4. Geotechnical Soil Analysis Report

Exhibit 5 SUB25-1003

Report of Geotechnical Engineering Services

Camas Woods Phase 3

Camas, Washington

February 18, 2025

Geotechnical ■ Environmental ■ Special Inspections

Columbia West Engineering, Inc

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February 18, 2025

HSR Capital LLC
500 East Broadway, Suite 120
Vancouver, WA 98660

Attn: Kevin Miller

Re: Report of Geotechnical Engineering Services
Camas Woods Phase 3
26514 and 26416 SE 8th Street
Camas, Washington
CWE Project: HSR-4-01-1

Columbia West Engineering, Inc. (Columbia West) is pleased to present this geotechnical engineering report for the Camas Woods Phase 3 project located at 26514 and 26416 SE 8th Street (parcel numbers 178209000 and 178109000) in Camas, Washington. Our services were conducted in accordance with our proposal dated September 6, 2024.

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

Sincerely,

Michael A. Chacon, PE
Senior Staff Engineer

Daniel E. Lehto, PE, GE
Principal Engineer

Signed 02/18/2025

Expires 06/05/2025

cc: Bryce Hanson, AKS Engineering & Forestry

MAC:ASR:DEL:kat
Attachments
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EXECUTIVE SUMMARY

This section provides a summary of the geotechnical considerations associated with the Camas Woods Phase 3 project in Camas, Washington. Our conclusions and recommendations are based on the subsurface information presented in the report and proposed development information provided by the design team. A detailed discussion of the geotechnical considerations summarized here is presented in respective sections of the report.

- The proposed lightly loaded residential structures can be supported by conventional spread footings bearing on firm soil as described in the report.

- The near-surface native soil is sensitive to disturbance when at a moisture content that is above optimum. As discussed in the report, the subgrade should be protected from disturbance and damage by construction traffic.

- Cobbles and boulders were encountered in the explorations at the site. Cobbles and boulders will result in difficult excavation and trenches may be wider than anticipated, increasing the amount of backfill material required.

- Moisture conditioning will likely be required to use the on-site soil as structural fill. Accordingly, extended dry weather will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact the on-site soil during the rainy season or during prolonged periods of rainfall.

- Groundwater was encountered at 12 feet BGS in test pit TP-6 during our subsurface exploration on December 31, 2024. Dewatering may be required for deeper utilities, particularly in areas of cut and in the wet season.

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ABBREVIATIONS AND ACRONYMS

AC	asphalt concrete
AOS	apparent opening size
ASCE	American Society of Civil Engineers
ASTM	ASTM International
BGS	below ground surface
CSZ	Cascadia subduction zone
g	gravitational acceleration (32.2 feet/second^2)
GIS	geographic information system
HMA	hot mix asphalt
H:V	horizontal to vertical
IBC	International Building Code
in/hr	inch(es) per hour
km	kilometers
MCE	maximum considered earthquake
M_W	moment magnitude
NAVD 88	North American Vertical Datum of 1988
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
psf	pounds per square foot
psi	pounds per square inch
PVC	polyvinyl chloride
QA/QC	quality assurance/quality control
USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
WSS	Washington Standard Specifications for Road, Bridge, and Municipal Construction (2024)

REPORT OF GEOTECHNICAL ENGINEERING SERVICES
CAMAS WOODS PHASE 3
CAMAS, WASHINGTON

1.0 INTRODUCTION

Columbia West is pleased to submit this geotechnical engineering report for the Camas Woods Phase 3 project in Camas, Washington. The approximately 8.82-acre site is comprised of parcel numbers 178209000 and 178109000 and is located at 26514 and 26416 SE 8th Street in Camas, Washington. The site is shown relative to surrounding physical features on Figure 1. Figure 2 shows the existing conditions at the site and our exploration locations. Abbreviations and acronyms used herein are defined immediately following the Table of Contents.

Development plans include construction of a single-family residential subdivision with associated infrastructure. Infrastructure specifics and grading plans were not available for review at the time this report was prepared. Foundation loads were also unknown at the time this report was prepared; however, we estimate maximum column and wall loads will be less than 30 kips and 4 kips per lineal foot, respectively.

2.0 BACKGROUND

Based on historical aerial photographs, the site has been an undeveloped property since at least the 1950s, with single-family residences constructed in the 1970s. The site is bounded by a church to the west; single-family rural development to the north and south; and vacant, forested land to the east.

3.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. Specifically, we completed the following tasks:

- Reviewed information available in Columbia West's files from previous geological and geotechnical studies conducted in the site vicinity.
- Coordinated and managed the field exploration program, which included locating public utilities, coordinating site access, and scheduling subcontractors and Columbia West field staff.
- Explored subsurface conditions at the site by excavating six test pits to depths between 12.5 and 16 feet BGS.
- Collected soil samples from the explorations for laboratory testing and maintained a log of encountered soil and groundwater conditions in the explorations.
- Conducted infiltration testing in three of the test pits at depths of 3 and 6 feet BGS.
- Performed laboratory testing on select soil samples collected from the explorations, including the following:
  - Seven moisture content determinations in general accordance with ASTM D2216
  - Six particle-size analyses in general accordance with ASTM D1140
  - One particle-size analysis in general accordance with ASTM D6913
  - Two Atterberg limits tests in general accordance with ASTM D4318

- Prepared this geotechnical engineering report that includes the following:
  - Summary of subsurface conditions at the site
  - Results of research of existing geologic and seismic maps and literature to determine relevant seismic risks, including locations of faults and earthquake magnitudes
  - Assessment of seismic hazards
  - Laboratory testing results
  - Foundation support recommendations, including allowable bearing capacity, estimated foundation settlement, and lateral resistance parameters
  - Recommendations for floor slab subgrade preparation
  - Recommendations for retaining walls, including lateral earth pressures, backfill, compaction, and drainage
  - Recommendations for site preparation, including grading and drainage, stripping depths, fill type for imported material, compaction criteria, trench excavation and backfill, use of on-site soil, and wet/dry weather earthwork
  - Recommendations for managing groundwater conditions that may affect the performance of structures and site improvements
  - Stormwater disposal recommendations
  - Code-based seismic design parameters in accordance with the 2021 IBC

4.0 SITE CONDITIONS

4.1 GEOLOGY

The 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 portion of the Portland/Vancouver Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide.

The near-surface soil is expected to consist of Pleistocene- to Pliocene-aged, semi-consolidated, pebble- to cobble-sized sedimentary Conglomerate (QTc). The conglomerate is underlain by Oligocene aged Elkhorn Mountain basaltic andesite flows (Evarts and O'Connor 2008). Well logs for 26416 SE 8th Street indicate that the conglomerate extends to a depth of at least 160 feet BGS (Washington State Department of Ecology 2025).

The USDA Web Soil Survey identifies the surface soil as Hesson clay loam (USDA 2025). Hesson series soils are generally fine-grained clays and silts with low permeability, moderate to high water capacity, and low shear strength. They are generally moisture sensitive, somewhat compressible, and described as having low to moderate shrink-swell potential. The erosion hazard is slight primarily based on slope grade.

4.2 SEISMOLOGY

Recent research and subsurface mapping investigations in the Pacific Northwest appear to suggest the historical 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.

Three scenario earthquakes are possible with the local seismic setting. Two of the possible earthquake sources are associated with the 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.

4.2.1 CSZ

The CSZ is the region where the Juan de Fuca Plate is being subducted beneath the North American Plate. This subduction is occurring in the coastal region between Vancouver Island and northern California. Evidence has accumulated suggesting 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). The fault trace is mapped approximately 50 to 120 km off the Washington Coast.

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 M_W 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 M_W of up to 8.0.

4.2.2 Crustal Events

A significant earthquake could occur on a local fault near the site within the design life of the development. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, although the duration would be shorter. Table 1 provides information on local faults close to the site.

Table 1. Faults within the Site Vicinity[1]

Fault Name	Proximity to Site (km)	Mapped Length (km)
Lacamas Lake fault	1	24
Portland Hills fault	22	49
East Bank fault	27	29

1. Reported by USGS (2025)

4.3 SURFACE CONDITIONS

The site is relatively undeveloped and flat. The site is primarily forested and contains two single-family residential structures. According to Clark County GIS, site elevations range from approximately 382 feet at the northwest area of the site to 390 feet in the southeast area of the site (NAVD 88).

4.4 SUBSURFACE CONDITIONS

Subsurface conditions at the site were explored by excavating six test pits (TP-1 through TP-6) to depths between 12.5 and 16 feet BGS. The exploration locations are shown on Figure 2. A description of our field exploration program and the exploration logs are presented in Appendix A. A description of the laboratory testing program and the testing results are presented in Appendix B. Photograph taken during our subsurface explorations are presented in Appendix C. A summary of the subsurface conditions is presented below.

4.4.1 Root and Topsoil Zones

The topsoil zone is generally 6 to 12 inches thick and consists of sandy silt with trace organics. The topsoil zone generally contains a 3-inch-thick root zone. Areas covered by forest may have deeper root zones or thicker topsoil zones.

4.4.2 Near-Surface Soil

Beneath the topsoil, the soil generally consists of silty gravel with sand and cobbles or clayey sand to silty sand with gravel to the maximum depth explored of 16 feet BGS. Variable amounts boulders up to 24 inches in diameter were encountered in several locations. Based on laboratory testing, the moisture content varied from 23 to 30 percent at the time of exploration.

4.4.3 Groundwater

Groundwater seepage was observed in test pit TP-6 at a depth of 12 feet BGS on December 31, 2024. Based on our knowledge of the surrounding area, perched water could be present in isolated, discontinuous zones below the ground surface and particularly where higher infiltrating soil is present above lower infiltrating soil.

4.5 INFILTRATION TESTING

Infiltration testing was completed in three of the test pits in December 2024 to assist in the evaluation of stormwater infiltration facilities for the project. The infiltration testing was conducted in general accordance with the recommendations for the encased falling head method in general accordance with the Clark County Stormwater Manual (Clark County 2021). Table 2 summarizes our infiltration testing results.

Table 2. Infiltration Testing Results

Location	Depth (feet BGS)	Soil Type	Fines Content[1] (percent)	Coefficient of Permeability, k (in/hr)
TP-1	3	Silty GRAVEL with sand (GM)	35	4
	6	Silty GRAVEL with sand (GM)	21	19
TP-3	3	Silty GRAVEL with sand (GM)	32	5
	6	GRAVEL with silt and sand (GP-GM)	20	4
TP-6	3	Clayey SAND (SC)	43	5
	6	Silty GRAVEL with sand (GM)	14	20

1. Fines content: percent passing U.S. Standard No. 200 sieve

Recommendations for design of infiltration system are provided in Section 6.6.3 (Stormwater Infiltration Systems).

5.0 GEOLOGICALLY HAZARDOUS AREAS

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 are identified: (1) erosion hazard areas, (2) landslide hazard areas, and (3) seismic hazard areas.

Columbia West conducted a geologic hazard review to assess whether these hazards are present at the site proposed for development and, if so, to provide mitigation recommendations. The geologic hazard review was based on physical and visual reconnaissance, subsurface exploration, laboratory testing 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.

5.1 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 on review of slope grade mapping published by Clark County Maps Online, maximum slope grades of 15 percent are mapped in the northeast corner of the site. Therefore, site slopes do not meet the definition of an erosion hazard according to Camas Municipal Code.

5.2 LANDSLIDE HAZARDS

Columbia West conducted a review of available mapping and Clark County GIS data and conducted a site reconnaissance to evaluate the potential presence of a landslide hazard on or near the site. Due to the relatively flat topography, the site does not pose a significant landslide hazard.

5.3 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.

5.3.1 Soil Liquefaction and Dynamic Settlement

According to the Liquefaction Susceptibility Map of Clark County, Washington (Palmer et al. 2004), the site is mapped as very low susceptibility for liquefaction. 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.

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

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

5.3.2 Ground Shaking Amplification

Review of the Site Class Map of Clark County, Washington, (Palmer et al. 2004) indicates that site soil may be represented by Site Class C as defined in 2021 IBC Section 1613.3.2. A designation of Site Class C indicates that minor amplification of seismic energy may occur during a seismic event due to subsurface conditions. However, this is typical for many areas within Clark County, 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 6.3 (Seismic Design Criteria).

5.3.3 Fault Rupture

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

6.0 DESIGN

Based on the results of our explorations, laboratory testing, and analysis, the proposed project is feasible, provided the recommendations presented in this report are incorporated into design and implemented during construction.

6.1 SHALLOW FOUNDATION SUPPORT

6.1.1 General

The 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 the moisture content of the footing subgrade soil is above optimum moisture content, we recommend that a minimum of 6 inches of compacted aggregate be placed over exposed subgrade soil. The aggregate pad should extend 6 inches beyond the edges of the foundations and consist of imported granular material as described in Section 7.6.1 (Structural Fill). Columbia West should observe exposed subgrade conditions prior to placement of crushed aggregate to verify adequate subgrade support.

6.1.2 Footing Dimensions and Bearing Capacity

Continuous perimeter wall and isolated spread footings should have minimum widths of 18 and 24 inches, respectively. The bases of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bases of interior footings should be 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 1,500 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.

6.1.3 Settlement

Provided the subgrade soil is prepared as described above and in Section 7.1 (Site Preparation), 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.

6.1.4 Resistance to Sliding

Lateral foundation loads can be resisted by passive earth pressure on the sides of footings and by friction at the bases of 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 down-gradient 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.

6.2 *FLOOR SLABS*

Floor slabs can be supported on firm, competent, native soil or engineered structural fill prepared as described in this report. Disturbed soil and unsuitable fill in proposed slab locations, if encountered, should be removed and replaced with structural fill. Floor slabs with a maximum floor load of 100 psf may be designed assuming a modulus of subgrade reaction, k, of 125 pci.

To provide a capillary break, slabs should be underlain by at least 6 inches of compacted crushed aggregate that contains less than 5 percent fines by dry weight. Geotextile may be used below the crushed aggregate layer to increase subgrade support. Recommendations for floor slab aggregate base and subgrade geotextile are discussed in Section 7.6 (Materials).

6.3 SEISMIC DESIGN CRITERIA

The structures will likely be constructed in accordance with the 2021 IBC, which references ASCE 7-16 for design parameters. Based on our literature review of surrounding sites, the appropriate seismic site class for design is C. Seismic design parameters in accordance with ASCE 7-16 are provided in Table 3.

Table 3. ASCE 7-16 Seismic Design Parameters[1]

Parameter	Short Period (T_s)	1-Second Period (T_1)
MCE spectral response acceleration, S	S_s = 0.787 g	S_1 = 0.345 g
Site class	C	
Site coefficient, F	F_a = 1.2	F_v = 1.5
Adjusted spectral response acceleration, S_M	S_{MS} = 0.945 g	S_{M1} = 0.518 g
Design spectral response acceleration, S_D	S_{DS} = 0.630 g	S_{D1} = 0.345 g

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

Columbia West recommends the project structural engineer evaluate the requirements and exceptions presented in ASCE 7-16 to determine if the parameters for Site Class C provided in Table 3 can be used for design or if a site-specific seismic hazard evaluation is required.

6.4 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 the specifications in Section 6.1 (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 soil 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 WSS 9-03.12(2) – Gravel Backfill for Walls, equivalent fluid pressures of 34 pcf and 60 pcf are applicable for active and at-rest earth pressures, respectively.

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 pressure 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 $9H^2$ 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.

6.4.1 Wall Drainage and Backfill

A minimum 6-inch-diameter, perforated collector pipe should be placed at the bases 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 in Section 7.6 (Materials). Perforated collector pipes should discharge at an appropriate location away from the base of the wall. Discharge pipes 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 walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular material placed and compacted as described in Section 7.6.1 (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.

6.5 *PAVEMENT*

We recommend that public roadways for the subdivision be constructed in accordance with City of Camas standards. For dry weather construction, pavement surface sections should bear on competent subgrade consisting of scarified and compacted native soil or engineered structural fill. Wet weather construction may require an increased thickness of aggregate base as discussed in Section 7.2 (Construction Traffic and Staging). Refer to Section 7.6.3.2 (Cold Weather Paving Considerations) for compaction requirements.

6.6 *DRAINAGE*

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. In general, drainage design 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 in Section 7.6 (Materials). Drains should be closely monitored after construction to assess their effectiveness. If additional surface or shallow subsurface seepage become evident, the drainage provisions may require modification or additional drains. We should be consulted to provide appropriate recommendations.

6.6.1 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-diameter, 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 0.5 percent. The invert elevation of the drainpipe should be at least 18 inches below the elevation of the floor slab.

6.6.2 Subdrains

Subdrains should be considered if portions of the site are cut below surrounding grades. Shallow groundwater or seepage should be conveyed via a 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 subsurface drainage may result in soil slumping or unanticipated settlement of structures exceeding tolerable limits.

6.6.3 Stormwater Infiltration Systems

Based on the tested infiltration rates, on-site infiltration systems are viable in the native soil at the site. The rates in Table 2 are field infiltration rates and factors of safety have not been applied. Correction factors should be applied to the recommended infiltration rates to account for soil variations and the potential for long-term clogging due to siltation and buildup of organic material. Confirmation testing of infiltration systems should be completed as described below. In addition, the local jurisdiction may require a limit on the design infiltration rates. We recommend the stormwater system designer determine if a design rate limit is required.

We recommend a contingency be in place if tested rates do not meet design rates. Columbia West should be allowed to review the final design and provide comments, as necessary. The infiltration flow rate of disposal systems will diminish over time as suspended solids and precipitates in the stormwater slowly clog the void spaces between soil particles in the zone of infiltration. Accordingly, systems may eventually fail and need to be replaced.

7.0 CONSTRUCTION

7.1 SITE PREPARATION

7.1.1 General

Site grading should be performed in accordance with the requirements specified in the 2021 IBC, Chapter 18 and Appendix J, with exceptions noted in this report. Site preparation should be observed and documented by Columbia West.

7.1.2 Demolition

Where required, demolition includes removal of structural features that may be at the site. Abandoned foundations and utilities, if present, will need to be removed and the resulting excavations backfilled. Utility lines should be completely removed or, with prior approval, grouted full if left in place. Excavations left from demolition and removal of existing structures should be backfilled with compacted structural fill in accordance with the recommendations in Section 7.6 (Materials).

7.1.3 Stripping and Grubbing

The existing root zones should be stripped and removed from all areas to receive new structural improvements. A stripping depth of approximately 12 inches is anticipated in areas where the entire topsoil zone is removed. The actual stripping depth should be based on field observations at the time of construction and may increase in areas of heavy vegetation or deep tree roots. Stripped material should be transported offsite for disposal or used in landscaped areas on slopes less than 25 percent. The post-construction maximum depth of landscape fill placed or spread at any location onsite should not exceed 1 foot.

Trees and shrubs should be removed from fill areas. In addition, root balls should be grubbed out to the depth of the roots, which could exceed 3 feet BGS. Depending on the methods used to remove root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill. Columbia West recommends removing undocumented fill completely and backfilling, as needed, with clean structural fill. Undocumented structural fill material should be evaluated by Columbia West prior to being reused as structural fill to determine suitability.

7.1.4 Test Pits

Test pits excavated during our explorations were backfilled loosely with on-site soil. These excavations should be located and properly backfilled with structural fill during site improvement construction. Trees, stumps, and associated roots should also be removed from structural areas, individually and carefully. Resulting cavities and excavation areas should be backfilled with engineered structural fill.

7.1.5 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-tired 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.

7.2 CONSTRUCTION TRAFFIC AND STAGING

The fine-grained soil present on this site is easily disturbed. If not carefully executed, site preparation, utility trench work, and roadway excavation can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The aggregate base thickness for pavement areas is intended to support post-construction design traffic loads and is not designed to support construction traffic. Moreover, if construction is planned for periods when the subgrade soil is wet, staging areas and haul roads with increased thicknesses of base rock will be required. The amount of staging areas and haul roads, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment and should, therefore, be the responsibility of the contractor. Based on our experience, between 12 and 18 inches of imported granular material are generally required in staging areas and between 18 and 24 inches in haul roads. The contractor should also be responsible for selecting the type of material for construction of haul roads and staging areas. A geotextile fabric can be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic to help prevent silt migration into the base rock. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in Section 7.6 (Materials).

Cement amendment is an alternative to thickened crushed rock sections, haul roads, and utility work zones. Cement amendment recommendations are presented in Section 7.6.4 (Soil Amendment with Cement).

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.

7.3 CUT AND FILL SLOPES

Fill slopes should consist of structural fill material as discussed in Section 7.6.1 (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 6 feet in height should be vertically keyed into the existing subsurface soil. 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. The 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 the top of the cut or fill slope face or overall slope height divided by three (H/3), whichever is greater.

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 2 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.

7.4 EXCAVATION

7.4.1 General

Conventional earthmoving equipment in proper working condition should be capable of making necessary site excavations. Temporary excavation sidewalls should maintain a vertical cut to a depth of approximately 4 feet BGS in the near-surface silt, 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 1.5H:1V and groundwater seepage does not occur. Excavation side slopes should be reduced to a stable inclination if excessive sloughing or raveling occurs.

Groundwater seepage was observed in test pit TP-6 at a depth of 12 feet BGS on December 31, 2024. Recommendations as described in Section 7.5 (Dewatering) should be considered where subsurface construction activities intersect the shallow groundwater table.

Shoring may be required if open-cut excavations are not feasible. 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. If box shoring is used, the contractor should understand it is a safety feature used to protect workers and does not prevent caving. If excavations are left open, caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

All excavations should be made in accordance with applicable OSHA requirements and regulations of the state, county, and local jurisdiction. While this report describes certain approaches to excavation and dewatering, the contract documents should specify that the contractor is responsible for selecting the excavation and dewatering methods, monitoring the excavations for safety, and providing shoring (as required) to protect personnel and adjacent structural elements.

7.4.2 Cobbles and Boulders

Cobbles and boulders were encountered in the explorations at the site. Construction considerations associated with cobbles and boulders include the following:

- Excavations can become difficult, if not impossible, with conventional equipment.
- Excavation volumes for utility trenches may be greater than anticipated due to sloughing and the need to remove oversized material.
- We recommend that project bid documents include a contingency for boulder removal, as well as the associated increased trench volumes for backfilling.

7.5 DEWATERING

Groundwater or perched water tables may be encountered at the site. Therefore, groundwater may be encountered in utility trench excavations and in areas of cut. General recommendations for temporary construction dewatering are presented in the following section.

7.5.1 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 soil. During rough and finished grading of building pad areas, the contractor should keep all footing excavations and slab subgrade soil free of standing water.

The contractor's proposed dewatering plan should be capable of maintaining groundwater levels at least 2 feet below the bases 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 bases 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 soil 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 in Section 7.6 (Materials).

7.6 MATERIALS

7.6.1 Structural Fill

7.6.1.1 General

Areas proposed for fill placement should be appropriately prepared as described in Section 7.1 (Site Preparation). Engineered fill placement should be observed by Columbia West. Compaction of engineered structural fill should be verified by proof rolling or 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, should have a maximum particle size of less than 6 inches, and should meet the specifications provided in the following sections. Representative samples of proposed engineered structural fill should be submitted for laboratory testing and approval by Columbia West prior to placement.

7.6.1.2 On-Site Soil

The near-surface soil at the site generally consists of fine-grained soil. The native surficial soil at the site is generally suitable for use as structural fill if adequately dried or moisture conditioned to achieve recommended compaction specifications. Based on laboratory testing, we anticipate the moisture content of the soil will generally be above the optimum moisture content required to meet compaction requirements and drying of the soil will be necessary. Accordingly, extended dry weather will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact the on-site soil during the rainy season or during prolonged periods of rainfall.

On-site soil used as structural fill should be placed in loose lifts not exceeding 8 inches in thickness 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 ASTM D698.

The on-site soil will likely expand during excavation and transport and consolidate during compaction. Development of site-specific expansion and consolidation factors is beyond the scope of this study. We can provide site-specific factors upon request.

7.6.1.3 Imported Granular Material

Imported granular material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and sand. Imported granular material should be placed in loose lifts not exceeding 12 inches in thickness and compacted to at least 95 percent of maximum dry density as determined by 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.

7.6.1.4 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 organic material 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 compacted to a firm, unyielding condition. Equipment with vibratory action should not be used when compacting stabilization material over wet, fine-grained soil. 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.

7.6.1.5 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 the specifications in WSS 9-03.12(3) – Gravel Backfill for Pipe Zone Bedding. Pipe zone backfill should be compacted to at least 90 percent of maximum dry density as determined by 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 material meeting the specifications in WSS 9-03.19 – Bank Run Gravel for Trench Backfill or WSS 9-03.14(2) – Select Borrow with a maximum particle size of 2½ 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 maximum dry density as determined by 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 maximum dry density as determined by ASTM D1557 or as required by the local jurisdictional agency or pipe manufacturer.

7.6.1.6 Pavement and Floor Slab Aggregate Base

Imported granular material used as base rock for building floor slabs and pavement should consist of 1¼-inch-minus crushed aggregate meeting the specifications in WSS 9-03.9(3) – Crushed Surfacing. Pavement and slab aggregate base should be compacted to at least 95 percent of maximum dry density as determined by ASTM D1557.

7.6.1.7 Retaining Wall Backfill

Backfill placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular material as described above and should have less than 7 percent fines by dry weight. We recommend the wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

Wall backfill should be compacted to a minimum of 95 percent of maximum dry density as determined by ASTM D1557. However, backfill located within a horizontal distance of 3 feet from a retaining wall should only be compacted to approximately 90 percent of maximum dry density as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (e.g., jumping jack or vibratory plate compactor). If flatwork (sidewalks or pavement) will be placed atop the wall backfill, we recommend that the upper 2 feet of material be compacted to 95 percent of maximum dry density as determined by ASTM D1557.

7.6.1.8 Retaining Wall Leveling Pad

Crushed aggregate used as a leveling pad for retaining wall footings should consist of ¾- or 1¼-inch-minus crushed rock and should have less than 7 percent fines by dry weight. The leveling pad material should be compacted to at least 95 percent of maximum dry density as determined by ASTM D1557.

7.6.1.9 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches. The material should be free of roots, organic material, and other unsuitable material; should have less than 2 percent fines by dry weight; and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

7.6.2 Geotextile Fabric

7.6.2.1 Subgrade Geotextile

A geotextile separation fabric will be required at the interface of the existing soil and imported granular material beneath proposed walls. In addition, geotextile fabric may be required where soft subgrade is encountered. The separation fabric should meet the specifications in WSS 9-33.2(1) – Geotextile Properties (Table 3) for soil separation. The geotextile should be installed in conformance with the specifications in WSS 2-12 – Construction Geosynthetic.

7.6.2.2 Drainage Geotextile

Drainage geotextile should meet the specifications in WSS 9-33.2(1), Table 2, Geotextile for Underground Drainage Filtration Properties. The AOS should be between U.S. Standard No. 70 and No. 100 sieves. 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.

7.6.3 Pavement

7.6.3.1 AC

The AC should conform to the specifications in WSS 5-04 – Hot Mix Asphalt and WSS 9-03.8 – Aggregates for Hot Mix Asphalt. The asphalt cement binder should be PG 28-22 Performance Grade Asphalt Cement meeting WSS 9-02.1(4) – Performance Graded (PG) Asphalt Binder. The AC should be ½-inch HMA. The lift thickness should be 2 to 3 inches. The AC should be compacted to 92 percent of maximum specific gravity of the mix as determined by ASTM D2041.

7.6.3.2 Cold Weather Paving Considerations

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. In Washington, the AC surface temperature during paving should be at least 40 degrees Fahrenheit for lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thickness between 2 and 2.5 inches.

If AC paving activities must take place during cold weather construction as defined above, the contractor and design team should discuss options for minimizing risk of pavement serviceability.

7.6.4 Soil Amendment with Cement

The on-site soil can be amended with portland cement to obtain suitable properties for use as wet weather structural fill or subbase for pavement. The effectiveness of soil amendment is highly dependent on proper mixing techniques, soil moisture conditioning, and the quantity of cement. The quantity of cement applied during amendment should be based on an assumed dry unit weight of 100 pcf for the site soil.

7.6.4.1 Subbase Stabilization

Specific recommendations for soil amendment should be based on exposed site conditions at the time of construction. For preliminary design purposes, we recommend cement-amended subgrade for building pads and pavement subbase (below the aggregate base layer) achieve a target strength of 100 psi. The quantity of cement required to achieve the target strength will vary with moisture content and soil type. Laboratory testing of cement-amended soil should be used to confirm design expectations.

Based on our experience, near-surface soil will require approximately 6 to 7 percent cement by weight to achieve the target strength of 100 psi. This cement percentage assumes that the soil moisture content does not exceed 20 percent at the time of amendment. If the soil moisture content is in the range of 25 to 35 percent, 7 to 8 percent cement by weight may be required to achieve the target strength. The amount of cement added to the soil at the time of construction should be based on observed field conditions and subgrade performance. During extended periods of dry weather, water may need to be applied during the amendment and tilling process to achieve the optimum moisture content required for compaction.

Cement-amendment equipment should have balloon tires to minimize softening, rutting, and disturbance of fine-grained site soil. A sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction. Rollers with vibratory action should not be used to compact fine-grained, cement-amended soil. Final compaction should be conducted with a smooth-drum roller with a minimum applied linear force of 700 pounds per inch. The amended soil should be compacted to at least 95 percent of maximum dry density as determined by ASTM D558.

Following cement amendment, a minimum curing time of four days is required prior to exposure to construction traffic. Construction traffic should not be allowed on unprotected, cement-amended subgrade. To protect cement-amended areas from damage, the finished surface should be covered with 4 to 6 inches of imported granular material. The protective layer of crushed rock often becomes contaminated with soil during construction, particularly in staging and haul road areas. Contaminated aggregate, where present, should be removed and replaced with clean crushed aggregate prior to construction of pavement or other permanent site improvements supported by aggregate base.

Cement amendment should not be attempted during moderate to heavy precipitation or when the ambient air temperature is below 40 degrees Fahrenheit. Cement should not be placed in areas of standing water or where saturated subgrade conditions exist.

7.6.4.2 Cement-Amended Structural Fill

If adequate compaction is not achievable with the on-site fine-grained soil due to moisture or weather conditions, the soil may be cement amended and placed as general structural fill. Prior to placement of cement-amended fill, subgrade soil should be prepared as described in Section 7.1 (Site Preparation). Where multiple lifts of cement-amended fill are necessary to meet finished grade, consecutive lifts may be placed immediately following amendment and compaction of the underlying lift. However, where the final lift of cement-amended fill will serve as building pad or pavement subbase material, the four-day cure period as discussed above is recommended.

7.6.4.3 QA/QC Testing and Inspection

Cement amendment of site soil should be observed and tested by Columbia West to document conformance with design recommendations. Cement spread rate should be verified with a pan sample test conducted at one random location per lift per 20,000 square feet of cement-amended fill. Amendment depth should be verified through excavation of a small test pit and measurement at one random location per lift of cement-amended fill. Adequate compaction and moisture content should be verified by conducting nuclear gauge density testing at a frequency of approximately one test per 5,000 square feet of cement-amended fill in accordance with ASTM D6938. At least one representative sample should be collected per day of cement amendment, cured for seven days, and tested for unconfined compressive strength in accordance with ASTM D1633. The tested samples should have a minimum seven-day, unconfined compressive strength of 100 psi.

7.7 EROSION CONTROL

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

8.0 OBSERVATION OF CONSTRUCTION

Satisfactory pavement, earthwork, and foundation performance depends to a large degree on the quality of construction. 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. Columbia West should be retained to observe subgrade preparation, fill placement, foundation excavations, drainage system installation, and pavement placement and to review laboratory compaction and field moisture-density information.

Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

9.0 LIMITATIONS

We have prepared this report for use by the addressee and members of the design and construction team for the proposed project. This report is subject to the limitations expressed in Appendix D.

♦ ♦ ♦

We appreciate the opportunity to be of service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

Michael A. Chacon, PE
Senior Staff Engineer

Daniel E. Lehto, PE, GE
Principal Engineer

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FIGURES

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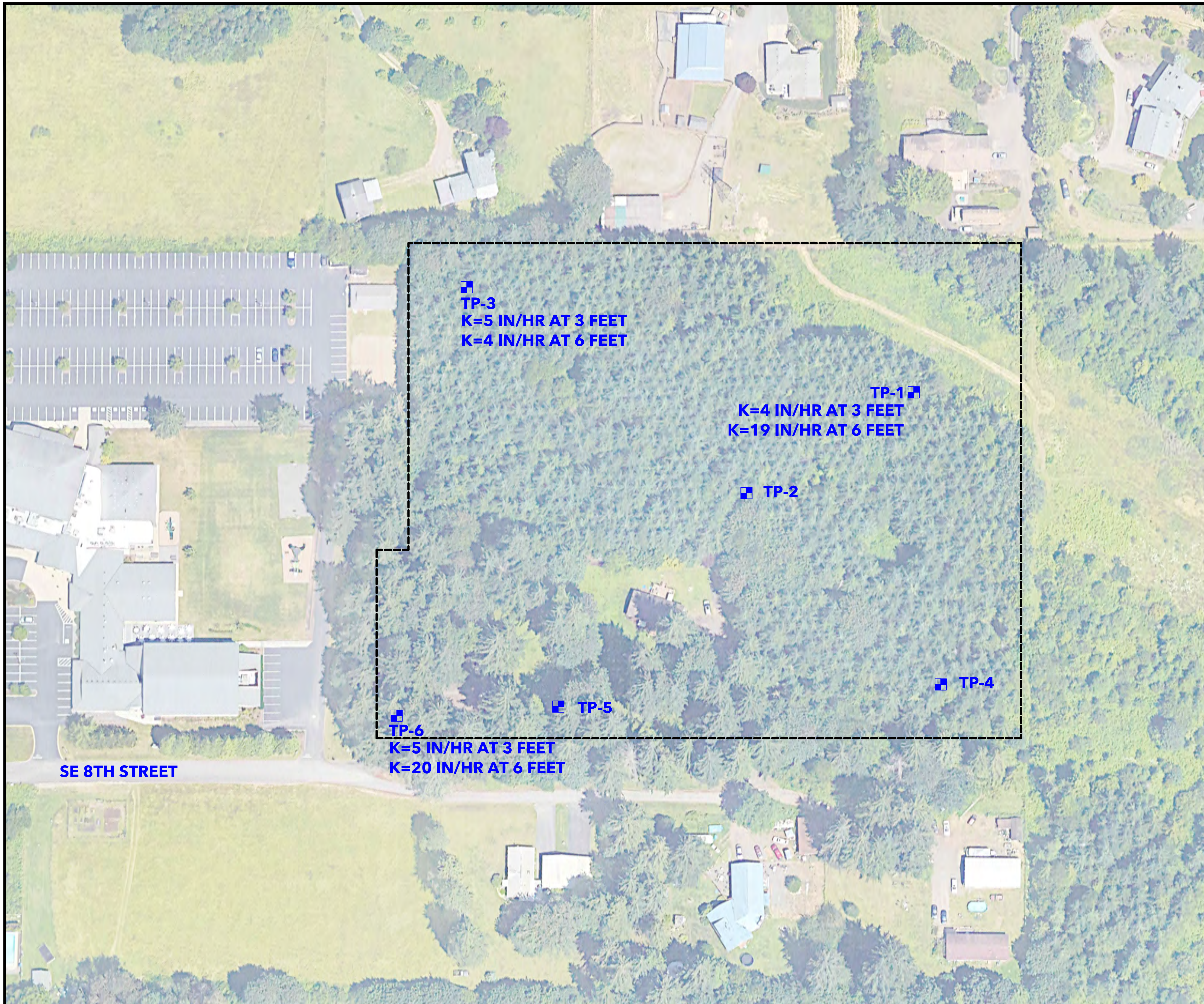
PROJECT NO.: HSR-4-01-1

FEBRUARY 2025

VICINITY MAP
CAMAS WOODS PHASE 3
CAMAS, WASHINGTON

FIGURE 1



LEGEND

SITE BOUNDARY

TEST PIT

K UNFACTORED COEFFICIENT OF PERMEABILITY

N

0 26 52 78 80 130 260

NOTES:
1. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
2. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
3. REFER TO REPORT TEXT FOR EXPLORATION DESCRIPTIONS.

Columbia West Engineering, Inc
Geotechnical • Environmental • Special Inspections

CAMAS WOODS PHASE 3
CAMAS, WASHINGTON
26514 AND 26416 SE 8TH STREET

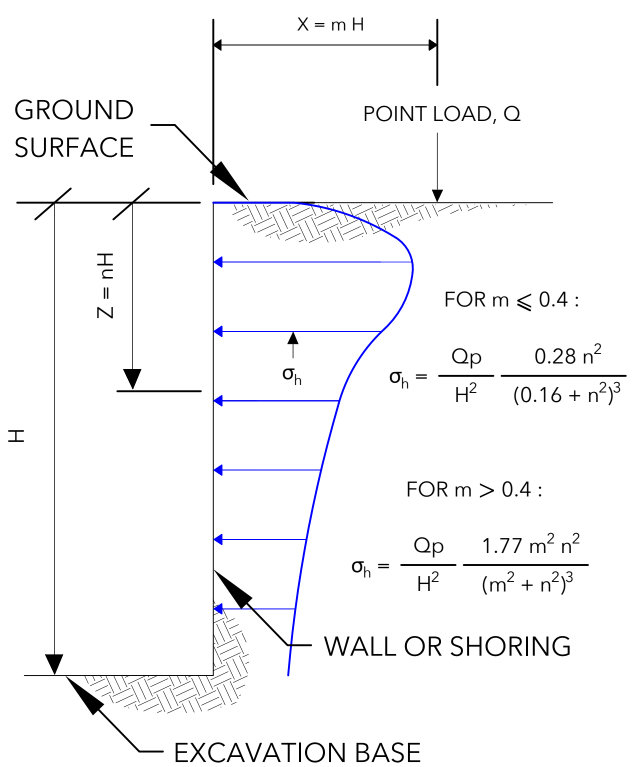
SITE PLAN

PROJECT NO:
HSR-4-01-1

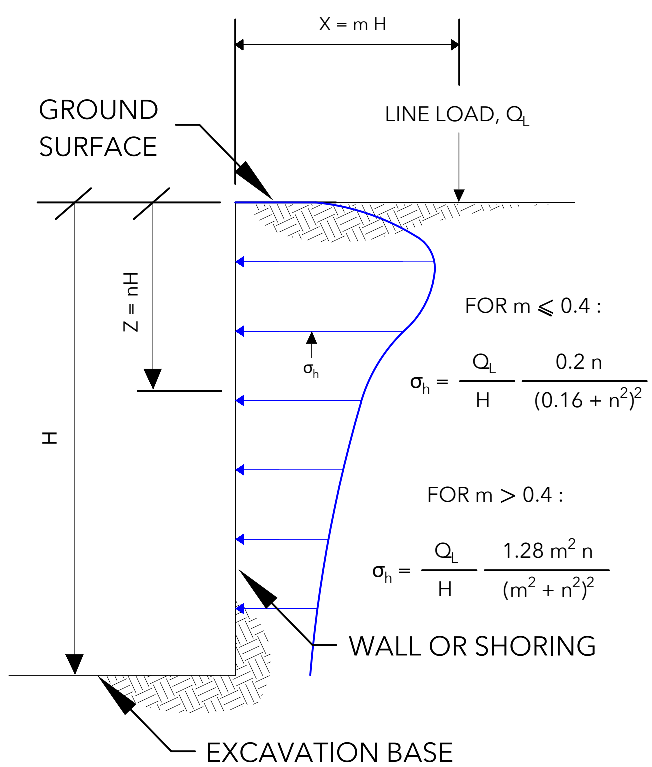
FEBRUARY 2025

FIGURE
2

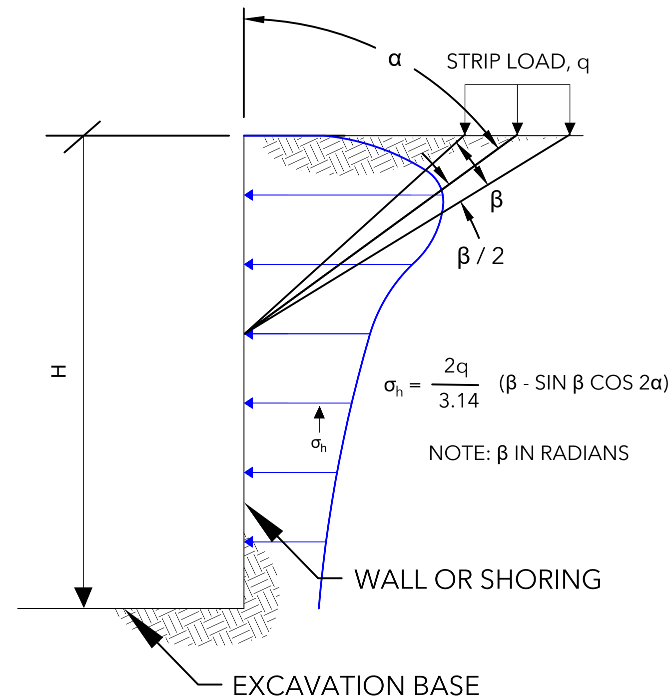
VERTICAL POINT LOAD



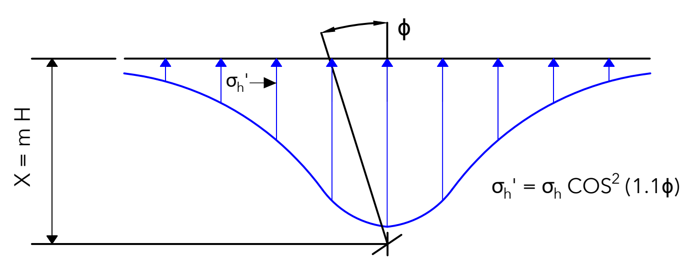
LINE LOAD PARALLEL TO WALL



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.

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SURCHARGE-INDUCED LATERAL EARTH PRESSURES

FIGURE 3

APPENDIX A

APPENDIX A
FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by excavating six test pits (TP-1 through TP-6) to depths between 12.5 and 16 feet BGS. Excavation services were provided by L&S Contracting LLC of Yacolt, Washington, on December 31, 2024. The explorations were logged on a full-time basis by Columbia West personnel. The exploration logs are presented in this appendix.

The approximate exploration locations are shown on Figure 2. The exploration locations are approximate and were not surveyed.

SOIL SAMPLING

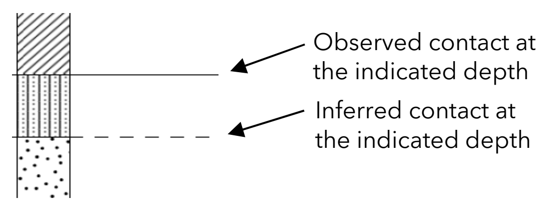
Representative disturbed samples of soil observed in the test pit explorations were collected from the test pit walls and base using the excavator bucket.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the "Exploration Key," "Soil Classification System," and "AASHTO Soil Classification System," which are presented in this appendix. The exploration logs indicate 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 logs.

EXPLORATION LEGEND

SAMPLER TYPE	DESCRIPTION
SPT	Sample collected from the indicated depth in general accordance with ASTM D1586, *Standard Test Method Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils,* using an SPT sampler and 140-pound hammer
SH	Sample collected from the indicated depth in general accordance with ASTM D1587, *Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes,* using a thin-walled Shelby tube
D&M	Sample collected from the indicated depth in general accordance with ASTM D3550, *Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils,* using a Dames & Moore sampler and 140-pound hammer or pushed
CSS	Sample collected from the indicated depth in general accordance with ASTM D3550, *Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils,* using a 3-inch-outside diameter California split-spoon sampler and 140-pound hammer
DP	Sample collected from the indicated depth in general accordance with ASTM D6282, *Standard Guide for Direct Push Soil Sampling for Environmental Site Characterizations,* using a direct push soil sampler
GRAB	Grab sample collected from the indicated depth
CORE	Pavement or rock core interval at the indicated depth



GEOTECHNICAL ABBREVIATIONS			
ATT	Atterberg limits	PP	Pocket penetrometer
CBR	California bearing ratio	P200	Percent passing No. 200 sieve
CON	Consolidation test	RES	Resilient modulus
DD	Dry density	SIEV	Sieve analysis
DS	Direct shear	TS	Torvane shear
HYD	Hydrometer	tsf	Tons per square foot
MC	Moisture content	UC	Unconfined compressive strength
MD	Moisture-density relationship	UU	Unconsolidated undrained triaxial test
NP	Non-plastic	VS	Vane shear
OC	Organic content	WD	Wet density
ENVIRONMENTAL ABBREVIATIONS			
CA	Sample submitted for chemical analysis	ND	Not detected
		NS	No sheen
PID	Photoionization detector headspace analysis	SS	Slight sheen
		MS	Moderate sheen
ppm	Parts per million	HS	Heavy sheen

SOIL CLASSIFICATION SYSTEM

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	0 to 2	0 to 3	Less than 0.25
Soft	2 to 4	3 to 6	0.25 to 0.5
Medium stiff	4 to 8	6 to 12	0.5 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	Greater than 30	Greater than 30	Greater than 4.0

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

TERM	FIELD IDENTIFICATION
Dry	Very low moisture, dry to touch
Moist	Damp, color appears darkened, without visible moisture, cohesive soil will clump, sand will bulk
Wet	Visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Percent	SILT AND CLAY IN		Percent	SAND AND GRAVEL IN		Percent	SECONDARY MATERIAL
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil		Organics and Man-Made Debris
< 5	trace	trace	< 5	trace	trace	< 4	trace
5 - 12	minor	with	5 - 15	minor	minor	4 - 12	some
> 12	some	silty/clayey	15 - 30	with	with		
			> 30	sandy/gravelly	with		

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES

General Classification	Granular Materials (35 Percent or Less Passing No. 200 Sieve [0.075 mm])			Silty-Clay Materials (More Than 35 Percent Passing No. 200 Sieve [0.075 mm])			
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. 40)							
Liquid limit	--	--	**	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.	Non-plastic	**	10 max.	10 max.	11 min.	11 min.
General rating as subgrade	Excellent to Good			Fair to Poor			

* 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.
** See Table 2 for values.

TABLE 2. CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES

General Classification	Granular Materials (35 Percent or Less Passing No. 200 Sieve [0.075 mm])							Silty-Clay Materials (More Than 35 Percent Passing No. 200 Sieve [0.075 mm])			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 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.		Non-plastic	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual types of significant constituent materials	Stone fragments Gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soil		Clayey soil	
General rating as subgrade	Excellent to Good							Fair to Poor			

*Plasticity index of A-7-5 subgroup is equal to or less than liquid limit minus 30 (i.e., plastic limit greater than 30 percent). Plasticity index of A-7-6 subgroup is greater than liquid limit minus 30 (i.e., plastic limit less than 30 percent).

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TEST PIT NUMBER: TP-1

PROJECT NAME Camas Woods Phase 3
CLIENT HSR Capital LLC
PROJECT NO. HSR-4-01-1 **LOGGED BY** S. Chandra
PROJECT LOCATION Camas, Washington
CONTRACTOR L&S Contracting LLC
EQUIPMENT CAT 307E2
CAVING Not observed
DATE COMPLETED 12/31/2024
GROUNDWATER Not observed
TIME STARTED 11:20 AM **TIME COMPLETED** 2:22 PM

DEPTH (ft)	SAMPLE ID	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	POCKET PEN (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS (LL-PL-PI)	FINES (%)	REMARKS
				Medium stiff, brown sandy SILT, trace organics, moist (12 inches of topsoil, 3-inch-thick root zone). 1.0					
	TP1.1		GM	Medium dense, brown silty GRAVEL with sand and cobbles, moist, gravel is fine to coarse, sand is fine, cobbles are subrounded and up to 12 inches in diameter.					
	TP1.2					29	48-30-18	34	Infiltration test at 3 feet.
5									
	TP1.3					23		21	Infiltration test at 6 feet. Increase in cobbles at 6 feet.
									Decrease in fines at 9 feet.
10									
				With boulders at 11 feet.					
				15.0					
15	TP1.4		SM	Medium dense, brown-tan-orange silty SAND with gravel, moist, sand is fine to coarse, gravel is fine. 16.0					
				Exploration completed at 16 feet.					

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TEST PIT NUMBER: TP-2

PROJECT NAME	Camas Woods Phase 3	**CLIENT**	HSR Capital LLC
PROJECT NO.	HSR-4-01-1 **LOGGED BY** S. Chandra	**PROJECT LOCATION**	Camas, Washington
CONTRACTOR	L&S Contracting LLC	**EQUIPMENT**	CAT307E2
CAVING	Not observed	**DATE COMPLETED**	01/01/2025
GROUNDWATER	Not observed	**TIME STARTED**	11:32 AM **TIME COMPLETED** 12:07 PM

DEPTH (ft)	SAMPLE ID	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	POCKET PEN (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS (LL-PL-PI)	FINES (%)	REMARKS
				Medium stiff, brown sandy SILT, trace organics, moist (10 inches of topsoil, 4-inch-thick root zone). 0.8					
			GM	Medium dense, brown silty GRAVEL with sand and cobbles, moist, gravel is fine to coarse, sand is fine, cobbles are subrounded and up to 12 inches in diameter.					
	TP2.1					24	53-31-22	25	AASHTO soil classification: A-2-7(1)
5	TP2.2								
									Decrease in fines at 9 feet.
10									
	TP2.3			12.5					
				Exploration completed at 12.5 feet.					
15									

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TEST PIT NUMBER: TP-3

PROJECT NAME Camas Woods Phase 3
CLIENT HSR Capital LLC
PROJECT NO. HSR-4-01-1 **LOGGED BY** S. Chandra
PROJECT LOCATION Camas, Washington
CONTRACTOR L&S Contracting LLC
EQUIPMENT CAT307E2
CAVING Minor at 6.5 feet
DATE COMPLETED 12/31/2024
GROUNDWATER Not observed
TIME STARTED 9:49 AM **TIME COMPLETED** 10:50 AM

DEPTH (ft)	SAMPLE ID	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	POCKET PEN (tsf)	MOISTURE CONTENT (%)	FINES (%)	REMARKS
				Medium stiff, brown sandy SILT, trace organics, moist (12 inches of topsoil, 5-inch-thick root zone). 1.0				
	TP3.1		GM	Medium dense, brown silty GRAVEL with sand and cobbles, moist, gravel is fine to coarse, sand is fine, cobbles are subrounded and up to 12 inches in diameter.				
	TP3.2					27	32	Infiltration test at 3 feet.
5				6.0				
	TP3.3		GP-GM	Medium dense, brown GRAVEL with silt, sand, and cobbles, moist, gravel is fine to coarse, sand is fine, cobbles are subrounded to rounded and up to 12 inches in diameter.		25	20	Infiltration test at 6 feet. Minor caving at 6.5 feet.
10								
	TP3.4			13.5				
				Exploration completed at 13.5 feet.				
15								

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TEST PIT NUMBER: TP-4

PROJECT NAME Camas Woods Phase 3
CLIENT HSR Capital LLC
PROJECT NO. HSR-4-01-1 **LOGGED BY** S. Chandra
PROJECT LOCATION Camas, Washington
CONTRACTOR L&S Contracting LLC
EQUIPMENT CAT307E2
CAVING Not observed
DATE COMPLETED 12/31/2024
GROUNDWATER Not observed
TIME STARTED 12:33 PM **TIME COMPLETED** 1:05 PM

DEPTH (ft)	SAMPLE ID	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
0.0–0.8				Medium stiff, brown sandy SILT, trace organics, moist (10 inches of topsoil, 3-inch-thick root zone). 0.8			
0.8–14.0	TP4.1 (5); TP4.2 (9)		GM	Medium dense, brown silty GRAVEL with sand and cobbles, moist, gravel is fine to coarse, sand is fine to medium, cobbles are subrounded and 3 to 12 inches in diameter. With boulders, boulders are 18 to 24 inches in diameter at 13 feet. 14.0			Decrease in fines at 9 feet.
14.0–15.5	TP4.3 (15)		SM	Medium dense, brown-orange-tan silty SAND with gravel, moist, sand is fine to medium, gravel is coarse. 15.5			

Exploration completed at 15.5 feet.

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TEST PIT NUMBER: TP-5

PROJECT NAME Camas Woods Phase 3
CLIENT HSR Capital LLC
PROJECT NO. HSR-4-01-1 **LOGGED BY** S. Chandra
PROJECT LOCATION Camas, Washington
CONTRACTOR L&S Contracting LLC
EQUIPMENT CAT307E2
CAVING Not observed
DATE COMPLETED 12/31/2024
GROUNDWATER Not observed
TIME STARTED 9:10 AM **TIME COMPLETED** 9:40 AM

DEPTH (ft)	SAMPLE ID	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
0–0.5				Medium stiff, brown sandy SILT, trace organics, moist (6 inches of topsoil, 3-inch-thick root zone). 0.5			
0.5–2.5	TP5.1		SC	Medium dense, brown clayey SAND with gravel, moist, sand is fine, gravel is fine to coarse. 2.5			
2.5–13.5	TP5.2 TP5.3		GM	Medium dense, brown silty GRAVEL with sand, cobbles, and boulders, moist, gravel is fine to coarse, sand is fine, cobbles are rounded to subrounded, boulders are subrounded and up to 16 inches in diameter. 13.5			
				Exploration completed at 13.5 feet.			

Depth scale: 5, 10, 15

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TEST PIT NUMBER: TP-6

PROJECT NAME Camas Woods Phase 3
CLIENT HSR Capital LLC
PROJECT NO. HSR-4-01-1 **LOGGED BY** S. Chandra
PROJECT LOCATION Camas, Washington
CONTRACTOR L&S Contracting LLC
EQUIPMENT CAT307E2
CAVING Minor from 12 to 13 feet.
DATE COMPLETED 12/31/2024
GROUNDWATER Moderate seepage at 12 feet
TIME STARTED 8:15 AM **TIME COMPLETED** 11:13 AM

DEPTH (ft)	SAMPLE ID	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	POCKET PEN (tsf)	MOISTURE CONTENT (%)	FINES (%)	REMARKS
0.0–0.5				Medium stiff, brown sandy SILT, trace organics, moist (6 inches of topsoil, 3-inch-thick root zone). 0.5				
0.5–4.0	TP6.1		SC	Medium dense, brown clayey SAND, trace gravel, moist, sand is fine to coarse. 4.0				
	TP6.2					30	43	Infiltration test at 3 feet.
4.0–13.0			GM	Medium dense, brown silty GRAVEL with sand and cobbles, moist, gravel is fine to coarse, sand is fine, cobbles are subrounded and up to 12 inches in diameter.				
5	TP6.3					24	14	Infiltration test at 6 feet.
10								
	TP6.4			Wet at 12 feet. 13.0				Minor caving from 12 to 13 feet.
15				Exploration completed at 13 feet.				

APPENDIX B

APPENDIX B
LABORATORY TESTING

GENERAL
Laboratory testing was conducted on select soil samples to confirm field classifications and determine the index engineering properties. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications. The locations of the tested samples are shown on the exploration logs. Descriptions of the tests are presented below, and results of the testing are presented in this appendix.

MOISTURE CONTENT
The natural moisture content of select soil samples was determined in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to dry soil in a test sample and is expressed as a percentage.

PARTICLE-SIZE ANALYSIS
Particle-size analysis was performed on a select soil sample 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 was also performed on select soil samples in general accordance with ASTM D1140 (P200). This test is a quantitative determination of the percent passing the U.S. Standard No. 200 sieve by dry weight.

ATTERBERG LIMITS TESTING
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.

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MOISTURE CONTENT, PERCENT PASSING NO. 200 SIEVE BY WASHING

PROJECT	CLIENT	PROJECT NO.	
Camas Woods Phase 3 26514 and 26416 SE 8th Street Camas, Washington	HSR Capital LLC 500 East Broadway, Suite 120 Vancouver, WA 98660	HSR-4-01-1	
		ISSUE DATE: 01/21/25	PAGE: 1 of 1
		DATE SAMPLED: 12/31/24	SAMPLED BY: S. Chandra

LABORATORY TEST DATA

TEST PROCEDURE
ASTM D2216 - Method A, ASTM D1140

LAB ID	CONTAINER MASS (g)	MOIST MASS + CONTAINER (g)	DRY MASS + CONTAINER (g)	AFTER WASH DRY MASS + CONTAINER (g)	FIELD ID	SAMPLE DEPTH (ft)	PERCENT MOISTURE CONTENT	PERCENT PASSING NO. 200 SIEVE
S25-0073	777.12	3,002.92	2,505.70	1,907.74	TP1.2	3	29%	35%
S25-0074	772.20	4,644.63	3,912.20	3,259.20	TP1.3	6	23%	21%
S25-0075	579.10	4,256.49	3,550.51	sieved sample	TP2.1	2	24%	25%
S25-0076	780.04	4,041.64	3,338.44	2,519.96	TP3.2	3	27%	32%
S25-0077	787.34	5,353.00	4,449.88	3,714.65	TP3.3	6	25%	20%
S25-0078	771.52	2,452.58	2,069.49	1,514.03	TP6.2	3	30%	43%
S25-0079	752.77	4,771.61	3,983.45	3,517.80	TP6.3	6	24%	14%

NOTES:
Sample weights received for Lab ID: S25-0073, 0074, 0075, 0076, 0077, 0078, and 0079 did not meet the minimum size requirements; entire sample used for analysis.

DATE TESTED	TESTED BY
01/13/25	M. Scherette

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	
Camas Woods Phase 3 26514 and 26416 SE 8th Street Camas, Washington	HSR Capital LLC 500 East Broadway, Suite 120 Vancouver, WA 98660	HSR-4-01-1	
		ISSUE DATE 01/21/25	PAGE 1 of 1
		LAB ID S25-0073	FIELD ID TP1.2
		DATE SAMPLED 12/31/24	SAMPLED BY S. Chandra

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty GRAVEL with Sand	Test Pit TP-1 depth = 3 feet	no data provided

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318 - Method A

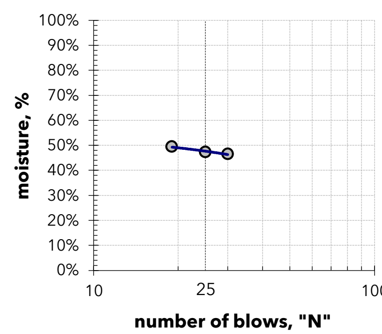
ATTERBERG LIMITS

liquid limit = 48
plastic limit = 30
plasticity index = 18

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.98	32.61	32.76	
dry soil + pan weight, g =	29.11	28.79	28.78	
pan weight, g =	20.79	20.71	20.73	
N (blows) =	30	25	19	
moisture, % =	46.5 %	47.3 %	49.4 %	

LIQUID LIMIT



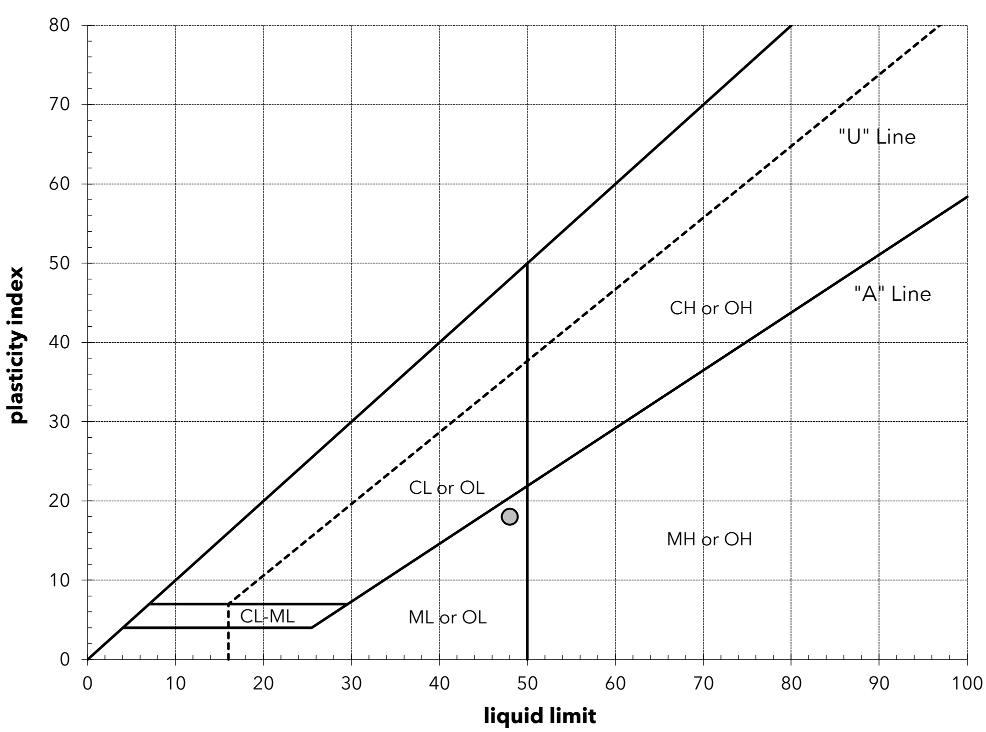
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.05	27.97		
dry soil + pan weight, g =	26.40	26.32		
pan weight, g =	20.95	20.94		
moisture, % =	30.3 %	30.7 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = n/a
% sand = n/a
% silt and clay = n/a
% silt = n/a
% clay = n/a
moisture content = 29%

DATE TESTED	TESTED BY
01/15/25	B. Taylor

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	
Camas Woods Phase 3 26514 and 26416 SE 8th Street Camas, Washington	HSR Capital LLC 500 East Broadway, Suite 120 Vancouver, WA 98660	HSR-4-01-1	
		ISSUE DATE: 01/21/25	PAGE: 1 of 2
		LAB ID: S25-0075	FIELD ID: TP2.1
		DATE SAMPLED: 12/31/24	SAMPLED BY: S. Chandra

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty GRAVEL with Sand	Test Pit TP-2 depth = 2 feet	GM, Silty Gravel with Sand

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-2-7(1)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter, air-dried prep, hand washed, composite sieve - #4 split	ASTM D6913, Method A

ADDITIONAL DATA

initial dry mass (g) =	2971.41		
as-received moisture content =	24%	coefficient of curvature, C_C =	n/a
liquid limit =	53	coefficient of uniformity, C_U =	n/a
plastic limit =	31	effective size, $D_{(10)}$ =	n/a
plasticity index =	22	$D_{(30)}$ =	0.138 mm
fineness modulus =	n/a	$D_{(60)}$ =	7.692 mm

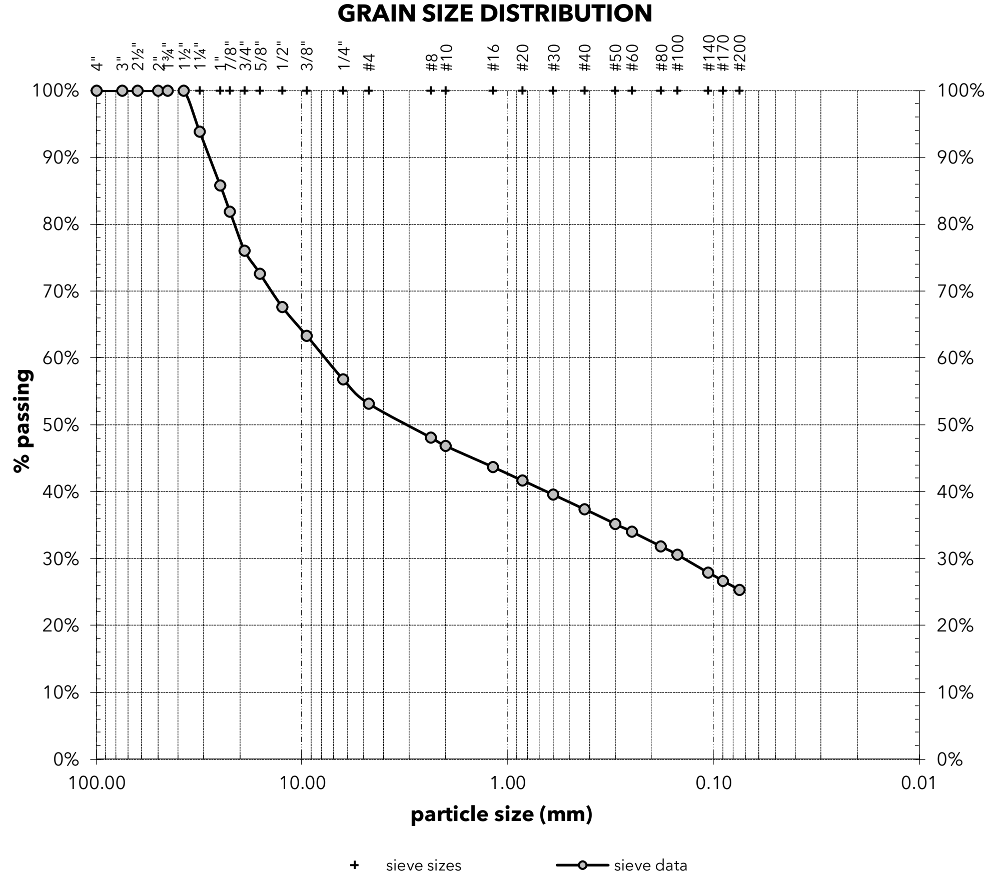
NOTE: Entire sample used for analysis; did not meet minimum size required.

SIEVE DATA

% gravel = 46.8%
% sand = 27.8%
% silt and clay = 25.3%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5	100%			
	1.25"	31.5		94%		
	1.00"	25.0	86%			
	7/8"	22.4		82%		
	3/4"	19.0	76%			
	5/8"	16.0		73%		
	1/2"	12.5	68%			
	3/8"	9.50	63%			
	1/4"	6.30	57%			
	#4	4.75	53%			
SAND	#8	2.36		48%		
	#10	2.00	47%			
	#16	1.18		44%		
	#20	0.850	42%			
	#30	0.600		40%		
	#40	0.425	37%			
	#50	0.300		35%		
	#60	0.250	34%			
	#80	0.180		32%		
	#100	0.150	31%			
	#140	0.106		28%		
	#170	0.090		27%		
	#200	0.075	25%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
01/15/25	M. Scherette

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	
Camas Woods Phase 3 26514 and 26416 SE 8th Street Camas, Washington	HSR Capital LLC 500 East Broadway, Suite 120 Vancouver, WA 98660	HSR-4-01-1	
		ISSUE DATE 01/21/25	PAGE 2 of 2
		LAB ID S25-0075	FIELD ID TP2.1
		DATE SAMPLED 12/31/24	SAMPLED BY S. Chandra

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty GRAVEL with Sand	Test Pit TP-2 depth = 2 feet	GM, Silty Gravel with Sand

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318 - Method A

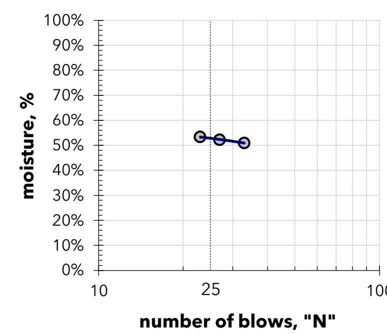
ATTERBERG LIMITS

liquid limit = 53
plastic limit = 31
plasticity index = 22

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.89	32.65	33.50	
dry soil + pan weight, g =	28.13	28.57	28.91	
pan weight, g =	20.75	20.76	20.31	
N (blows) =	33	27	23	
moisture, % =	51.0 %	52.2 %	53.4 %	

LIQUID LIMIT



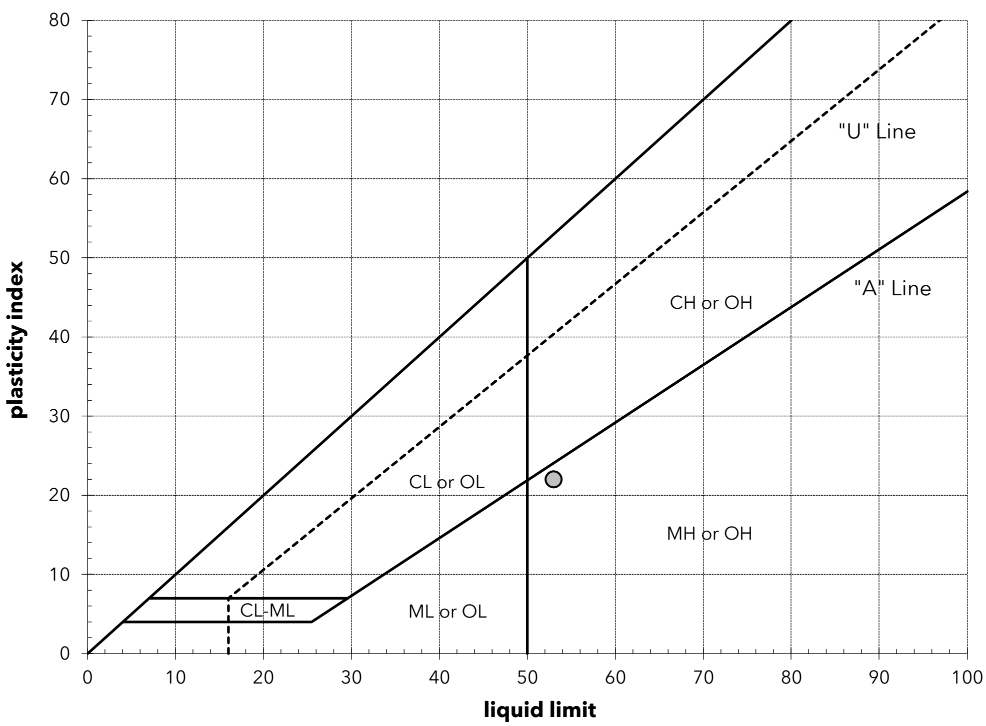
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.99	29.51		
dry soil + pan weight, g =	26.32	27.47		
pan weight, g =	20.93	20.92		
moisture, % =	31.0 %	31.2 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 46.8%
% sand = 27.8%
% silt and clay = 25.3%
% silt = n/a
% clay = n/a
moisture content = 24%

DATE TESTED	TESTED BY
01/16/25	B. Taylor

COLUMBIA WEST ENGINEERING, INC. authorized signature

APPENDIX C

APPENDIX C
PHOTO LOG

Photographs of the site are presented in this appendix.

Central portion of the site. Photograph taken facing south.

Test pit TP-1 profile.

Test pit TP-2 profile.

Test pit TP-3 profile.

Test pit TP-4 profile.

Test pit TP-5 profile.

Test pit TP-6 profile.

APPENDIX D

Geotechnical • Environmental • Special Inspections
Columbia West
Engineering, Inc

APPENDIX D
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 Observation

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 construction observation services 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. The 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

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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.