDATIONS

We understand that public roadways for the subdivision will be constructed in accordance with City of Camas standards. For dry weather construction, pavement surface sections should bear upon competent subgrade consisting of scarified and compacted native soil or engineered structural fill. Wet weather construction may require an increased thickness of base aggregate as discussed later in Section 9.2, Construction Traffic and Staging.

In general, AC paving is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress. Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix, as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction.

If AC paving must take place during cold-weather construction as defined in this section, the contractor and design team should discuss options for minimizing risk to pavement serviceability.

#### 8.5 DRAINAGE

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. Drainage design in general should conform to City of Camas regulations. Finished site grading should be conducted with positive drainage away from structures at a minimum 2 percent slope for a distance of at least 10 feet. Depressions or shallow areas that may retain ponding water should be avoided.

Recommendations for foundation drains and subdrains are presented in the following sections. Drain rock and geotextile drainage fabric should meet the requirements presented in Section 9.6, *Materials*. Drains should be closely monitored after construction to assess their effectiveness. If additional surface or shallow subsurface seeps become evident, the drainage provisions may require modification or additional drains. We should be consulted to provide appropriate recommendations.

#### 8.5.1 Foundation Drains

Roof drains are recommended for all structures. Perimeter building foundation drains should be considered for shallow foundations constructed below existing site grades but are not necessary for the functionality of the buildings.



Foundation and roof drains, where installed, should consist of separate systems that gravity flow away from foundations to an approved discharge location. Perimeter foundation drains should consist of 4-inch perforated PVC pipe surrounded by a minimum 2-foot-wide zone of clean, washed drain rock wrapped with geotextile drainage fabric. The wrapped drain rock zone should extend up the sides of embedded walls to within 12 inches of proposed finished grade. Foundation drains should be constructed with a minimum slope of ½ percent. The drainpipe's invert elevation should be at least 18 inches below the elevation of the floor slab. Figure 4 presents a typical foundation drain detail.

#### 8.5.2 Subdrains

Subdrains should be considered if portions of the site are cut below surrounding grades. Shallow groundwater or seeps should be conveyed via drainage channel or perforated pipe into an approved discharge. Recommendations for design and installation of perforated drainage pipe may be performed on a case-by-case basis by Columbia West during construction. Failure to provide adequate surface and sub-surface drainage may result in soil slumping or unanticipated settlement of structures exceeding tolerable limits. A typical perforated drainpipe trench detail is presented in Figure 5.

#### 8.5.2 Drainage Mat

Site improvements construction in some areas may occur at or near the shallow groundwater table, particularly if work is conducted during wet-weather conditions. A drainage mat is typically required in areas that require structural fill placed on top of a natural drainage swale or known seep or spring area. Dewatering may be necessary, and a drainage mat may be required to achieve sufficient elevation for fill placement. A typical drainage mat is shown on Figure 6. Columbia West should determine drainage mat location, extent, and thickness when subsurface conditions are exposed. Drainage mats may need to be constructed in conjunction with subdrains to convey captured water to an approved discharge location

#### 9.0 CONSTRUCTION RECOMMENDATIONS

#### 9.1 SITE PREPARATION AND GRADING

Site vegetation primarily consisted of up to 10 inches of forest duff at the time of our exploration. Thicker root zones may be present in areas of mature trees and shrub growth. Where encountered, pavement, vegetation, organic material, unsuitable fill, and deleterious material should be cleared from areas identified for structures and site grading. Vegetation, root zones, organic material, and debris should be removed from the site. Stripped topsoil should also be removed or used only as landscape fill in nonstructural areas with slopes less than 25 percent. The post-construction maximum depth of landscape fill placed or spread at any location onsite should not exceed one foot. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed.

#### 9.1.1 Subgrade Evaluation

Upon completion of stripping and prior to the placement of structural fill or pavement improvements, exposed subgrade soil should be evaluated by proof rolling with a fully-loaded dump truck or similar heavy, rubber tire construction equipment. When the subgrade is too wet for proof rolling, a foundation probe may be used to identify areas of soft, loose, or unsuitable soil.



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Subgrade evaluation should be performed by Columbia West. If soft or yielding subgrade areas are identified during evaluation, we recommend the subgrade be over-excavated and backfilled with compacted imported granular fill.

#### 9.2 CONSTRUCTION TRAFFIC AND STAGING

Where encountered, near-surface fine-textured soils will be easily disturbed during construction. This includes native clayey soils, as well as potential imported fine-textured soils. If not carefully executed, site preparation, excavation, and grading can create extensive soft areas resulting in significant repair costs. Earthwork planning should include considerations for minimizing subgrade disturbance, particularly during wet-weather conditions.

If construction occurs during wet-weather conditions, or if the moisture content of the surficial soil is more than a few percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Under these conditions, granular haul roads and staging areas will also be necessary to provide a firm support base and sustain construction equipment.

The recommended base aggregate thickness for pavement sections is intended to support post-construction design traffic loads and will not provide adequate support for construction traffic. Staging areas and haul roads will require an increased base thickness during wet weather conditions. The configuration of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's means and methods. Therefore, design and construction of staging areas and haul roads should be the responsibility of the contractor. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul road areas. In areas of heavy construction traffic, geotextile separation fabric may be placed between the subgrade soil and imported granular material to increase subgrade support and minimize fines migration into the base aggregate layer.

Project stakeholders should understand that wet weather construction is risky and costly. Proper construction methods and techniques are critical to overall project integrity and should be observed and documented by Columbia West.

#### 9.3 CUT AND FILL SLOPES

Fill slopes should consist of structural fill material as discussed in Section 9.6.1, *Structural Fill*. Fill placed on existing grades steeper than 5H:1V should be horizontally benched at least 10 feet into the slope. Fill slopes greater than six feet in height should be vertically keyed into existing subsurface soil. A typical fill slope cross-section is shown in Figure 7. Drainage implementations, including subdrains or perforated drainpipe trenches, may also be necessary in proximity to cut and fill slopes if seeps or springs are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by Columbia West during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion.

Final cut or fill slopes at the site should not exceed 2H:1V or 10 feet in height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 8.



Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Fill slopes should be overbuilt, compacted, and trimmed at least two feet horizontally to provide adequate compaction of the outer slope face. Proper cut and fill slope construction is critical to overall project stability and should be observed and documented by Columbia West.

#### 9.4 EXCAVATION

The site was explored to a maximum depth of 10 feet BGS with an excavator. Weathered and competent basalt bedrock was encountered as shallow as 1.5 feet BGS onsite (TP-11) and at the ground surface in the off-site sewer alignment (S-1 and S-2) and all test pits terminated by refusal on competent bedrock. Intact basalt was observed at or near the surface throughout the site. Excavation at the site will require blasting or special rock excavation techniques to alter the surface.

Groundwater was not encountered, however the basalt formation is known to have seeps and springs show themselves after periods of wet weather, sometimes several days after an event. Recommendations as described in Section 9.5, *Dewatering*, should be considered where subsurface construction activities intersect shallow groundwater table.

Temporary excavation sidewalls should maintain a vertical cut to a depth of approximately 4 feet in the near-surface clay, provided groundwater seepage is not present in the sidewalls. In sandy soil, excavations will likely slough and cave, even at shallow depths. Open-cut excavation techniques may be used to excavate trenches between 4 and 8 feet deep, provided the walls of the excavation are cut at a maximum slope of 1H:1V and groundwater seepage is not present. Excavation slopes should be reduced to 1.5H:1V or 2H:1V if excessive sloughing or raveling occurs.

Shoring may be required if open-cut excavations are infeasible or if excavations are proposed adjacent to existing infrastructure. Typical methods for stabilizing excavations consist of solider piles and timber lagging, sheet pile walls, tiebacks and shotcrete, or prefabricated hydraulic shoring. As a wide variety of shoring and dewatering systems are available, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

The contractor should be held responsible for site safety, sloping, and shoring. All excavation activity should be conducted in accordance with applicable OSHA requirements. Columbia West is not responsible for contractor activities and in no case should excavation be conducted in excess of applicable local, state, and federal laws.

#### 9.5 DEWATERING

Groundwater was not observed at the time of our investigation. Seeps and springs are likely to be encountered during periods of wet weather. Generalized recommendations for temporary construction dewatering are presented in the following section.

#### 9.5.1 Construction Dewatering

The contractor should be responsible for temporary drainage of surface water, perched water, and groundwater. Dewatering should be performed to the extent necessary to prevent standing water and/or erosion of exposed site soils. During rough and finished grading of building pad areas, the contractor should keep all footing excavations and slab subgrade soils free of standing water.



The contractor's proposed dewatering plan should be capable of maintaining groundwater levels at least two feet below the base of proposed trench excavations. Without adequate trench dewatering, running soil, caving, and sloughing will increase backfill volumes and may result in damage to adjacent structures or utilities. Significant pumping and dewatering may be required to temporarily reduce the groundwater elevation to the recommended depth. Dewatering via a sump within excavation zones may be insufficient to control groundwater and provide excavation side slope stability. Dewatering may be more feasibly conducted by installing a system of temporary well points and pumps around proposed excavation areas or utility trenches. Depending on proposed utility depths, a site-specific dewatering plan may be necessary.

If groundwater is present at the base of utility excavations, we recommend placing 18 to 24 inches of stabilization material at the base of the excavation. Subgrade geotextile placed directly over trench subgrade soils may reduce the required thickness of the stabilization material. The actual thickness of stabilization material should be determined at the time of construction based on observed field conditions. Trench stabilization material should be placed in one lift and compacted until well keyed. Stabilization material and geotextile fabric should meet the requirements presented in Section 9.6, *Materials*.

#### 9.6 MATERIALS

#### 9.6.1 Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in Section 9.1, *Site Preparation and Grading*. Engineered fill placement should be observed by Columbia West. Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with *ASTM D6938*. Field compaction testing should be performed for each vertical foot of engineered fill placed.

Various materials may be acceptable for use as structural fill. Structural fill should be free of organic material or other unsuitable material and meet specifications provided in the following sections. Representative samples of proposed engineered structural fill should be submitted for laboratory analysis and approval by Columbia West prior to placement.

#### 9.6.1.1 Onsite Soil

Minimal residual soil was encountered during the site investigation. Most onsite soil will be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native clay soil with a plasticity index greater than 25, if encountered, should be evaluated and approved by Columbia West prior to use as structural fill. Laboratory analysis indicated that the moisture content of the near-surface clay was above optimum at the time of exploration. Moisture conditioning will likely be necessary to dry the soil prior to applying compaction effort. In addition, the near-surface clay will be moisture sensitive and difficult, if not impossible, to compact during wet weather conditions. Therefore, structural fill placement using onsite soil should be performed during dry summer months if possible. Onsite soil may also require addition of moisture during extended periods of dry weather.

If significant grading is proposed, blasting or specialized rock excavation techniques will be required in areas of shallow bedrock. Due to the minor weathering of much of the basalt encountered, blast



spoils will likely require further crushing to reduce particle size and produce a well-graded product suitable for reuse as structural fill.

Onsite soil used as structural fill should be placed in loose lifts not exceeding 8 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within a few percentage points of optimum conditions. The soil should be compacted to at least 95 percent of maximum dry density as determined by the modified Proctor moisture-density relationship test (ASTM D1557). Compacted onsite fill soils should be covered shortly after placement.

#### 9.6.1.2 Imported Granular Material

Imported granular material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material should also be durable, angular, and fairly well graded between coarse and fine material; should have less than 5 percent fines (material passing the U.S. Standard No. 200 sieve) by dry weight; and should have at least two mechanically fractured faces. Imported granular material should be placed in loose lifts not exceeding 12 inches in depth and compacted to at least 95 percent of maximum dry density as determined by the modified Proctor moisture-density relationship test (ASTM D1557). During wet-weather conditions or where wet subgrade conditions are present, the initial loose lift of granular fill should be approximately 18 inches thick and should be compacted with a smooth-drum roller operating in static mode.

#### 9.6.1.3 Stabilization Material

Stabilization material should consist of durable, 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is free of organics and other deleterious material. The material should have a maximum particle size of 6 inches with less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve. The material should have at least two mechanically fractured faces.

Stabilization material should be placed in loose lifts between 12 and 24 inches thick and be compacted to a firm, unyielding condition. Equipment with vibratory action should not be used when compacting stabilization material over wet, fine-textured soils. If stabilization material is used to stabilize soft subgrade below pavement or construction haul roads, a subgrade geotextile should be placed as a separation barrier between the soil subgrade and the stabilization material.

#### 9.6.1.4 Trench Backfill

Trench backfill placed below, adjacent to, and up to at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material meeting WSDOT 9-03.12(3) specifications for *Gravel Backfill for Pipe Zone Bedding*. Pipe zone backfill should be compacted to at least 90 percent of maximum dry density, as determined by the modified Proctor moisture-density relationship test (ASTM D1557), or as required by the local jurisdictional agency or pipe manufacturer.

Within structural areas (below pavement and building pads), trench backfill above the pipe zone should consist of WSDOT 9-03.19 Bank Run Gravel for Trench Backfill or WSDOT 9-03.14(2) Select Borrow with a maximum particle size of 2 ½-inches. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). Remaining trench backfill should be compacted to at least 95 percent of the maximum dry density as



determined by the modified Proctor moisture-density relationship test (ASTM D1557), or as required by the local jurisdictional agency or pipe manufacturer.

Outside of structural areas, trench backfill placed above the pipe zone should be compacted to at least 90 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test (ASTM D1557), or as required by the local jurisdictional agency or pipe manufacturer.

#### 9.6.1.5 Floor Slab Base Aggregate

Base aggregate for building floor slabs should consist of 1 ¼"-minus crushed aggregate meeting WSDOT 9-03.9(3) specifications for Crushed Surfacing. Slab base aggregate should be compacted to at least at least 95 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test (ASTM D1557).

#### 9.6.2 Pavement Base Aggregate

Base aggregate for pavement should consist of 1 ¼"-minus crushed aggregate meeting WSDOT 9-03.9(3) specifications for Crushed Surfacing. Pavement base aggregate should be compacted to at least at least 95 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test (ASTM D1557).

#### 9.6.2.1 Retaining Wall Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½ H, where H is the height of the retaining wall, should consist of free-draining granular material meeting WSDOT 9-03.12(2) specifications for *Gravel Backfill for Walls*. The wall backfill should be separated from structural fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

Wall backfill located within a horizontal distance of 3 feet from the face of a retaining wall should be compacted to 90 percent of the maximum dry density, as determined by *ASTM D1557*. Backfill placed within 3 feet of the wall should be compacted in loose lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). Remaining wall backfill should be compacted to at least 95 percent of the maximum dry density, as determined by *ASTM D1557*.

#### 9.6.2.2 Retaining Wall Leveling Pad

Crushed aggregate used as a leveling pad for retaining wall footings should consist of 1 ¼"-minus crushed aggregate meeting WSDOT 9-03.9(3) specifications for Crushed Surfacing. The leveling pad material should be compacted to at least 95 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test (ASTM D1557).

#### 9.6.2.3 **Drain Rock**

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and less than 2 percent by weight passing the No. 200 sieve. Drain rock should be free of roots, organic debris, and other unsuitable material and should have at least two mechanically fractured faces. Drain rock should be compacted to a firm, unyielding condition. Drain rock should be completely wrapped in a geotextile drainage fabric meeting the requirements presented below.



#### 9.6.3 Geotextile Fabric

#### 9.6.3.1 Subgrade Geotextile

Subgrade geotextile should meet the specifications provided in WSDOT 9-33.2(1), Table 3, Geotextile for Separation or Soil Stabilization. The geotextile should be installed in accordance with the manufacturer's recommendations. A minimum initial aggregate base lift of 6 inches is required over geotextiles. All stabilization material should be underlain by a subgrade geotextile.

#### 9.6.3.2 Drainage Geotextile

Subgrade geotextile should meet the specifications provided in WSDOT 9-33.2(1), Table 2, Geotextile for Underground Drainage Filtration Properties. The AOS should be between the No. 70 and No. 100 sieve. The water permittivity should be greater than 1.5/sec. The geotextile should be installed in accordance with the manufacturer's recommendations. A minimum initial aggregate base lift of 6 inches is required over geotextiles.

#### 9.6.4 Geotextile Fabric

### 9.6.4.1 Subgrade Geotextile

Subgrade geotextile should meet the specifications provided in WSDOT 9-33.2(1), Table 3, Geotextile for Separation or Soil Stabilization. The geotextile should be installed in accordance with the manufacturer's recommendations. A minimum initial aggregate base lift of 6 inches is required over geotextiles. All stabilization material should be underlain by a subgrade geotextile.

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#### 9.6.5 Pavement

#### 9.6.5.1 Asphaltic Concrete

Asphaltic concrete should consist of HMA Class ½" adhering to WSDOT 9-03.8(6), HMA Proportions of Materials. The asphalt binder should consist of PG 58-22 meeting WSDOT 9-02.1(4), Performance Graded (PG) Asphalt Binder. Asphalt should be compacted to 91 percent of the theoretical maximum density as determined by ASTM D2041. Minimum and maximum asphalt lift thicknesses should be 2 and 3 inches, respectively. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with WSDOT and City of Camas specifications.

#### 9.7 EROSION CONTROL MEASURES

Soil at this site is susceptible to erosion by wind and water; therefore, erosion control measures should be carefully planned and installed before construction begins. Surface water runoff should be collected and directed away from sloped areas to prevent water from running down the slope face. Measures that can be employed to reduce erosion include the use of silt fences, hay bales, buffer



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zones of natural growth, sedimentation ponds, and granular haul roads. All erosion control methods should be in accordance with local jurisdiction standards.

#### 10.0 OBSERVATION OF CONSTRUCTION

Satisfactory earthwork and foundation performance depends to a large degree on the quality of construction. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated. In addition, sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications.

#### 11.0 CONCLUSIONS AND LIMITATIONS

This geotechnical site investigation report was prepared in accordance with accepted standard conventional principles and practices of geotechnical engineering. This investigation pertains only to material tested and observed as of the date of this report and is based upon proposed site development as described in the text herein. This report is a professional opinion containing recommendations established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. Soil conditions may differ between tested locations or over time. Slight variations may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent QA/QC construction observation and testing to verify soil conditions are as anticipated in this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Columbia West cannot accept responsibility for deviations from recommendations described in this report. Future performance of structural facilities is often related to the degree of construction observation by qualified personnel. These services should be performed to the full extent recommended.

This report is not an environmental assessment and should not be construed as a representative warranty of site subsurface conditions. The discovery of adverse environmental conditions, or subsurface soils that deviate from those described in this report, should immediately prompt further investigation. The above statements are in lieu of all other statements expressed or implied.

Sincerely,

Columbia West Engineering, Inc.

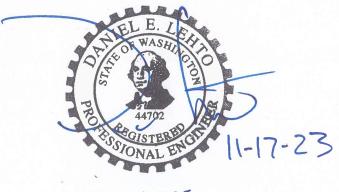
Daniel E. Lehto, PE, GE

Principal

**DEL:ASR** 

Document ID: Mills Property Geotechnical Report.docx





Expires: 6-5-25

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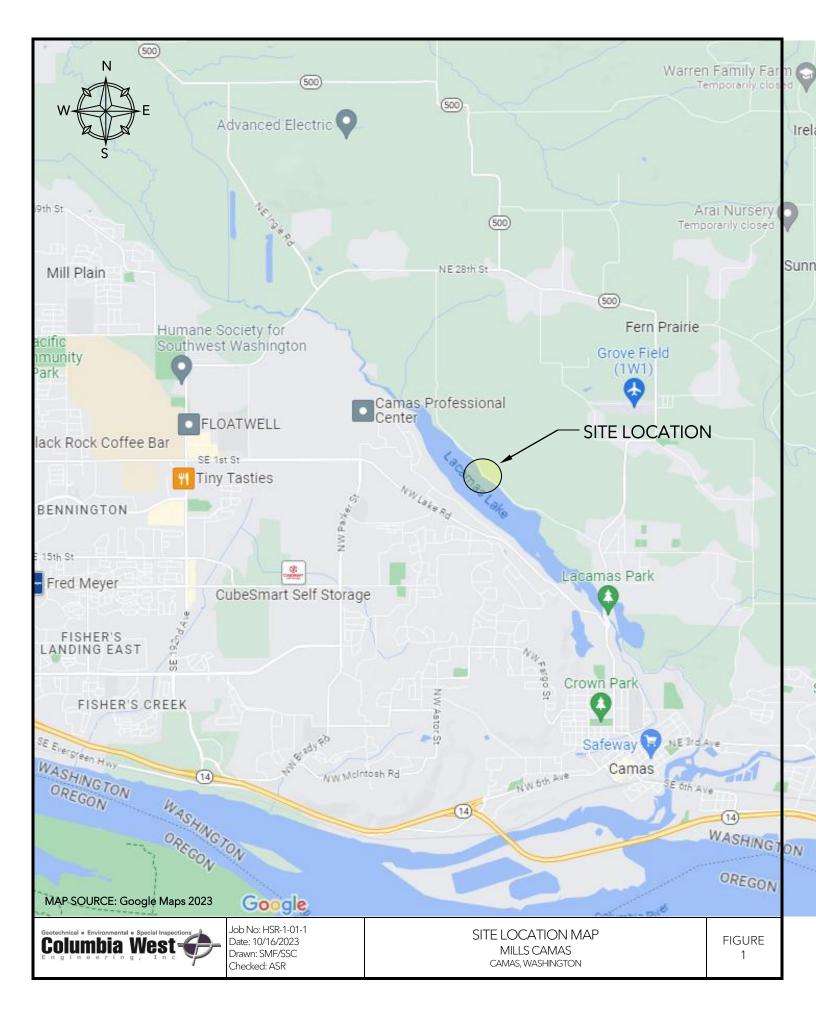
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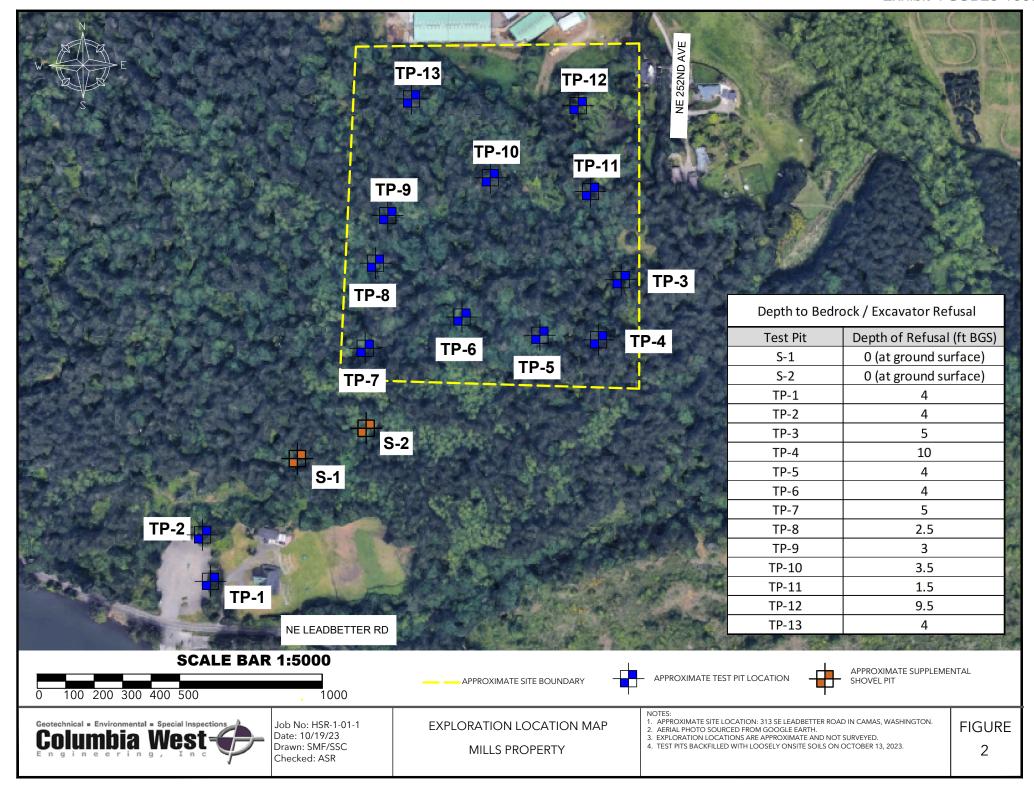
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# **FIGURES**

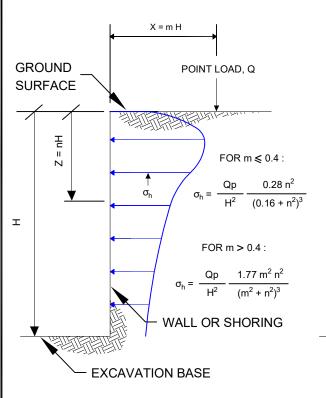


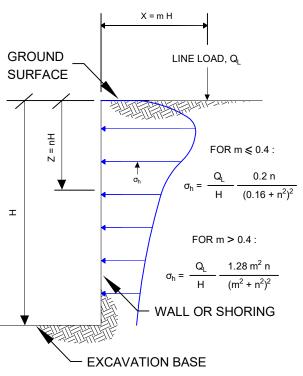


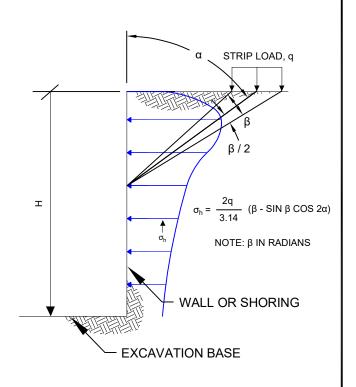


#### LINE LOAD PARALLEL TO WALL

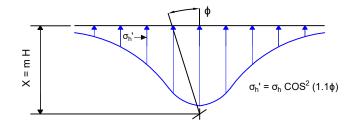
#### STRIP LOAD PARALLEL TO WALL





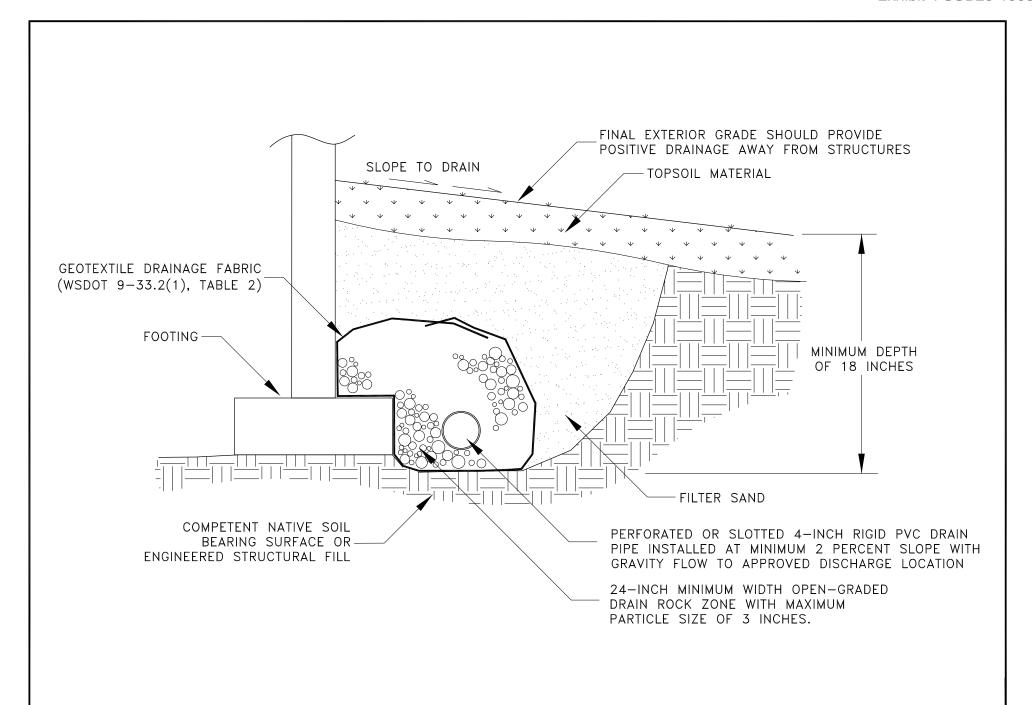


# VERTICAL POINT LOAD HORIZONTAL PRESSURE DISTRIBUTION

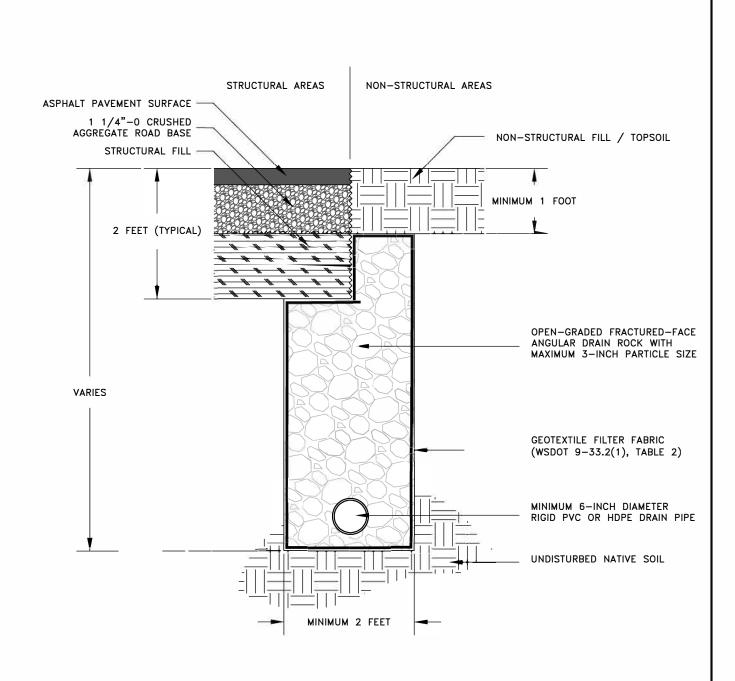


#### NOTES:

- 1. FIGURE SHOULD BE USED JOINTLY WITH RECOMMENDATIONS PRESENTED IN THE REPORT TEXT.
- 2. LATERAL EARTH PRESSURES ASSUME RIGID WALLS WITH BACKFILL MATERIALS HAVING A POISSON'S RATIO OF 0.5.
- 3. TOTAL LATERAL EARTH PRESSURES RESULTING FROM COMBINED LOADS MAY BE CALCULATED USING SUPERPOSITION.
- 4. DRAWING IS NOT TO SCALE.

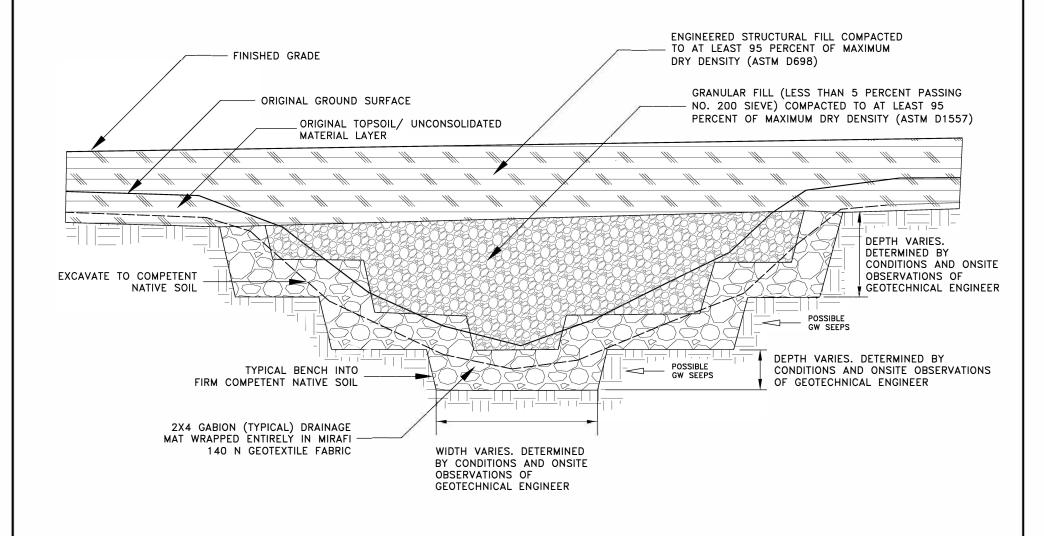






NOTE: LOCATION, INVERT ELEVATION, DEPTH OF TRENCH, AND EXTENT OF PERFORATED PIPE REQUIRED MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER DURING CONSTRUCTION BASED UPON FIELD OBSERVATION AND SITE—SPECIFIC SOIL CONDITIONS.

#### TYPICAL DRAINAGE MAT CROSS-SECTION





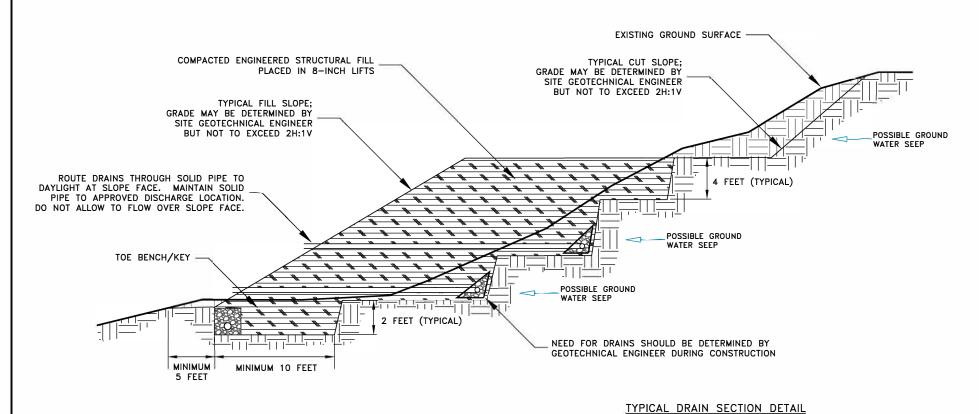
TYPICAL DRAINAGE MAT CROSS SECTION

NOTES

1. DRAWING IS NOT TO SCALE.

2. DRAWING REPRESENTS TYPICAL DRAINAGE MAT SECTION AND MAY NOT BE SITE-SPECIFIC. **FIGURE** 

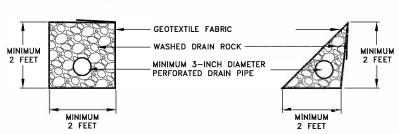
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#### DRAIN SPECIFICATIONS

GEOTEXTILE FABRIC SHALL MEET WSDOT 9-33.2(1), TABLE 2, GEOTEXTILE FOR UNDERGROUND DRAINAGE FILTRATION PROPERTIES WITH AOS BETWEEN No. 70 AND No. 100 SIEVE. WATER PERMITIVITY SHOULD BE GREATER THAN 1.5/SEC.

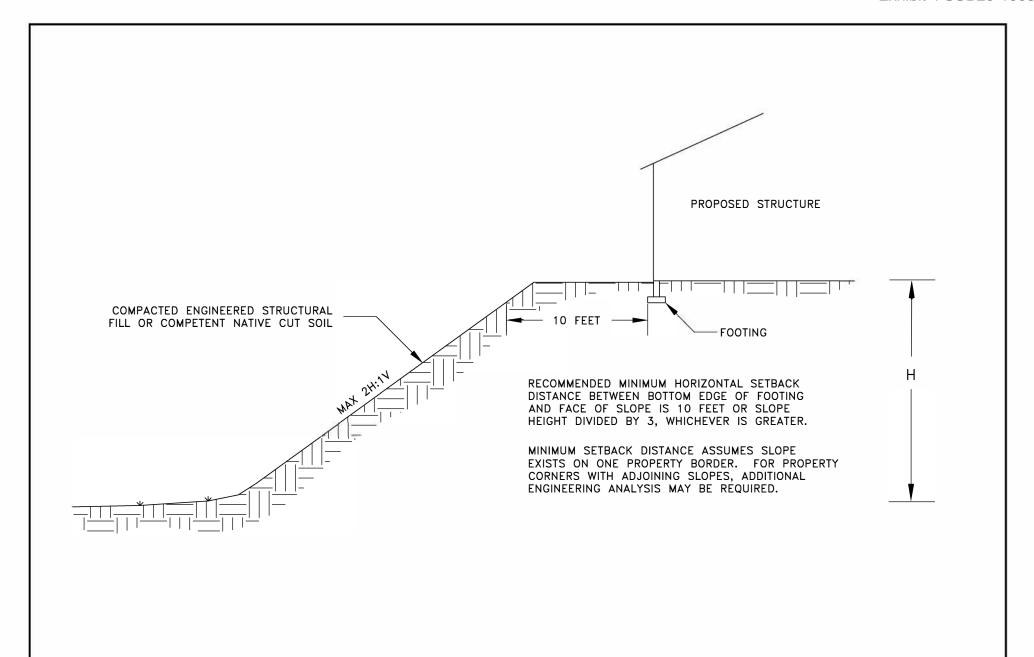
WASHED DRAIN ROCK SHALL BE OPEN-GRADED ANGULAR DRAIN ROCK WITH LESS THAN 2 PERCENT PASSING THE No. 200 SIEVE AND A MAXIMUM PARTICLE SIZE OF 2 INCHES.





<sup>1.</sup> DRAWING IS NOT TO SCALE.

DRAWING REPRESENTS TYPICAL CUT AND FILL SLOPE CROSS SECTION AND MAY NOT BE SITE—SPECIFIC.



NOTES:

1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FOUNDATION
SETBACK DETAIL AND MAY NOT BE SITE—SPECIFIC.

# APPENDIX A SUBSURFACE EXPLORATION PROGRAM FIELD EXPLORATIONS

#### **GENERAL**

We explored subsurface conditions at the site by excavating thirteen test pits (TP-1 through TP-13) to depths between 1.5 and 10 feet BGS. Excavation services were provided by L&S Contractors of Battle Ground, Washington on October 13, 2023. The test pit locations are shown in Figures 2. The test pit logs are presented in this appendix.

#### **SOIL SAMPLING**

Representative grab samples of the soil observed in the test pit explorations were obtained from the walls and/or base of the test pits using the excavator bucket.

#### **SOIL CLASSIFICATION**

The soil samples were classified in accordance with the Unified Soil Classification System presented in Appendix A. The exploration log indicates the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration log.



# **EXPLORATION LEGEND**

Symbol	Description		
SPT	Sample obtained from the indicated depth in general accordance with ASTM D1586, Standard Penetration Test and Split-Barrel Sampling of Soils		
SHELBY	Sample obtained from the indicated depth using thin-wall Shelby tube in general accordance with ASTM D1587, Thin-Walled Tube Sampling of Fine-Grained Soils		
D&M 300	Sample obtained from the indicated depth using Dames & Moore sampler and 300-pound hammer or pushed		
D&M 140	Sample obtained from the indicated depth using Dames & Moore sampler and 140-pound hammer or pushed		
CSS	Sample obtained from the indicated depth using 3-inch-outer-diameter California split-spoon sampler and 140-pound hammer		
GRAB	Grab sample obtained from the indicated depth	Graphical Log of Subsurface Lithology	
CORE	Rock core interval at the indicated depth	Observed contact at the indicated depth	
	Water level observed during exploration	Inferred contact at the indicated depth	

	Geotechnical Acronyms			
AASHTO	American Association of State Highway and Transportation Officials	Р	Push Sample	
ASTM	American Society for Testing and Materials	PP	Pocket Penetrometer	
ATT	Atterberg Limits	PSF	Pounds Per Square Foot	
BGS	Below Ground Surface	P200	Percent Passing No. 200 Sieve	
CBR	California Bearing Ratio	RES	Resilient Modulus	
CON	Consolidation Test	SIEV	Sieve Analysis	
DCPT	Dynamic Cone Penetration Test	SPT	Standard Penetration Test	
DD	Dry Density	TS	Torvane Shear	
DS	Direct Shear	UC	Unconfined Compressive Strength	
HYD	Hydrometer	UU	Unconsolidated Undrained Triaxial Test	
IR	Infiltration Rate	USCS	United Soil Classification System	
МС	Moisture Content	VS	Vane Shear	
MD	Moisture-Density Relationship	WD	Wet Density	
ос	Organic Content			

## SOIL DESCRIPTION AND CLASSIFICATION

### **Particle-Size Classification**

COMPONENT	ASTM / USCS		AASHTO		
	size range	sieve size range	size range	sieve size range	
Boulders	Greater than 300 mm	Greater than 12 inches	-	-	
Cobbles	75 mm to 300 mm	3 inches to 12 inches	Greater than 75 mm	Greater than 3 inches	
Gravel	75 mm to 4.75 mm	3 inches to No. 4 sieve	75 mm to 2.00 mm	3 inches to No. 10 sieve	
Coarse	75 mm to 19.0 mm	3 inches to 3/4-inch sieve	-	-	
Fine	19.0 mm to 4.75 mm	3/4-inch to No. 4 sieve	-	-	
Sand	4.75 mm to 0.075 mm	No. 4 to No. 200 sieve	2.00 mm to 0.075 mm	No. 10 to No. 200 sieve	
Coarse	4.75 mm to 2.00 mm	No. 4 to No. 10 sieve	2.00 mm to 0.425 mm	No. 10 to No. 40 sieve	
Medium	2.00 mm to 0.425 mm	No. 10 to No. 40 sieve	-	-	
Fine	0.425 mm to 0.075 mm	No. 40 to No. 200 sieve	0.425 mm to 0.075 mm	No. 40 to No. 200 sieve	
Fines (Silt and Clay)	Less than 0.075 mm	Passing No. 200 sieve	Less than 0.075 mm	Passing No. 200 sieve	

### **Consistency for Cohesive Soil**

CONSISTENCY	SPT N-VALUE (BLOWS PER FOOT)	D&M N-VALUE (BLOWS PER FOOT)	POCKET PENETROMETER (UNCONFINED COMPRESSIVE STRENGTH, tsf)
Very Soft	Less than 2	Less than 3	Less than 0.25
Soft	2 to 4	3 to 6	0.25 to 0.50
Medium Stiff	4 to 8	6 to 12	0.50 to 1.0
Stiff	8 to 15	12 to 25	1.0 to 2.0
Very Stiff	15 to 30	25 to 65	2.0 to 4.0
Hard	30 to 60	65 to 145	Greater than 4.0
Very Hard	Greater than 60	Greater than 145	-

### **Relative Density for Granular Soil**

RELATIVE DENSITY	SPT N-VALUE (BLOWS PER FOOT)	D&M N-VALUE (BLOWS PER FOOT)
Very Loose	0 to 4	0 to 11
Loose	4 to 10	11 to 26
Medium Dense	10 to 30	26 to 74
Dense	30 to 50	74 to 120
Very Dense	Greater than 50	Greater than 120

### **Moisture Designations**

### **Additional Constituents**

TERM	FIELD IDENTIFICATION	
Dry	No moisture. Dusty or dry.	
Damp	Some moisture. Cohesive soils are usually below plastic limit and are moldable.	
Moist	Grains appear darkened, but no visible water is present. Cohesive soils will clump. Sand will bulk. Soils are often at or near plastic limit.	1 -
Wet	Visible water on larger grains. Sand and silt exhibit dilatancy. Cohesive soil can be readily remolded. Soil leaves wetness on the hand when squeezed. Soil is much wetter than optimum moisture content and is above plastic limit.	_

	Silt and Cla	ay In:		Sand and Gravel In:		
Percent	Fine- Grained Soil	Coarse- Grained Soil	Percent	Fine-Grained Soil	Coarse- Grained Soil	
< 5	trace	trace	< 5	trace	trace	
5 – 12	minor	with	5 – 15	minor	minor	
> 12	some	silty/clayey	15 – 30	with	with	
			> 30	sandy/gravelly	with (approx. percentage)	

### **AASHTO SOIL CLASSIFICATION SYSTEM**

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

General Classification	(35 Pei	Granular Materi		Silt-Clay Materials (More than 35 Percent Passing 0.075)			
Group Classification	A-1	A-3	A-2	A-4	A-5	A-6	A-7
Sieve analysis, percent passing:							
2.00 mm (No. 10)	-	-	-				
0.425 mm (No. 40)	50 max	51 min	-	-	-	-	-
0.075 mm (No. 200)	25 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No	o. 40 <u>)</u>						
Liquid limit				40 max	41 min	40 max	41 min
Plasticity index	6 max	N.P.		10 max	10 max	11 min	11 min
General rating as subgrade	Excellent to good				Fai	r to poor	

Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

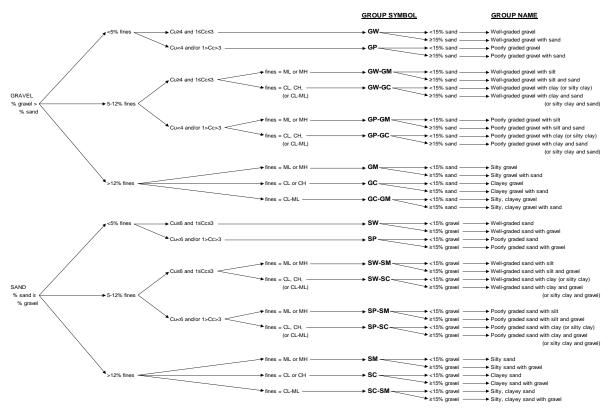
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

				Granular M	aterials				Silt-0	Clay Materials	s	
General Classification		(35 Percent or Less Passing 0.075 mm)						(More tha	(More than 35 Percent Passing 0.075 mm)			
	<u> </u>	<b>\-1</b>			А	2					A-7	
											A-7-5,	
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-6	
Sieve analysis, percent passing:												
2.00 mm (No. 10)	50 max	-	-	-	-	-	-	-	-	-	-	
0.425 mm (No. 40)	30 max	50 max	51 min	-	-	-	-	-	-	-	-	
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min	
Characteristics of fraction passing 0.425 mm (No.	40)											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min	
Plasticity index	6	max	N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11min	
Usual types of significant constituent materials	Stone	fragments,	Fine									
	grave	el and sand	sand	(	Silty or clayey	gravel and sa	and	Sil	ty soils	Clay	ey soils	
General ratings as subgrade				Excellent to	Good				Fai	r to poor		

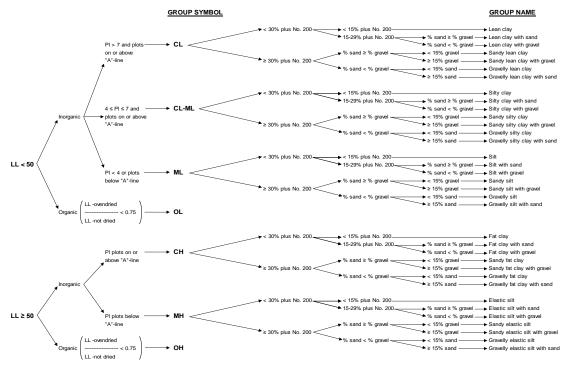
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

## **UNIFIED SOIL CLASSIFICATION SYSTEM**



Flow Chart for Classifying Coarse-Grained Soils (More Than 50% Retained on No. 200 Sieve)



Flow Chart for Classifying Fine-Grained Soil (50% or More Passes No. 200 Sieve)

### **ROCK CLASSIFICATION SYSTEM**

STRENGTH	DESCRIPTION	UNCONFINED COMPRESSIVE STRENGTH (PSI)
Extremely Weak (R0)	Easily indented by thumbnail	35 to 150
Very Weak (R1)	Scratched with fingernail, peeled by knife, indented by rock pick	150 to 275
Weak (R2)	Peeled by knife, indented by rock pick	725 to 3,500
Medium Strong (R3)	Cannot be peeled or scraped with a knife	3,500 to 7,250
Strong (R4)	Requires more than one blow with a rock hammer to fracture it	7,250 to 14,500
Very Strong (R5)	Requires many blows with a rock hammer to fracture it	14,500 to 36,250
Extremely Strong (R6)	Can only be chipped with a rock hammer	Greater than 36,250

WEATHERING	DESCRIPTION
Decomposed	A soil formed in place with original texture of rock destroyed
Completely Weathered	Rock wholly weathered but rock texture preserved
Highly Weathered	Rock weakened so that large pieces can be broken by hand
Moderately Weathered	Rock mass is decomposed locally
Slightly Weathered	Discoloration along discontinuities
Fresh	No visible signs of weathering or discoloring

JOINT SPACING	DESCRIPTION
Very Close	Less than 0.2 foot
Close	0.2 foot to 1 foot
Moderately Close	1 foot to 3 feet
Wide	3 feet to 10 feet
Very Wide	Greater than 10 feet

FRACTURING	FRACTURE SPACING
Very Intensely Fractured	Chips, fragments, with scattered short core lengths
Intensely Fractured	0.1 foot to 0.3 foot with scattered fragments
Moderately Fractured	0.3 foot to 1 foot
Slightly Fractured	1 foot to 3 feet
Very Slightly Fractured	Greater than 3 feet
Unfractured	No fractures observed

HEALING	DESCRIPTION
Not Healed	Discontinued surface, fractured zone, sheared material, filling is not cemented
Partly Healed	Less than 50% of fractures or sheared zone bonding
Moderately Healed	Greater than 50% fractures or sheared zone bonding
Totally Healed	All fragments are bonded

QUALITY	RQD (%)
Very poor	Less than 25%
Poor	25 to 50%
Fair	51 to 75%
Good	76 to 90%

Rock Quality Designation (RQD) is a measure of quality of rock core taken from a borehole. The length of core pieces is measured along center line of the pieces. All pieces of intact rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run to obtain RQD value



PROJECT Mills C	r NAME Camas					CLIENT HSR Development		PROJECT HSR	т no. - <b>1-01-</b> 1		TEST PIT	NO.
	T LOCATION IS, Washin	aton				contractor L&S Contractors Inc.	EQUIPMENT Excavator	TECHNIC			DATE 10-13-	-2023
TEST PIT	LOCATION igure 2	gion				GROUNDWATER DEPTH Groundwater not observed		START 1 0755			FINISH TI 0820	
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRII	PTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0				GW		Grey GRAVEL and cobbles, moist, dense, angular aggregate (0.25 to 10 inches). Shoulder of haul road built for site access.			-			
-	TP1.1			GW-GN		Brown GRAVEL with silt, moist, medium dense to dense, fine angular aggregate (0.1 to 0.25 inch).  Bottom of test pit at 4 feet due to practical refusal on bedrock.						
- 5												
- 10												
15												



Mills Cames PROJECT GRANDIN Cames, Washington Cames, Washington Cames, Washington Cames, Washington See Figure 2  See Figure 3  See Figure 4  See Figure 4  See Figure 5  See Figure 6  See Figure 6  See Figure 7  See Figure 7  See Figure 8  See Figure 8  See Figure 9	PROJECT	T NAME					CLIENT		PROJEC	T NO.	1	TEST PIT	NO.
Camas, Washington  L&S. Contractors inc.  See Figure 2  See Survey 10-13-2023  Report Figure 2  See Survey 10-13-2023  Report Field In Discription  See Figure 2  See Survey 10-12-12-12-12-12-12-12-12-12-12-12-12-12-							HSR Development	EQUIPMENT	TECHNIC		ı	DATE	
See Figure 2  Deptit Sample (rect) Subject of Field (r			gton				L&S Contractors Inc.		SSC			10-13	-2023
Depth Field (Reef) Sall Survey (Reef) Soll Survey (Reef) (Reef) (Reef) (Reef) (Reef) (Reef) Soll Survey (Reef) (Reef							l .			IME			IME
2 to 3 inches forest duff (topsoil). Fine tree roots extend to 8 inches in depth. Fine tree roots extend to 8 i	366 1	igure z					Groundwater not obse	rvea.	0622	Φ		0033	
2 to 3 inches forest duff (topsoil). Fine tree roots extend to 8 inches in depth. Fine tree roots extend to 8 i	Depth (feet)	Field	Soil Survey	Soil	Soil	Graphic Log	LITHOLOGIC DESCRI	PTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	
GW-GN Brown GRAVEL with silt, moist, dense, fine angular aggregate (0.1 to 0.25 inch).  GC Brown GRAVEL with silt, moist, dense, fine angular aggregate (0.1 to 0.25 inch).  Decomposed/weathered bedrock, very dense clayey gravel.  Bottom of test pit at 4 feet due to practical refusal on bedrock.	0						2 to 3 inches forest duff	(topsoil).					
angular aggregate (0.1 to 0.25 inch).  Decomposed/weathered bedrock, very dense clayey gravel. Bottom of test pit at 4 feet due to practical refusal on bedrock.  1 10						-4							
Clayey gravel.  Bottom of test pit at 4 feet due to practical refusal on bedrock.								t, moist, dense, fine to 0.25 inch).					
Clayey gravel.  Bottom of test pit at 4 feet due to practical refusal on bedrock.	-												
Boltom of test pit at 4 feet due to practical refusal on bedrock.	-				GC		Decomposed/weathered	d bedrock, very dense					
							Bottom of test pit at 4 fe	et due to practical					
	- 5												
	-												
	-												
	-												
- 15 - 15	- 10												
- 15 - 15	-												
- 15 - 15 - 15	_												
- 15 - 15	-												
- 15 - -													
	- 15												
	-												



PROJEC	T NAME					CLIENT		PROJEC	T NO.		TEST PIT	NO.
Mills C	T LOCATION					HSR Development	EQUIPMENT	TECHNIC	-1-01-1		TP-3	
	as, Washin	gton				L&S Contractors Inc.	Excavator	SSC			10-13-	2023
TEST PIT	LOCATION					GROUNDWATER DEPTH		START 1	IME		FINISH TI 0917	ME
See F	igure 2			I	I	Groundwater not obser	rved.	0909	0		0917	
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIF	PTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0					n, <u>w</u>	6 to 8 inches forest duff	with fine tree roots.					
-				GC		Grey weathered bedrock dense clayey gravel.  Difficult digging from 1 to						
- 5 - -					6/0/0	Bottom of test pit at 5 fee refusal on bedrock.	et due to practical					
- 10												
<u> </u>												
- 15												
<u> </u>												



PROJECT	T NAME					CLIENT		PROJEC	T NO.		TEST PIT	NO.
	T LOCATION					HSR Development CONTRACTOR	EQUIPMENT	TECHNIC	1-01-1	l	DATE	
	ıs, Washin	gton				L&S Contractors Inc.	Excavator	SSC			10-13	
	location igure 2					GROUNDWATER DEPTH Groundwater not obse	rved.	START T 0918	IME		FINISH TI 0935	ME
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS			Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0					<u> </u>	6 to 8 inches forest duff	with fine tree roots.		2			
- 5	TP4.1		A-7-6(4)	ML		Difficult digging from 8 to weathered bedrock gravieet.  Brown SILT with trace of moist, medium stiff to standard	pravel (0.1 to 0.25 inch), iff. o 10 feet. Intermixed yel and boulders at 8	41	45	41	15	
- 10 - -						Bottom of test pit at 10 f refusal on bedrock.	eet due to practical					
- - 15 -												



PROJECT	NAME					CLIENT		PROJECT	NO.		TEST PIT	NO.
Mills C	T LOCATION					HSR Development	EQUIPMENT	HSR-			TP-5	
	s, Washir	ngton				L&S Contractors Inc.	Excavator	SSC			10-13	2023
	LOCATION					GROUNDWATER DEPTH		START TI	ME		FINISH TI	ME
See F	igure 2	_				Groundwater not obse	rved.	0950			1005	
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRI	PTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0					#	8 to 10 inches forest duff with tree roots.  Grey weathered bedrock, moist, dense to very dense clayey gravel.  Fine tree roots extend to 2 feet in depth.  Difficult digging from 2 to 4 feet.  Bottom of test pit at 4 feet due to practical refusal on bedrock.						
-				GC								
- 5												
- 10												
- 15												



PROJECT Mills C	NAME amas					CLIENT HSR Development		PROJECT HSR	T NO. - <b>1-01-</b>	1	TEST PIT	NO.
	T LOCATION	aton				CONTRACTOR L&S Contractors Inc.	EQUIPMENT Excavator	TECHNIC	CIAN		DATE 10-13	2022
	s, Washir	igion				GROUNDWATER DEPTH	Excavator	START 1	IME		FINISH T	
	igure 2					Groundwater not obse	erved.	1007	IIVIL		1015	IIVIL
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log		PTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0					M. 301 301.	8 to 10 inches forest du	ff with tree roots.					
					- 4 - 4 - 4 - 4 - 4 - 4 - 4 - 4 - 4 - 4	Fine tree roots extend to	o 1.5 feet in depth.					
-				GC		Grey weathered bedroc dense clavey gravel.	k, moist, dense to very					
-						dense clayey gravel.  Difficult digging from 1.5 to 4 feet.						
- 5					<u> </u>	Bottom of test pit at 4 fe refusal on bedrock.	eet due to practical					
- 10												
- 15												



PROJECT	Γ NAME					CLIENT		PROJEC	T NO.		TEST PIT	NO.
Mills C	T LOCATION					HSR Development CONTRACTOR	EQUIPMENT	TECHNIC	-1-01-1		TP-7	
	ıs, Washin	gton				L&S Contractors Inc.	Excavator	SSC			10-13-	2023
TEST PIT	LOCATION					GROUNDWATER DEPTH		START 1	IME		FINISH TI	ME
See F	igure 2					Groundwater not obser	rved.	1020			1033	
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRII	PTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0					w	8 to 10 inches forest dut	f with tree roots.					
						Fine tree roots extend to	1.5 feet in depth.					
-				GC		Grey weathered bedrood dense clayey gravel.  Difficult digging from 3 to	k, moist, dense to very					
- 5 - -						Bottom of test pit at 5 fer refusal on bedrock.	et due to practical					
- 10												
-												
- 15												





PROJECT Mills C	T NAME					CLIENT HSR Development		PROJEC HSR:	T NO. - <b>1-01-</b>	1	TEST PIT	NO.
PROJECT	T LOCATION IS, Washir	naton				CONTRACTOR L&S Contractors Inc.	EQUIPMENT Excavator	TECHNIC		•	DATE 10-13	-2023
	LOCATION	igion				GROUNDWATER DEPTH	Executor	START T	IME		FINISH T	
See F	igure 2					Groundwater not obser	rved.	1040			1045	
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIF	PTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0					<u></u>	8 to 10 inches forest duf	ff with tree roots.					
					4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4	Fine tree roots extend to	1.5 feet in depth.					
_				GC		Grey weathered bedrock dense clayey gravel.						
-						Bottom of test pit at 2.5 f refusal on bedrock.	feet due to practical					
- 5 - -												
-												
- 10 -												
-												
- 15 - -												



PROJECT NAME Mills Camas			CLIENT HSR Development		PROJEC	T NO. - <b>1-01-</b> 1	<u> </u>	TEST PIT	NO.
PROJECT LOCATION			CONTRACTOR	EQUIPMENT	TECHNIC			DATE	
Camas, Washington			L&S Contractors Inc.	Excavator	SSC			10-13-	
test pit location See Figure 2			GROUNDWATER DEPTH Groundwater not obse	rved	START T	IME		FINISH TI 1108	ME
			Groundwater not obser	ivea.		e			
(feet)   Field   Soil Survey   S	SHTO USCS Soil Soil Type	Graphic Log	LITHOLOGIC DESCRI	PTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0		_ <del>*</del>	8 to 10 inches forest du	ff with tree roots.					
		- 4 - 4 - 4 - 4 - 4 - 4	Fine tree roots extend to	1.5 feet in depth.					
- TP9.1 A-	-4(1)		Brownish red silty SANI inch), moist, dense to vermedium plasticity.	O with gravel (0.1 to 0.25 ery dense, low to	20	44	39	8	
-			Bottom of test pit at 3 fer refusal on bedrock.	et due to practical					
- 5									
-									
-									
-									
- 10									
- 10									
_									
_									
-									
-									
- 15									
-									
-									



						ILSI FII LUU					·
PROJECT Mills C	Camas					CLIENT HSR Development		-1-01-1	1	TEST PIT	NO.
	t location is, Washir	ngton				CONTRACTOR EQUIPMENT EXCAVATOR	SSC			DATE 10-13	
TEST PIT See F	igure 2					GROUNDWATER DEPTH Groundwater not observed.	1130	ΓIME		FINISH TI 1146	ME
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
- 5 - 10 - 15	ID	Description	Type	GC	Log	8 to 10 inches forest duff with tree roots.  Fine tree roots extend to 1.5 feet in depth.  Grey-brown weathered bedrock, moist, dense to very dense clayey gravel.  Difficult digging from 1.5 to 3.5 feet.  Bottom of test pit at 3.5 feet due to practical refusal on bedrock.		No. 2		Pla I	resuling
-											



PROJECT Mills C	NAME Samas					CLIENT HSR Development		PROJEC HSR	T NO. - <b>1-01-</b>	<u> </u>	TEST PIT	NO.
PROJEC	T LOCATION					CONTRACTOR	EQUIPMENT	TECHNIC			DATE	
	s, Washin	igton				L&S Contractors Inc.	Excavator	SSC			10-13-	
TEST PIT	location igure 2					GROUNDWATER DEPTH	nuod	START 1 1150	IME		FINISH TI 1155	ME
See r	igure z			<u> </u>		Groundwater not obse	rvea.	1150	۵		1133	
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log			Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0					# # #	8 inches forest duff with	tree roots.		_			
				GC	70707	Grey weathered bedroc	k, moist, dense to very					
-						dense clayey gravel.						
-						Bottom of test pit at 1.5 feet due to practical refusal on bedrock.						
-												
- 5												
-												
-												
-												
- 10												
-												
-												
_												
- 15												
_												
-												



PROJECT Mills C	amas					CLIENT HSR Development			1-01-1		TEST PIT NO. TP-12			
	r location s, Washin	igton				CONTRACTOR L&S Contractors Inc.  GROUNDWATER DEPTH  EQUIPMENT Excavator			TECHNICIAN SSC START TIME			DATE 10-13-2023		
TEST PIT	LOCATION	<u> </u>										ME		
See F	igure 2				1	Groundwater not observed.		1210			1230			
Depth (feet)	Sample Field ID	ld   Soil Survey   Soil   Soil   Graphic   LITHOLOGIC DESCRIPTION AND REMARKS						Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing		
0					Mr	2 to 3 inches root zone underla topsoil.	ain by 6 inches							
- - - 5	TP12.1			ML		Brown SILT with gravel (0.1 to medium stiff, low plasticity.  Increase in gravel and weather feet.								
-	TP12.2					Difficult digging from 6 feet to 9  Weathered bedrock boulders a  Bottom of test pit at 9.5 feet du	ıt 9 feet.							
- 10 - - -						refusal on bedrock.	e to practical							
- 15 - -														





PROJECT	T NAME					CLIENT HSR Development		PROJEC	T NO.	1	TEST PIT	NO.		
	T LOCATION					CONTRACTOR EQUIPMENT			HSR-1-01-1 TECHNICIAN			DATE		
Cama	ıs, Washin	gton				L&S Contractors Inc.	SSC			10-13-2023				
TEST PIT	LOCATION					GROUNDWATER DEPTH	d	START 1	IME		FINISH TI 1300	ME		
See r	igure 2					Groundwater not obse	rvea.	1245		1300				
Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRI	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing			
0					# # # 3	8 to 10 inches forest du	ff with tree roots.							
					W _ 4	Tree roots extend to 1.5	feet in depth.							
-				ML			with weathered bedrock,							
-						moist, stiff, low plasticity	y.							
-	TP13.1	TP13.1 GC				Grey weathered bedroc dense clayey gravel. Difficult digging from 2.5								
- - 5						Bottom of test pit at 4 fe refusal on bedrock.	et due to practical							
-														
-														
-														
- 10														
-														
-														
-														
_														
15														
- 15														
-														
-														

# APPENDIX B LABORATORY TESTING

#### **CLASSIFICATION**

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration log if those classifications differed from the field classifications.

#### **ATTERBERG LIMITS**

Atterberg limits (plastic and liquid limits) testing was performed on select soil samples in general accordance with ASTM D4318. The plastic limit is defined as the moisture content where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented in this appendix.

#### **MOISTURE CONTENT**

We determined the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

### **PARTICLE-SIZE ANALYSIS**

We completed particle-size analysis on select soil samples in general accordance with ASTM D6913. This test is a quantitative determination of the soil particle size distribution expressed as a percentage of dry soil weight.





## PARTICLE-SIZE ANALYSIS REPORT

	CLIENT			NO		LADID		
PROJECT Mills Camas	HSR Development				1		22 125	6
	Leadbetter Road 500 E. Broadway, Suite 120							
	•		REPORT DATE FIELD ID TP4					
Camas, Washington 98607	Vancouver, Washington 98660			10/30/23 TP4.2  DATE SAMPLED 10/13/23 SAMPLED BY 10/13/23 SSC   JOSCS SOIL TYPE SM, Silty Sand with Gravel  ASHTO CLASSIFICATION A-7-6(4)  ASTM D6913, Method A  SIEVE DATA  % gravel = 22.0% % sand = 33.1% % silt and clay = 45.0%  PERCENT PASSIN SIEVE SPI act. interp. max  6.00" 150.0 100% 4.00" 100.0 100% 3.00" 75.0 100% 2.50" 63.0 100% 2.50" 63.0 100% 1.75" 45.0 99% 1.25" 31.5 94% 1.25" 31.5 94% 3/4" 19.0 89% 5/8" 16.0 88% 1/2" 12.5 86% 3/8" 9.50 84% 1/4" 6.30 81% 44 4.75 78%  #8 2.36 76% #10 2.00 76% #110 2.00 76% #16 1.18 72% #8 2.36 76% #10 2.00 76% #110 0.0150 52% #110 0.150 52% #110 0.150 52% #110 0.106 48%				
					3	O) WIII EEI		
MATERIAL DATA	-			, ==, ==				
MATERIAL SAMPLED	MATERIAL SOURCE							
brown Silty SAND with Gravel	Test Pit TP-04		SM,	Silty Sa	and w	ith Grav	vel	
	depth = 8 feet							
PECIFICATIONS					ATION			
none			Α-/-	J( <del>4</del> )				
ABORATORY TEST DATA								
ABORATORY EQUIPMENT								
Rainhart "Mary Ann" Sifter, air-dried prep	o, hand washed, composite sieve - #4 split				13, M	ethod A	<b>\</b>	
ADDITIONAL DATA			SIEVE D	ATA				
initial dry mass (g) = 1842.3	and Windowski and A					-		
as-received moisture content = 41%	coefficient of curvature, $C_C = n/a$			0.4				
liquid limit = 41	coefficient of uniformity, $C_U = n/a$			%	siit an	a ciay =	45.0%	
plastic limit = 26 plasticity index = 15	effective size, $D_{(10)} = n/a$ $D_{(30)} = n/a$				ı	DEDCEN.	T DACCINI	10
fineness modulus = n/a	$D_{(30)} = n/a$ $D_{(60)} = 0.317 \text{ m}$	m	SIEV	E SIZE				
NOTE: Entire sample used for analysis; did	. ,			1				m
2 cample acce to analysis, and						100%		
GRAIN SIZ	E DISTRIBUTION		4.00"	100.0		100%		
72.2.3.4.1.7.4.7.4.7.7.7.7.7.7.7.7.7.7.7.7.7.7	# # # # # # # # # # # # # # # # # # #							
4 ある 秋 デニ 年 ある こ あ こ 業 業業 100% O-O-O-O-t-+ ++++ + + + + + + + + + - + + + + + + - + + + + + + + +	# # # # ## ###	<u>+</u> 100%			1000/	100%		
		10070			100%	99%		
90%		90%	1 50"		97%			
90%		90 /8	1.25" 1.00"	31.5		94%		
		000/	1.00		91%			
80%		80%	1/8		900/	90%		
	<u>a</u>	<u> </u>			0976	88%		
70%		70%						
		]	3/8"	9.50	84%			
ත <sup>60%</sup>		60%				81%		
	Pag	1			78%	700/		
50%		50%			76%	76%		
1%		1			1070	72%		
40%		40%			70%	,.		
-		-	#30	0.600		66%		
30%		30%	₽ #40		63%			
		]	1.75		F70/	59%		
20%		20%	#00		5/%	5/10/		
		1			52%	J <del>+1</del> 70		
10%		10%			<del>-</del> - / · ·	48%		
		1070	#170			47%		
00/		00/	#200		45%			
100.00	1.00 0.10	── <del>-</del> 0% 0.01	DATE TES			TESTED		
	cle size (mm)		1	0/26/23	3		MRS	
parti	cie size (ilili)							
parti	cie size (iiiii)			1	1			_

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## ATTERBERG LIMITS REPORT

												· 	1
PROJECT Mills	Camas				ľ	CLIENT HSR Development						PROJECT NO. HSR-1-01-1	S23-1356
	SE Leadbett	or Roa	d						120			REPORT DATE	FIELD ID
						500 E. Broadway, Suite 120						10/30/23	TP4.2
Cama	as, Washing	607			Vancouver, Washington 98660						DATE SAMPLED	SAMPLED BY	
												10/13/23	SSC
MATER	IAL DATA				•							•	
	SAMPLED		<i>a</i> 1			MATERIAL SC						USCS SOIL TYPE	
brow	n Silty SAN		Test Pi						SM, Silty Sand	with Gravel			
						depth =	8 feet						
	ATORY TES		4										
	ORY EQUIPMENT d Limit Ma		Hand Da	llad								TEST PROCEDURE  ASTM D4318	
	ERG LIMITS	ciiiie,		MIT DETER	MINI A TI	ON						ASTW1 D4316	
AIIERD	ERG LIWITS		LIQUID LI	WIII DEIER	KIWIINAII	ON O	9		€	4		LIQ	JID LIMIT
li	quid limit =	41	wet soil	+ pan weig	ht. a =	32.63	33.85		33.77	33.71		100%	
	astic limit =	26		+ pan weig	_	29.16	29.98		29.91	29.83		90% -	
	city index =	15		pan weig		20.42	20.56		20.67	20.77		<b>8</b> 70% €	
					ows) =	33	26		22	16		<b>હ</b> 60% <del>[</del>	
				moistur	e, % =	39.7 %	41.1 %	, 4	1.8 %	42.8 %		50% + 6 40% + 6 40%	90-0
SHRINK	AGE		PLASTIC	LIMIT DETI	ERMINA	TION						0070 E	
						0	2		6	4		20%	
shrink	kage limit =	n/a	wet soil	+ pan weig	ht, g =	27.58	27.20					0%	
shrink	age ratio =	n/a	dry soil + pan weight, g =			26.23	25.81					10	25 100
				pan weig	ht, g =							number	of blows, "N"
				moistur	e, % =	25.5 %	25.8 %	)					
							_					ADDITIONAL DATA	
				PLA	STICIT	TY CHAR	RT					0/ 27212	22.00/
8	30 F								/		% gravel % sand		
		-								الممري			
7	70 ∔					/					% silt and clay		
	-											% silt	
6	,							مر	<b>′</b>   "U	" Line		% clay	
0	60							,,,,,				moisture content	= 41%
							مممد						
<b>×</b> <sup>5</sup>	50 + +						.000		+	$\uparrow$			
ing	-					مر			, "	A' Line			
<u>.</u> € 4	10						CH or	OH /					
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plasticity index	, [				Johnson								
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				۵ مممی	or OL								
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	-		- Landar		0		MH or	ЭН					
1	10		, por								_		
		CL	-ML	MI	or OL							DATE TEATED	TEOTER 21
	0		<b>_</b>				<b></b>		<b>_</b>		_	DATE TESTED	TESTED BY MRS
	0 10	)	20 3	30 4			60 7	)	80	90	100	10/27/23	MIKS
					liq	uid limit						dans 1	

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## **PARTICLE-SIZE ANALYSIS REPORT**

PROJECT	TOLLINIT	-	LADID
Mills Camas	CLIENT HSR Development	PROJECT NO. HSR-1-01-1	LAB ID S23-1357
313 SE Leadbetter Road	500 E. Broadway, Suite 120	REPORT DATE	FIELD ID
	• • • • • • • • • • • • • • • • • • •	10/30/23	TP9.1
Camas, Washington 98607	Vancouver, Washington 98660	DATE SAMPLED	SAMPLED BY
		10/13/23	SSC
	1	10/13/23	330
MATERIAL DATA			
MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE	4:41- C1
brownish red Silty SAND with Gravel	Test Pit TP-09	SWI, SHILY SAI	nd with Gravel
	depth = 2 feet		
SPECIFICATIONS		AASHTO CLASSIFICAT	TION
none		A-4(1)	
LABORATORY TEST DATA			
LABORATORY EQUIPMENT		TEST PROCEDURE	
Rainhart "Mary Ann" Sifter, air-dried prep,	hand washed composite sieve - #4 split	ASTM D691	3 Method A
ADDITIONAL DATA	nana washea, composite sieve "1 spire	SIEVE DATA	5, Wedfod 71
initial dry mass (g) = 1551.4		SIEVE DATA	% gravel = 15.6%
as-received moisture content = 20%	coefficient of curvature, $C_C = n/a$		% sand = 40.1%
		0/ -	% sand = $40.1\%$ ilt and clay = $44.4\%$
liquid limit = 39	, ,	% S	iii aliu Clay = 44.4%
plastic limit = 31	effective size, $D_{(10)} = n/a$	1	DEDOENT DAGGING
plasticity index = 8	$D_{(30)} = n/a$	CIE//E CIZE	PERCENT PASSING SIEVE   SPECS
fineness modulus = n/a	$D_{(60)} = 0.573 \text{ mm}$	SIEVE SIZE	
		US mm	'
CDAIN SIZE	DISTRIBUTION	6.00" 150.0 4.00" 100.0	100% 100%
GRAIN SIZE	DISTRIBUTION	3.00" 75.0	100%
7.7.7.7.7.7.7.7.7.7.7.7.7.7.7.7.7.7.7.	# # # # # # # # # # # # # # # # # # #	2.50" 63.0	100%
100% 0-00-000-000-0-0-1	+,+,+,+,+,+++++++++++++++++++++++++++++	2.00" 50.0	100%
		1.75" 45.0	100%
90%	90%	1.50" 37.5	100%
50%		1.25" 31.5	100%
		1.25" 31.5 1.00" 25.0 7/8" 22.4	100%
80%	80%	110 22.4	100%
			100%
70%	70%	5/8" 16.0 1/2" 12.5	99% 99%
		3/8" 9.50	98%
60% +	60%	1/4" 6.30	90%
Bu iiiiiiiiiiiiiiiiiiiiiiiiiiiiiiiiiiii		#4 4.75	84%
50%	50%	#8 2.36	73%
ŭ - ~ ·	111111111111111111111111111111111111111	#10 2.00	70%
8		#16 1.18	65%
40%	40%	#20 0.850	62%
		#30 0.600	60%
30%	30%	<b>9</b> #40 0.425	58%
		#40 0.425 #50 0.300 #60 0.250	56%
20%	20%	#60 0.250 #80 0.180	55% 52%
<u> </u>		#100 0.150	50%
10%	10%	#140 0.106	47%
1070		#170 0.090	46%
		#200 0.075	44%
100.00 10.00	1.00 0.10 0.01	DATE TESTED	TESTED BY
		10/26/23	MRS
partici	e size (mm)		
◆ sieve sizes		A	1 Conto
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## ATTERBERG LIMITS REPORT

							L			LFOR		T
PROJECT Mills	Camas				CLIENT HSR De	velopmen	t			PROJECT NO.	LAB ID	
	E Leadbette	m Doo	A				_		120		HSR-1-01-1 REPORT DATE	S23-1357 FIELD ID
						Broadway,			10/30/23	TP9.1		
Cama	as, Washingt	607			Vancou	ver, Wash	ington	98660		DATE SAMPLED	SAMPLED BY	
										10/13/23	SSC	
MATERI	IAL DATA											
	SAMPLED nish red Silt	ID with (	Fravel		MATERIAL SOI Test Pit					USCS SOIL TYPE SM, Silty Sand	with Gravel	
DIOWI	ilisii ica siii,	y SAI	VD WILLI	Jiavei		depth =					Sivi, Sifty Sand	with Graver
	ATODY TEST	- DAT	^		I							
	ATORY TEST DRY EQUIPMENT	DAI	4								TEST PROCEDURE	_
	d Limit Mac	hine,	Hand Ro	lled							ASTM D4318	
	ERG LIMITS			MIT DETER	RMINAT	ION					1.16	NUD LIMIT
					_	0	2		6	4		QUID LIMIT
	quid limit =	39	wet soil	+ pan weig	ht, g =	35.74	32.23		4.46	32.25	100% <del>-</del>	
	astic limit =	31	dry soil	+ pan weig	ht, g =	31.59	29.05		0.61	28.96	80%	
plastic	city index =	8		pan weig		20.66	20.76		0.76	20.82	<b>8</b> 70% €	
				,	= (awc	35	28		21	17	<b>2</b> 60% + 50% + 50% + 50%	
				moisture	e, % =	38.0 %	38.4 %	39	9.1 %	40.4 %	— is 40% ← — ←	0-0-0
SHRINK	AGE		PLASTIC	LIMIT DETE	ERMINA						0070	
					_	0	9		€	4	20% -	
	kage limit =	n/a	wet soil + pan weight, g =			27.76	27.39				0%	
shrink	age ratio =	n/a	dry soil + pan weight, g = pan weight, g = moisture, % =			26.10	25.87				10	25 100
						20.78	20.94				numb	er of blows, "N"
			ļ	moisture	e, % =	31.2 %	30.8 %				ADDITIONAL DATA	
				PLAS	STICI	TY CHAR	Т				ADDITIONAL DATA	
8	30										% grave	l = 15.6%
·										poor	% san	d = 40.1%
	-								_		% silt and cla	
7	70 +							·	2000		% sil	
	-								"1.1"	Line	% cla	
6	60							,,,,,,,		LIIIC		
	-							•			moisture conten	t = 20%
	-						2000					
<b>×</b> 5	50 +					1	2000					
plasticity index	-					بممر	1		"/	A' Line		
<b>.</b> ₹ 4	10			ļ		<del></del>	CH or O	Н/				
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2	20 +		/	CI CI	or OL	4				-		
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		Cl	_ML		or OL						DATE TESTED	TESTED BY
	0 10		20 3	BO 40	0	50 (	60 70	,	<del>                                     </del>	90 100	10/27/23	MRS
						uid limit	. •	`			1 1	Coto
					•						Jan	

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# APPENDIX C REPORT LIMITATIONS AND IMPORTANT INFORMATION





# Geotechnical and Environmental Report Limitations and Important Information Report Purpose, Use, and Standard of Care

This report has been prepared in accordance with standard fundamental principles and practices of geotechnical engineering and/or environmental consulting, and in a manner consistent with the level of care and skill typical of currently practicing local engineers and consultants. This report has been prepared to meet the specific needs of specific individuals for the indicated site. It may not be adequate for use by other consultants, contractors, or engineers, or if change in project ownership has occurred. It should not be used for any other reason than its stated purpose without prior consultation with Columbia West Engineering, Inc. (Columbia West). It is a unique report and not applicable for any other site or project. If site conditions are altered, or if modifications to the project description or proposed plans are made after the date of this report, it may not be valid. Columbia West cannot accept responsibility for use of this report by other individuals for unauthorized purposes, or if problems occur resulting from changes in site conditions for which Columbia West was not aware or informed.

### **Report Conclusions and Preliminary Nature**

This geotechnical or environmental report should be considered preliminary and summary in nature. The recommendations contained herein have been established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. The exploration and associated laboratory analysis of collected representative samples identifies soil conditions at specific discreet locations. It is assumed that these conditions are indicative of actual conditions throughout the subject property. However, soil conditions may differ between tested locations at different seasonal times of the year, either by natural causes or human activity. Distinction between soil types may be more abrupt or gradual than indicated on the soil logs. This report is not intended to stand alone without understanding of concomitant instructions, correspondence, communication, or potential supplemental reports that may have been provided to the client.

Because this report is based upon observations obtained at the time of exploration, its adequacy may be compromised with time. This is particularly relevant in the case of natural disasters, earthquakes, floods, or other significant events. Report conclusions or interpretations may also be subject to revision if significant development or other manmade impacts occur within or in proximity to the subject property. Groundwater conditions, if presented in this report, reflect observed conditions at the time of investigation. These conditions may change annually, seasonally or as a result of adjacent development.

### Additional Investigation and Construction QA/QC

Columbia West should be consulted prior to construction to assess whether additional investigation above and beyond that presented in this report is necessary. Even slight variations in soil or site conditions may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent QA/QC construction observation and testing to verify soil conditions do not differ materially or significantly from the interpreted conditions utilized for preparation of this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Actual subsurface conditions are more readily observed and discerned during the earthwork phase of construction when soils are exposed. Columbia West cannot accept responsibility for deviations from recommendations described in this



report or future performance of structural facilities if another consultant is retained during the construction phase or Columbia West is not engaged to provide construction observation to the full extent recommended.

### **Collected Samples**

Uncontaminated samples of soil or rock collected in connection with this report will be retained for thirty days. Retention of such samples beyond thirty days will occur only at client's request and in return for payment of storage charges incurred. All contaminated or environmentally impacted materials or samples are the sole property of the client. Client maintains responsibility for proper disposal.

### **Report Contents**

This geotechnical or environmental report should not be copied or duplicated unless in full, and even then only under prior written consent by Columbia West, as indicated in further detail in the following text section entitled *Report Ownership*. The recommendations, interpretations, and suggestions presented in this report are only understandable in context of reference to the whole report. Under no circumstances should the soil boring or test pit excavation logs, monitor well logs, or laboratory analytical reports be separated from the remainder of the report. The logs or reports should not be redrawn or summarized by other entities for inclusion in architectural or civil drawings, or other relevant applications.

### **Report Limitations for Contractors**

Geotechnical or environmental reports, unless otherwise specifically noted, are not prepared for the purpose of developing cost estimates or bids by contractors. The extent of exploration or investigation conducted as part of this report is usually less than that necessary for contractor's needs. Contractors should be advised of these report limitations, particularly as they relate to development of cost estimates. Contractors may gain valuable information from this report, but should rely upon their own interpretations as to how subsurface conditions may affect cost, feasibility, accessibility and other components of the project work. If believed necessary or relevant, contractors should conduct additional exploratory investigation to obtain satisfactory data for the purposes of developing adequate cost estimates. Clients or developers cannot insulate themselves from attendant liability by disclaiming accuracy for subsurface ground conditions without advising contractors appropriately and providing the best information possible to limit potential for cost overruns, construction problems, or misunderstandings.

### **Report Ownership**

Columbia West retains the ownership and copyright property rights to this entire report and its contents, which may include, but may not be limited to, figures, text, logs, electronic media, drawings, laboratory reports, and appendices. This report was prepared solely for the client, and other relevant approved users or parties, and its distribution must be contingent upon prior express written consent by Columbia West. Furthermore, client or approved users may not use, lend, sell, copy, or distribute this document without express written consent by Columbia West. Client does not own nor have rights to electronic media files that constitute this report, and under no circumstances should said electronic files be distributed or copied. Electronic media is susceptible to unauthorized manipulation or modification, and may not be reliable.



### **Consultant Responsibility**

Geotechnical and environmental engineering and consulting is much less exact than other scientific or engineering disciplines, and relies heavily upon experience, judgment, interpretation, and opinion often based upon media (soils) that are variable, anisotropic, and non-homogenous. This often results in unrealistic expectations, unwarranted claims, and uninformed disputes against a geotechnical or environmental consultant. To reduce potential for these problems and assist relevant parties in better understanding of risk, liability, and responsibility, geotechnical and environmental reports often provide definitive statements or clauses defining and outlining consultant responsibility. The client is encouraged to read these statements carefully and request additional information from Columbia West if necessary.



