Geotechnical Engineering Report

Peterson Subdivision

La Center, Washington

July 31, 2023

Geotechnical ■ Environmental ■ Special Inspections
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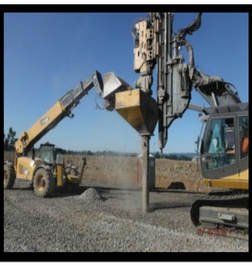


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GEOTECHNICAL ENGINEERING REPORT
PETERSON SUBDIVISION
LA CENTER, WASHINGTON

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Site Location:

34214 NW Pacific Highway
Parcel Nos. 258766000 & 258631000
La Center, Washington

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GEOTECHNICAL ENGINEERING REPORT
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1.0 INTRODUCTION

This report presents the results of our geotechnical engineering evaluation for the proposed Peterson Subdivision project located in La Center, Washington. The purpose of the evaluation is to provide geotechnical engineering recommendations for use in design and construction of the proposed development.

The approximately 5.5-acre site is located southeast of the intersection of NW Pacific Highway and NW Larson Drive. The site is shown relative to surrounding physical features on Figure 1. Exploration locations are shown on Figure 2. Subsurface exploration logs are presented in Appendix A. Laboratory test results are presented in Appendix B. Soil classification information is presented in Appendix C. A photo log is presented in Appendix D. This report is subject to the limitations expressed in Section 10.0, *Limitations*, and Appendix E.

2.0 PROJECT UNDERSTANDING

We understand that the site is planned for residential development with single-family building lots, paved public roadways, underground utilities, and stormwater management facilities. Foundation loads were not available at the time this report was prepared; however, we have assumed maximum column and wall loads of 50 kips and 3 kips per foot, respectively. We expect that floor loads will be less than 100 psf. Cuts and fills are expected to be up to 10 feet each. We should be contacted to revise our recommendations if the assumptions stated above are incorrect.

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 have completed the following tasks:

- Reviewed information available in our files from previous geological and geotechnical studies conducted at and in the vicinity of the site.
- Excavate nine test pits at the site to a maximum depth of 14 feet BGS.
- Collected disturbed soil samples from the borings and hand-auger explorations for laboratory analysis.
- Classified and logged observed soil and groundwater conditions.
- Prepared this geotechnical engineering report that provides our findings, conclusions, and recommendations with regard to:
    - Subsurface soil and groundwater conditions.

- Assessment and mitigation of geologically-hazardous areas in accordance with *La Center Municipal Code, Section 18.300*, *Critical Areas*.
- Settlement considerations.
- Site preparation, 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.
- Foundation support for proposed residential structures.
- Slab subgrade preparation and modulus of subgrade reaction.
- Recommendations for use in design of conventional retaining walls, including backfill and drainage requirements and lateral earth pressures
- Management of groundwater conditions that may affect the performance of structures or pavement.
- Pavement and public roadway construction.
- Seismic design parameters in accordance with *ASCE 7-16.*

4.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

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

Geologic mapping shows that the site is underlain by Missoula Flood Deposits (Ma et al, 2012). From about 21,000 to 12,000 years ago, dozens of gigantic floods swept down the Columbia River and through the Portland/Vancouver area as a huge lake in Montana broke through the glacier that dammed it. The floodwaters reached an elevation of about 400 feet above sea level and scoured many areas down to bedrock, while burying other areas under thick layers of gravel, sand, and silt.

Fine-grained flood deposits consist of sand and silt that were deposited in a series of distinct layers, a few inches to a few feet thick, each of which represents a single flood. These deposits fill most of the northern Willamette Valley, the entire Tualatin Valley, and large areas of Portland and Vancouver. Coarse-grained flood deposits consist mainly of huge overlapping sheets of loose gravel that extend from the mouth of the Columbia River Gorge at Troutdale all the way to the Willamette River.

Underlying the flood deposits is the Pliocene- to Pleistocene-aged Troutdale Formation, which consists of poorly- to moderately-consolidated, semi-cemented, subrounded to rounded sand and gravel conglomerate. The Troutdale Formation is underlain by the Miocene- to Pliocene-aged Sandy River Mudstone and the Miocene-aged Columbia River

Basalt Group (CRBG), which is a series of basalt flows that originated from southeastern Washington and northeastern Oregon.

5.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

Subsurface conditions were explored by excavating nine test pits (TP-1 through TP-9) using a track-mounted excavator at the approximate locations shown on Figure 2. The test pits were excavated on June 30, 2023 to a maximum depth of 14 feet BGS. Subsurface conditions were logged in accordance with the Unified Soil Classification System (USCS). Disturbed soil samples were collected at representative depth intervals. Test pit logs are presented in Appendix A. Analytical laboratory test results are presented in Appendix B. Soil descriptions and classification information are provided in Appendix C.

5.1 Surface Investigation and Site Description

The site is located at 34214 NW Pacific Highway in La Center, Washington and consists of tax parcel 258631000 and the southern portion of parcel 258766000 which totals approximately 5.5 acres. The site is bound by NW Pacific Highway to the north, NW Larson Drive to the west, a forested drainage ravine to the east, and a utility easement to the south. The site is currently undeveloped and utilized for agricultural purposes. No buildings were observed in the proposed development area which is primarily vegetated with grass. Most site terrain is relatively flat to gently rolling and characterized by grades of 5 to 10 percent. Steeper, densely-forested ravine slopes were observed along the eastern site boundary, discussion of which is presented later in Section 6.2, *Slope and Landslide Hazard Areas*.

5.2 Subsurface Conditions

The test pits were excavated through grass surface and a 3- to 4-inch-thick root zone. A low-organic till zone extended to approximately 18 inches. Underlying the surface vegetation, fine-grained alluvial deposits and sedimentary conglomerate were encountered to the maximum explored depth of 14 feet BGS. Subsurface lithology may generally be described by the soil units identified in the following text.

5.2.1 Fine-Grained Alluvium

Underlying the surface vegetation, stiff to very stiff clay and silt with varying proportions of fine sand was observed to depths of 9 to 14 feet BGS. Moisture content of the alluvium ranged from 20 to 27 percent at the time of exploration. Atterberg limits analysis indicates that the alluvium exhibits low to medium plasticity behavior.

5.2.2 Sedimentary Conglomerate

Underlying the fine-grained alluvium, sedimentary conglomerate of dense to very dense sand and gravel with varying proportions of silt and clay was observed to the maximum explored depth of 14 feet BGS. Moisture content of the conglomerate was approximately 22 percent at the time of exploration. Atterberg limits analysis indicates that the fine-textured constituents exhibit medium plasticity behavior.

5.2.3 Groundwater

Perched groundwater was observed at a depth of 13 feet BGS in test pits TP-5 and TP-8. Note that groundwater levels are subject to seasonal variance and may rise during extended periods of increased precipitation. Perched groundwater is typical in the La Center area, generally present near the surface during the wet season and dropping below depths of 10 to 15 feet in the dry season.

6.0 GEOLOGIC HAZARDS

City of La Center Municipal Code, Section 18.300 defines geologic hazard requirements for proposed development in areas subject to City of La Center jurisdiction. Three potential geologic hazards are identified: (1) erosion hazard areas, (2) slope and landslide hazard areas, and (3) seismic hazard areas. According to *Clark County Maps Online*, ravine slopes located along the eastern site boundary are mapped as potential erosion, slope, and landslide hazard areas.

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

6.1 Erosion Hazard Areas

According to *Clark County Maps Online*, the *Soil Survey of Clark County, Washington,* and field observations, the erosion hazard for site soils ranges from slight to severe depending upon slope grade. Therefore, according to the *City of La Center Municipal Code,* a soil erosion hazard area is present at the site. However, the soil erosion hazard can be successfully mitigated by preparation and adherence to a site-specific erosion control plan that identifies BMPs to reduce potential impacts on site soils during construction. Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Erosion control measures are discussed further in Section 8.7, *Erosion Control Measures*.

6.2 Slope and Landslide Hazard Areas

According to *City of La Center Municipal Code,* critical areas associated with slopes and landslide hazards are defined respectively as slopes with gradients meeting or exceeding 25 percent and areas subject to risk of mass movement due to a combination of geologic, topographic, and hydrologic factors.

Columbia West conducted review of available mapping, *Clark County GIS* data, and site reconnaissance to evaluate the potential presence of critical areas associated with slopes and landslide hazards on or near the subject site.

6.2.1 Geologic Literature Review

Columbia West reviewed *Slope Stability of Clark County* (Washington Department of Natural Resources, Division of Geology and Earth Resources, Fiksdal, 1975) to assess site slope characteristics. The Fiksdal report identifies four levels of potential slope instability within Clark County: (1) stable areas – no slides or unstable slopes, (2) areas of potential instability because of underlying geologic conditions and physical characteristics associated with steepness, (3) areas of historical or still active landslides, and (4) older landslide debris. The site is largely mapped as (1) stable areas – no slides or unstable slopes. The drainage ravine located along the eastern site boundary is mapped as (2) areas of potential instability because of underlying geologic conditions and physical characteristics associated with steepness.

Columbia West also reviewed the *Geologic Map of the Ridgefield Quadrangle, Clark County, Washington* (R.C. Evarts, Washington Division of Geology and Earth Resources, Scientific Investigations Map 2844, 2004), which indicates that no landslide deposits are mapped at the subject site or in the surrounding vicinity.

6.2.2 Slope Reconnaissance

To observe geomorphic conditions, Columbia West conducted visual and physical reconnaissance of the drainage ravine slopes located along the eastern property boundary. As previously described, test pit explorations conducted near the slopes indicated the presence of stiff to very stiff clay underlain by dense sedimentary conglomerate. No landslide debris was observed within subsurface soils explored near the slopes.

Review of topographic mapping indicates that vertical slope heights for the eastern drainage ravine slopes (east facing), as measured from toe to top-of-slope break, vary from approximately 40 to 50 feet. Slope grades generally range from 25 to 65 percent with localized steeper areas. Slopes currently support dense vegetation consisting of deciduous and conifer trees, blackberry vines, grasses, and shrubs. Slopes are generally planar with no observed evidence of instability. There was no observed direct evidence of large-scale, mass slope movements or historic landslides.

6.2.3 Slope Stability Assessment

Based upon the results of literature review, subsurface exploration, and field reconnaissance, Columbia West did not observe a combination of geologic, topographic, or hydrologic features suggesting significant risk of mass slope movement. However, slope grades along the eastern drainage ravine meet or exceed 25 percent in several locations and therefore meet the definition of a critical area according to *City of La Center Municipal Code*. The location of the critical area is indicated on Figure 2. Site development near the critical area may be successfully achieved by following the engineering and planning recommendations presented in this report and by maintaining an appropriate geotechnical buffer from the top-of-slope as presented in the following text sections.

6.2.4 Geotechnical Buffer

To reduce the risk of adverse impacts to slope stability within and near the critical area, residential structures, structural fill placement, and stormwater facility construction should be avoided within the geotechnical buffer identified on Figure 2, unless a case-by-case assessment as described in Section 6.2.6 is conducted. The buffer recommendations are intended to reduce potential for slope instability by limiting locations for large dynamic and static loads derived from earthwork, residential structures, retaining walls, roadways, and other significant developments. The geotechnical buffer line is based upon the slope reconnaissance and slope stability assessment described above and may be measured as 30 feet from the eastern ravine's existing top-of-slope.

Note that areas within the geotechnical buffer are not intended to be do-not-disturb conservation areas. Small disturbances such as minor landscaping, fence building, or pedestrian path construction are acceptable provided that the increased risk of soil sloughing and settlement within the buffer is understood. Deep-rooted vegetation generally results in reduced slope erosion and increased near-surface soil shear strength. The risk of slope instability increases with disturbance or alteration of existing slope vegetation. Removal of established slope vegetation within the buffer should be minimized. The text herein pertains only to the geotechnical aspect of construction within the recommended geotechnical buffer.

6.2.5 Grading Recommendations within the Geotechnical Buffer

The geotechnical buffer is intended to minimize adverse impacts to slope stability due to dynamic and static loading. Placement of engineered structural fill or stockpiles of disturbed soil should be avoided inside the geotechnical buffer without case-by-case evaluation per Section 6.2.6, *Potential Encroachment within the Geotechnical Buffer*. Soil excavation may be acceptable within the buffer, as driving forces may be reduced by removing soil mass. Columbia West should review mass grading plans as they relate to the geotechnical buffer.

6.2.6 Potential Encroachment within the Geotechnical Buffer

Encroachment of some site improvements or structural facilities inside the geotechnical buffer may be possible if evaluated in detail on a case-by-case basis. Feasibility of such encroachment will depend upon dimensions, locations, and specific design features of the proposed improvement. Often these data are not available until later in the design process. Encroachment within the geotechnical buffer area should be contingent upon a supplemental geotechnical investigation. The investigation should include additional exploratory activities and data analysis to develop appropriate design recommendations. Quantification of risk of slope instability and specialized design recommendations, if applicable or necessary, should be included.

6.3 Seismic Hazard Areas

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

6.3.1 Liquefaction

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand with low silt and clay content is the most susceptible to liquefaction. Low plasticity silty sand and silt may also be susceptible to seismic settlement during a seismic event under relatively higher levels of ground shaking; however, the magnitude of settlement at the ground surface is less than liquefaction settlement. Based on laboratory testing, liquefiable materials were not observed at the site to the depth explored. Accordingly, liquefaction is not a design consideration.

6.3.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard that occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. Since the site is not susceptible liquefaction, lateral spreading at the site is not a design consideration.

6.3.3 Ground Shaking Amplification

Review of the *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), indicates that site soils may be represented by Site Class C as defined by *ASCE 7-16, Chapter 20, Table 20.3-1*. 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 constitute a geologic hazard in our opinion, and will not prohibit development if properly accounted for during the design process.

6.3.4 Fault Rupture

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

7.0 DESIGN RECOMMENDATIONS

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

7.1 Shallow Foundation Support

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

Any loose or disturbed soil should be improved or removed and replaced with structural fill. If footing subgrade soils are above their optimum moisture content, we recommend that a minimum of 6 inches of compacted aggregate be placed over exposed subgrade soils. The aggregate pad should extend 6 inches beyond the edge of the foundations and consist of imported granular material as described in Section 8.6.1, *Structural Fill*. Columbia West should observe exposed subgrade conditions prior to placement of crushed aggregate to verify adequate subgrade support.

7.1.1 Bearing Capacity

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

Footings bearing on subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,000 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.

7.1.2 Shallow Foundation Settlement

Foundation settlement is a significant structural design consideration. Provided subgrade soils are prepared as described above and in Section 8.1, *Site Preparation and Grading*, we anticipate that post-construction static foundation settlement will be less than approximately 1 inch. Differential settlement between comparably-loaded foundations is not expected to exceed approximately 0.5 inch over a distance of 50 feet.

7.1.3 Resistance to Sliding

Lateral foundation loads can be resisted by passive earth pressure on the sides of the footing and by friction at the base of the footings. Recommended passive earth pressure for footings confined by native soil or engineered structural fill is 350 pcf. The upper 12 inches of soil should be neglected when calculating passive pressure resistance. Adjacent floor slabs and pavement, if present, should also be neglected from the analysis. The recommended passive pressure resistance assumes that a minimum horizontal clearance of 10 feet is maintained between the footing face and adjacent downgradient slopes.

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

7.1.4 Subgrade Observation

Subgrade should be evaluated by Columbia West prior to placing forms or reinforcing steel to verify subgrade support conditions are as described in this report. Subgrade observation should confirm that all undocumented fill, disturbed material, organic debris, remnant topsoil zones, and softened subgrades (if present) have been removed. Over-excavation of footing subgrade soils may be required to remove deleterious material, particularly if footings are constructed during wet-weather conditions.

7.1.5 Floor Slabs

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

To provide a capillary break, slabs should be underlain by at least 6 inches of compacted crushed aggregate that contains less than 5 percent by weight passing the No. 200 Sieve. Geotextile may be used below the crushed aggregate layer to increase subgrade support. Recommendations for floor slab base aggregate and subgrade geotextile are discussed in Section 8.6, *Materials*.

Floor slabs with maximum floor load of 100 psf may be designed assuming a modulus of subgrade reaction, k, of 125 pci.

7.2 Seismic Design Considerations

Seismic design for proposed structures is prescribed by *ASCE 7-16.* Based on literature review and results of subsurface exploration conducted by Columbia West, site soils meet the criteria for Site Class C. Seismic design parameters for Site Class C are presented in Table 1.

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

	Short Period	1 Second Period
MCE Spectral Acceleration	0.805	0.380
Site Class	C	
Site Coefficient	Fa = 1.2	Fv = 1.5
Adjusted Spectral Response Acceleration	S_{MS} = 0.966	S_{M1} = 0.570
Design Spectral Response Acceleration	S_{DS} = 0.644	S_{D1} = 0.380

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

As discussed in Section 6.3, *Seismic Hazards*, liquefaction and lateral spreading are not design considerations for the site.

7.3 Retaining Structures

Lateral earth pressures should be considered during design of retaining walls and below-grade structures. Hydrostatic pressure and additional surcharge loading should also be considered. Wall foundation construction and bearing capacity should adhere to specifications provided previously in Section 7.1, *Shallow Foundation Support*.

Permanent retaining walls that are not restrained from rotation should be designed for active earth pressures using an equivalent fluid pressure of 35 pcf. Walls that are restrained from rotation should be designed for an at-rest, equivalent fluid pressure of 55 pcf. The recommended earth pressures assume a maximum wall height of 10 feet with well-drained, level backfill. These values also assume that adequate drainage is provided behind retaining walls to prevent hydrostatic pressures from developing. Lateral earth pressures induced by surcharge loads may be estimated using the criteria presented on Figure 3.

Seismic forces may be calculated by superimposing a uniform lateral force of $7H^2$ 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.

7.3.1 Wall Drainage and Backfill

A minimum 6-inch-diameter, perforated collector pipe should be placed at the base of retaining walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of finished grade. The drain rock and geotextile drainage fabric should meet the specifications provided in Section 8.6, *Materials*. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drainage systems, unless measures are taken to prevent backflow into the drainage system of the wall.

Backfill material placed behind the walls and extending a horizontal distance of ½ H, where H is the height of the retaining wall, should consist of select granular material placed and compacted as described in Section 8.6.1, *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.

7.4 Pavement Recommendations

We understand that public roadways for the subdivision will be constructed in accordance with City of La Center standards. For dry weather construction, pavement surface sections should bear upon competent subgrade consisting of scarified and compacted native soil or engineered structural fill. Wet weather construction may require an increased thickness of base aggregate as discussed later in Section 8.2, *Construction Traffic and Staging*.

In general, AC paving is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress. Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix, as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. In Oregon, the AC surface temperature during paving should be at least 40 degrees Fahrenheit for lift thicknesses greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thicknesses between 2 and 2.5 inches.

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

7.5 Drainage

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

Recommendations for foundation drains and subdrains are presented in the following sections. Drain rock and geotextile drainage fabric should meet the requirements presented in Section 8.6, *Materials*. Drains should be closely monitored after construction to assess their effectiveness. If additional surface or shallow subsurface seeps become evident, the drainage provisions may require modification or additional drains. We should be consulted to provide appropriate recommendations.

7.5.1 Foundation Drains

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

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

7.5.2 Subdrains

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

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Site Preparation and Grading

Site vegetation primarily consisted of grass and a 3- to 4-inch-thick root zone at the time of our exploration. Thicker root zones may be present in areas of mature trees and shrub growth. Pavement, vegetation, organic material, unsuitable fill, and deleterious material should be cleared from areas identified for structures and site grading. Vegetation, root zones, organic material, and debris should be removed from the site. Stripped topsoil should also be removed, or used only as landscape fill in nonstructural areas with slopes less than 25 percent. The post-construction maximum depth of landscape fill placed or spread at any location onsite should not exceed one foot. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed.

8.1.1 Subgrade Evaluation

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

8.2 Construction Traffic and Staging

Near-surface clay will be easily disturbed during construction. If not carefully executed, site preparation, excavation, and grading can create extensive soft areas resulting in significant repair costs. Earthwork planning should include considerations for minimizing subgrade disturbance, particularly during wet-weather conditions.

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

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

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

8.3 Cut and Fill Slopes

Fill slopes should consist of structural fill material as discussed in Section 8.6.1, *Structural Fill*. Fill placed on existing grades steeper than 5H:1V should be horizontally benched at least 10 feet into the slope. Fill slopes greater than six feet in height should be vertically keyed into existing subsurface soil. A typical fill slope cross-section is shown in Figure 6. Drainage implementations, including subdrains or perforated drainpipe trenches, may also be necessary in proximity to cut and fill slopes if seeps or springs are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by Columbia West during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion.

Final cut or fill slopes at the site should not exceed 2H:1V or 10 feet in height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 7. Slope buffer recommendations for the eastern drainage ravine slopes were provided previously in Section 6.2.4, *Geotechnical Buffer*.

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

8.4 Excavation

The site was explored to a maximum depth of 14 feet BGS with an excavator. Conventional earthmoving equipment in proper working condition should be capable of making necessary site excavations.

Perched groundwater was observed at a depth of 13 feet BGS in test pits TP-5 and TP-8. Recommendations as described in Section 8.5, Dewatering, should be considered where subsurface construction activities intersect the shallow groundwater table.

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

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

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

8.5 Dewatering

Perched groundwater was observed as shallow as 13 feet BGS at the time of our field exploration. Based on this observation, groundwater may be encountered in utility trench excavations and in areas of cut. Generalized recommendations for temporary construction dewatering are presented in the following section.

8.5.1 Construction Dewatering

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

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

If groundwater is present at the base of utility excavations, we recommend placing 18 to 24 inches of stabilization material at the base of the excavation. Subgrade geotextile placed directly over trench subgrade soils may reduce the required thickness of the stabilization material. The actual thickness of stabilization material should be determined at the time of construction based on observed field conditions. Trench stabilization material should be placed in one lift and compacted until well keyed. Stabilization material and geotextile fabric should meet the requirements presented in Section 8.6, *Materials*.

8.6 Materials

8.6.1 Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in Section 8.1, *Site Preparation and Grading*. Engineered fill placement should be observed by Columbia West. Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with *ASTM D6938*. Field compaction testing should be performed for each vertical foot of engineered fill placed.

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

8.6.1.1 Onsite Soil

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

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

8.6.1.2 Imported Granular Material

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

8.6.1.3 Stabilization Material

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

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

8.6.1.4 Trench Backfill

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

Within structural areas (below pavement and building pads), trench backfill above the pipe zone should consist of *WSDOT 9-03.19 Bank Run Gravel for Trench Backfill* or *WSDOT 9-03.14(2) Select Borrow* with a maximum particle size of 2 ½-inches. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). Remaining trench backfill should be compacted to at least 95 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557),* or as required by the local jurisdictional agency or pipe manufacturer*.*

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

8.6.1.5 Floor Slab Base Aggregate

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

8.6.2 Pavement Base Aggregate

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

8.6.2.1 Retaining Wall Backfill

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

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

8.6.2.2 Retaining Wall Leveling Pad

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

8.6.2.3 Drain Rock

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

8.6.3 Geotextile Fabric

8.6.3.1 Subgrade Geotextile

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

8.6.3.2 Drainage Geotextile

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

8.7 Erosion Control Measures

Soil at this site is susceptible to erosion by wind and water; therefore, erosion control measures should be carefully planned and installed before construction begins. Surface water runoff should be collected and directed away from sloped areas to prevent water from running down the slope face. Measures that can be employed to reduce erosion include the use of silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads. All erosion control methods should be in accordance with local jurisdiction standards.

9.0 OBSERVATION OF CONSTRUCTION

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

10.0 LIMITATIONS

We have prepared this report for use by the client and members of the design and construction team for the proposed development. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Soil explorations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were not finalized at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

♦ ♦ ♦

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,

COLUMBIA WEST ENGINEERING, Inc.

Greg L. Williamson, P.E.
Senior Geotechnical Engineer

Brett A. Shipton, P.E., G.E.
Principal

07-31-23

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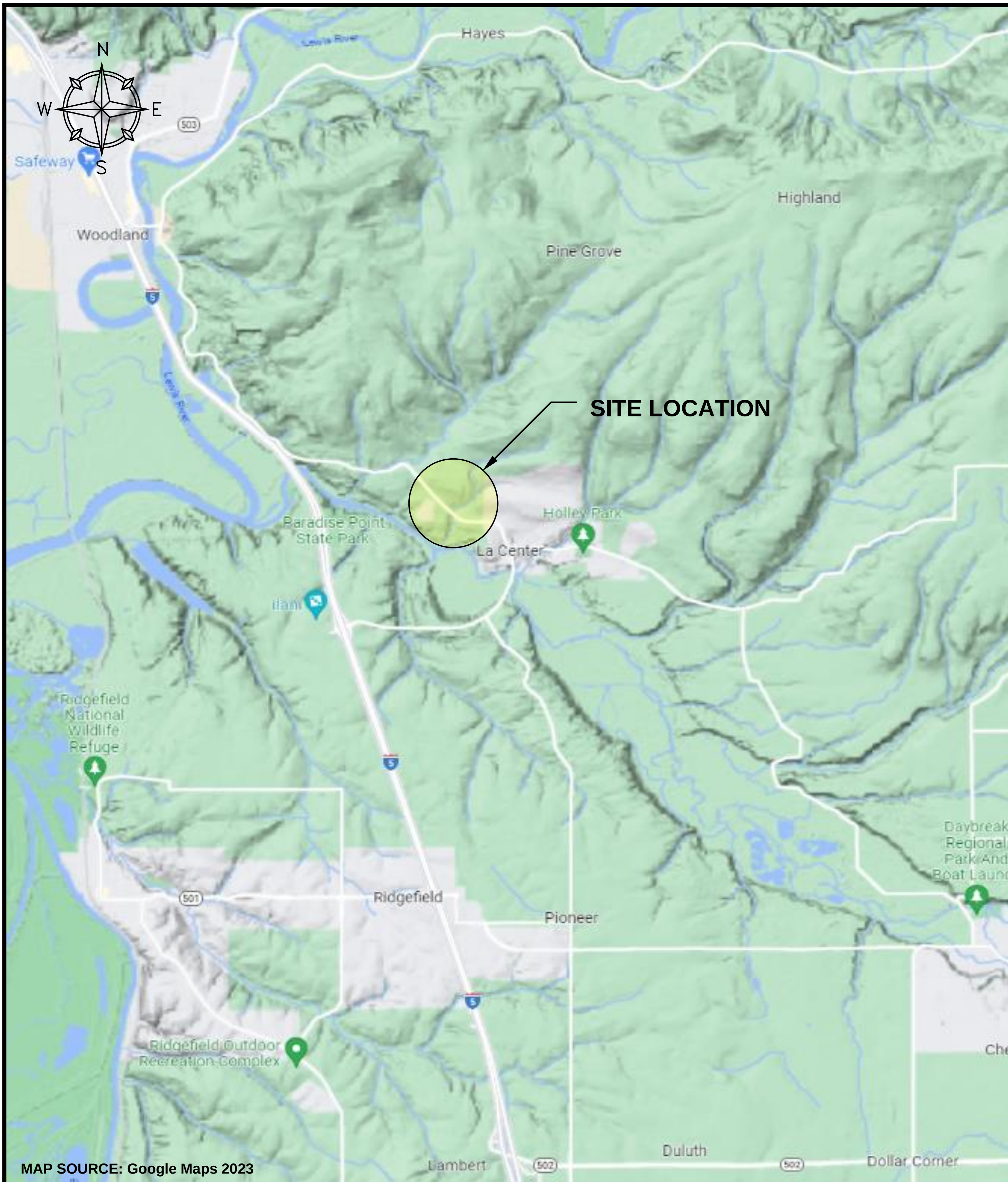
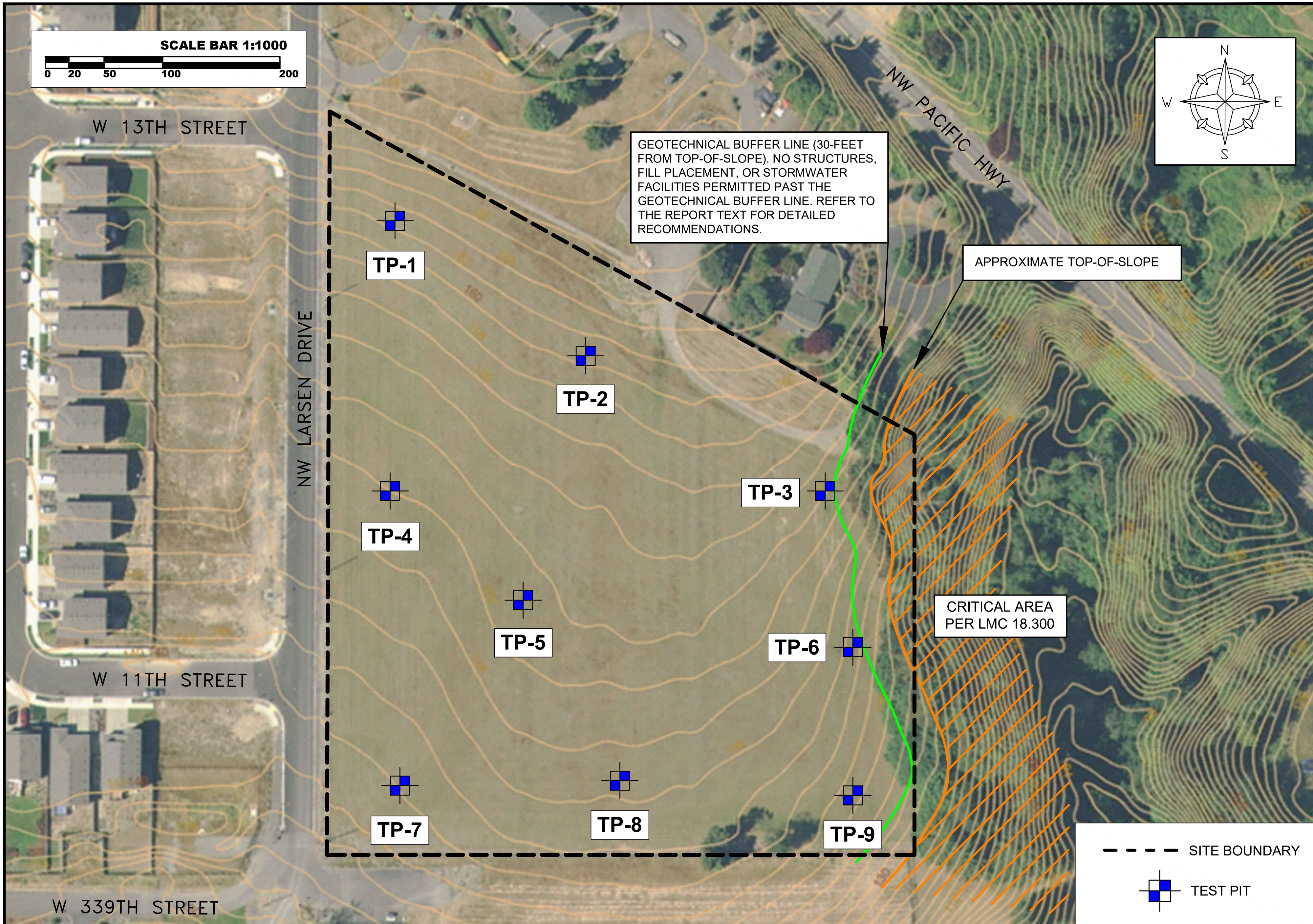
FIGURES

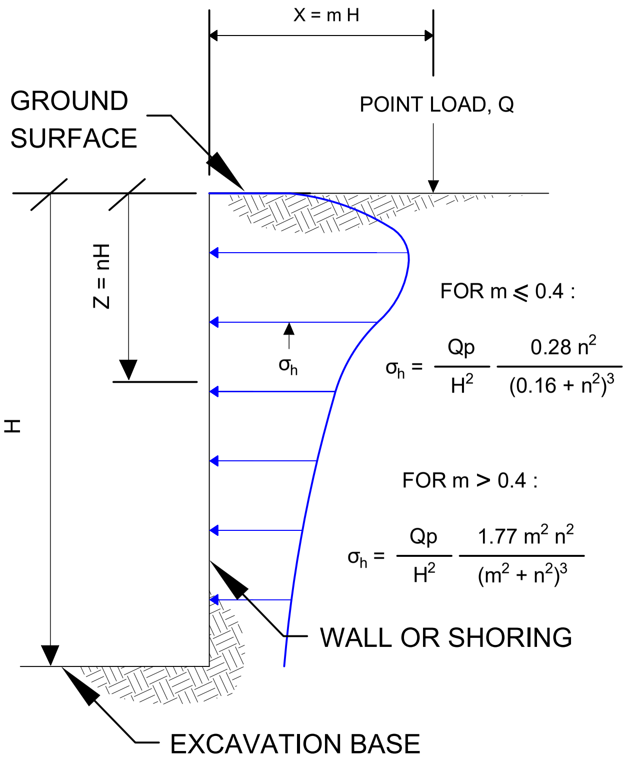


Geotechnical ▪ Environmental ▪ Special Inspections **Columbia West** Engineering, Inc	Job No: 23264 Date: 06/26/23 Drawn: GLW Checked: BAS	SITE LOCATION MAP PETERSON SUBDIVISION LA CENTER, WASHINGTON	FIGURE 1

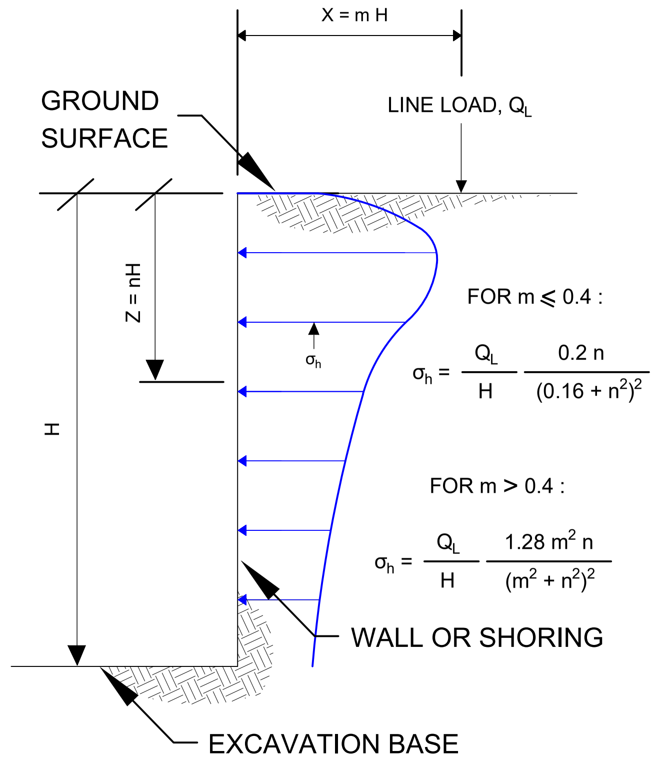


Geotechnical ▪ Environmental ▪ Special Inspections Columbia West Engineering, Inc	Project: 23264 Date: 06/20/23 Drawn: GLW Checked: BAS	EXPLORATION LOCATION MAP PETERSON SUBDIVISION	NOTES: 1. SITE LOCATION: 34214 NW PACIFIC HWY, LA CENTER, WASHINGTON. 2. SITE CONSISTS OF PORTIONS OF TAX PARCEL NOS. 258766000 AND 258631000 TOTALING APPROXIMATELY 5.5 ACRES. 3. BASE MAP OBTAINED FROM CLARK COUNTY MAPS ONLINE, 2023. 4. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.	FIGURE 2

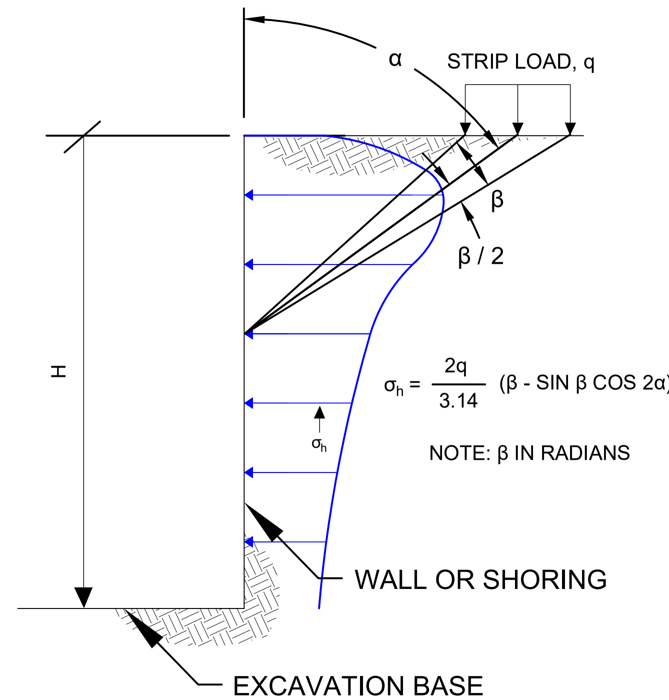
VERTICAL POINT LOAD



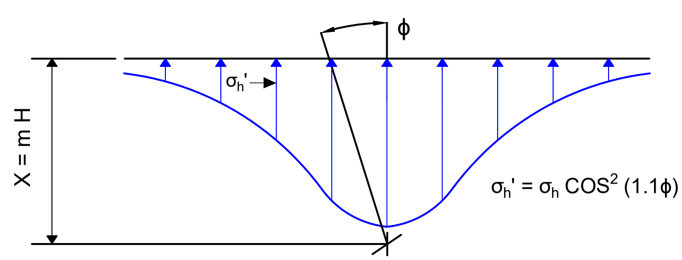
LINE LOAD PARALLEL TO WALL



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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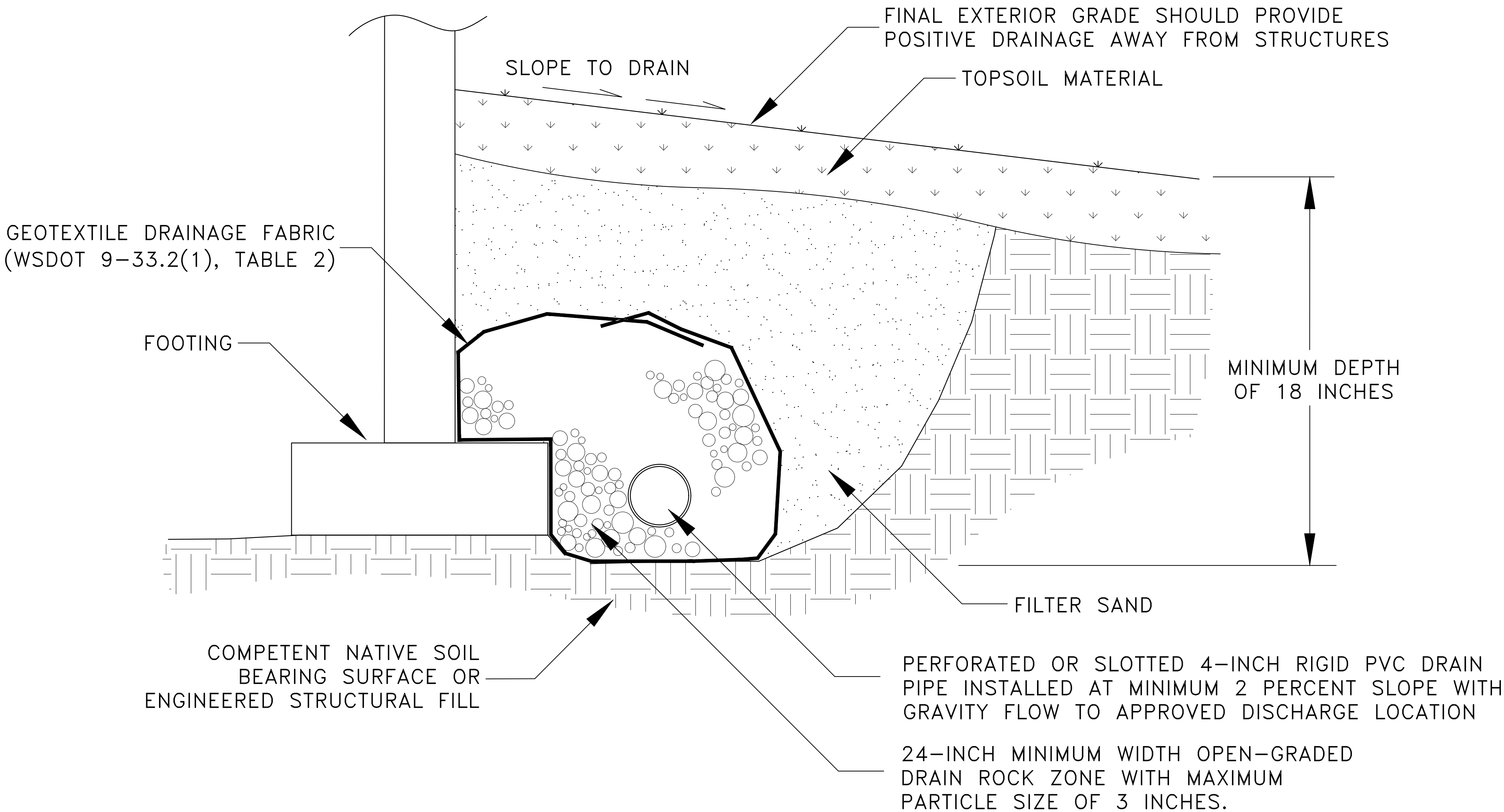
SURCHARGE–INDUCED LATERAL EARTH PRESSURES

FIGURE 3



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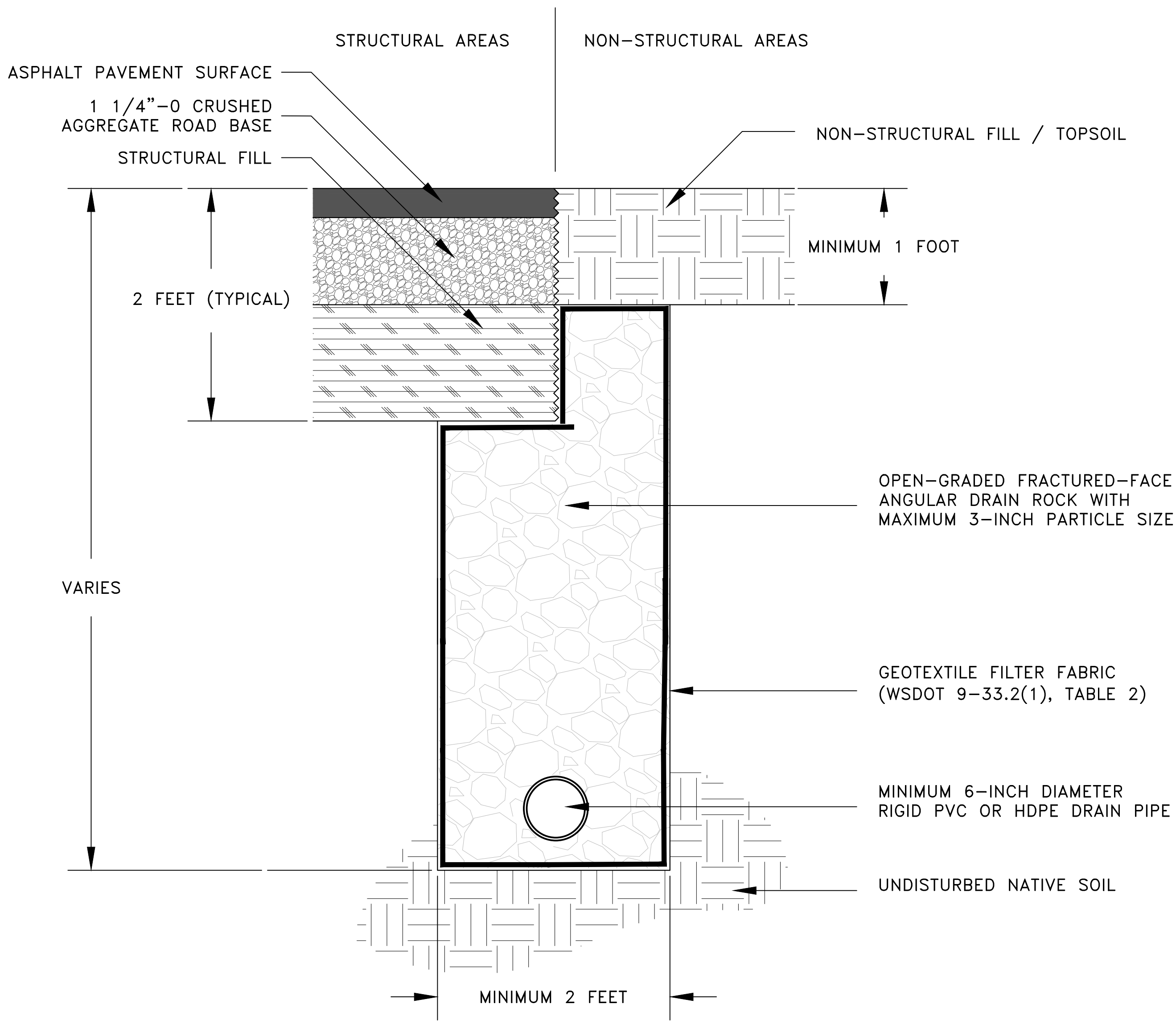
TYPICAL PERIMETER FOOTING DRAIN DETAIL

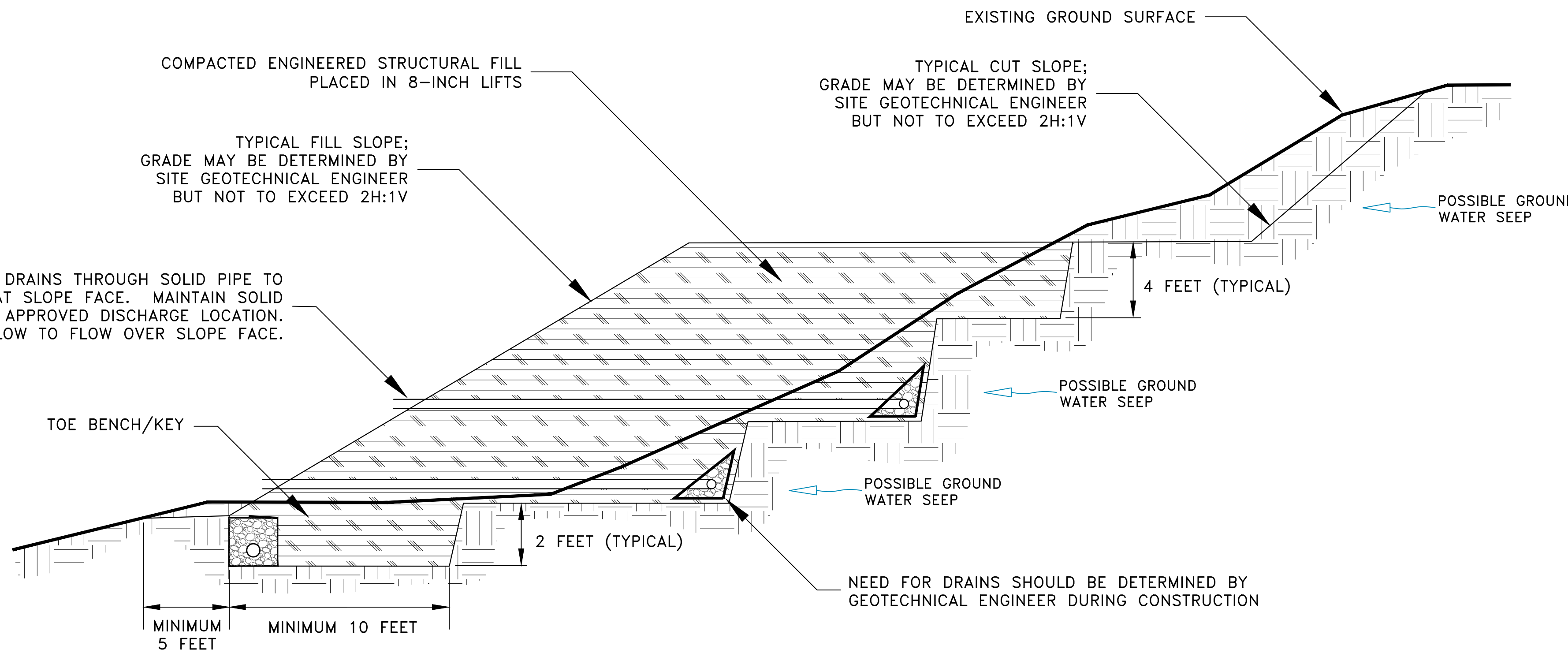
NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL PERIMETER FOOTING DRAIN DETAIL AND MAY NOT BE SITE-SPECIFIC.

FIGURE 4

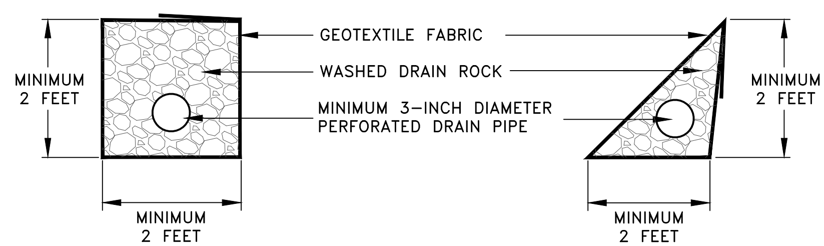


NOTE: LOCATION, INVERT ELEVATION, DEPTH OF TRENCH, AND EXTENT OF PERFORATED PIPE REQUIRED MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER DURING CONSTRUCTION BASED UPON FIELD OBSERVATION AND SITE–SPECIFIC SOIL CONDITIONS.

Geotechnical ▪ Environmental ▪ Special Inspections Columbia West Engineering, Inc	TYPICAL PERFORATED DRAINPIPE TRENCH DETAIL	NOTES: 1. DRAWING IS NOT TO SCALE. 2. DRAWING REPRESENTS TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL AND MAY NOT BE SITE–SPECIFIC.	FIGURE 5



TYPICAL DRAIN SECTION DETAIL



DRAIN SPECIFICATIONS

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

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

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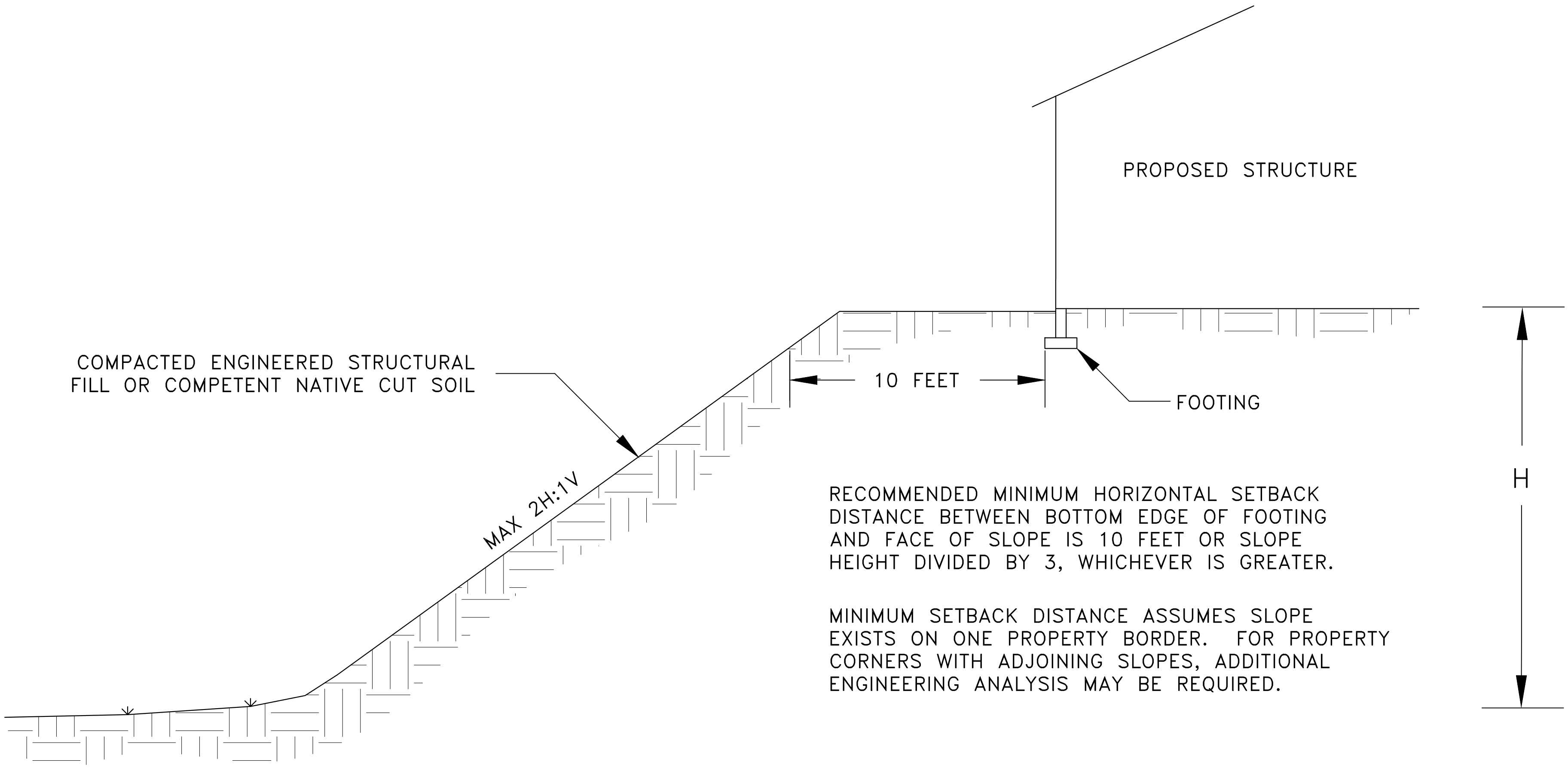
TYPICAL CUT AND FILL SLOPE CROSS SECTION

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL CUT AND FILL SLOPE CROSS SECTION AND MAY NOT BE SITE-SPECIFIC.

FIGURE 6



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TYPICAL MINIMUM FOUNDATION SLOPE SETBACK DETAIL

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

FIGURE 7

APPENDIX A
SUBSURFACE EXPLORATION PROGRAM
FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by excavating nine test pits (TP-1 through TP-9) to a maximum depth of 14 feet BGS using a track-mounted excavator. Excavation services were provided by L&S Contractors, Inc. of Battle Ground, Washington on June 30, 2023. The test pit locations are shown on Figure 2. Exploration logs are presented in this appendix.

SOIL SAMPLING

Representative grab samples of soil from the test pit explorations were obtained from the walls and/or base of the test pits using the excavator bucket. Sampling intervals are shown on the exploration logs.

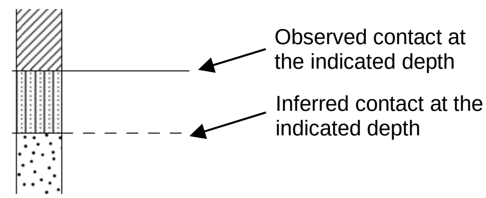
SOIL CLASSIFICATION

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

EXPLORATION LEGEND

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

Graphical Log of Subsurface Lithology



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

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Peterson Subdivision	MJS Investors		23264	TP-1
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Excavating	Excavator	MAC	6/30/23
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	Not Surveyed	See Page Notes	1245	1330

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Grass and 3- to 4-inch-thick root zone. Fine rootlets and low-organic till zone extends to approximately 18 inches.					
		Gee silt loam		CL		Brown to brown/orange/gray lean CLAY with sand, moist, stiff to very stiff, low to medium plasticity.					
5											
						Increased fine sand and decreased plasticity with depth.					
10											
						Bottom of test pit at 14.0 feet. No groundwater observed on 6/30/23.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Peterson Subdivision	MJS Investors		23264	TP-2
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Excavating	Excavator	MAC	6/30/23
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	Not Surveyed	See Page Notes	0800	0845

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Grass and 3- to 4-inch-thick root zone. Fine rootlets and low-organic till zone extends to approximately 18 inches.					
		Gee silt loam		CL		Brown to brown/gray, mottled lean CLAY with sand, moist, stiff to very stiff, low to medium plasticity.					
	TP-2.1		A-6(7)				27.0	71.7	35	11	
5											
						Increased fine sand and decreased plasticity with depth.					
	TP-2.2						27.0	56			
10											
				GC		Weathered brown to black sedimentary CONGLOMERATE of semi-consolidated clayey gravel with sand, moist, dense to very dense, medium plasticity fines.					
	TP-2.3										
						Bottom of test pit at 14.0 feet. No groundwater observed on 6/30/23.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Peterson Subdivision	MJS Investors		23264	TP-3
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Excavating	Excavator	MAC	6/30/23
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	Not Surveyed	See Page Notes	1115	1150

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Grass and 3- to 4-inch-thick root zone. Fine rootlets and low-organic till zone extends to approximately 18 inches.					
		Gee silt loam		CL		Brown to brown/gray, mottled lean CLAY with sand, moist, stiff to very stiff, low to medium plasticity.					
5											
						Increased fine sand and decreased plasticity with depth.					
				GC		Weathered brown to black sedimentary CONGLOMERATE of semi-consolidated clayey gravel with sand, moist, dense to very dense, medium plasticity fines.					
10											
						Bottom of test pit at 13.0 feet. No groundwater observed on 6/30/23.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Peterson Subdivision	MJS Investors		23264	TP-4
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Excavating	Excavator	MAC	6/30/23
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	Not Surveyed	See Page Notes	1220	1245

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Grass and 3- to 4-inch-thick root zone. Fine rootlets and low-organic till zone extends to approximately 18 inches.					
		Gee silt loam		CL		Brown to brown/orange/gray mottled lean CLAY with sand, moist, stiff to very stiff, low to medium plasticity.					
5											
10											
	TP-13.1						27.0	74			
						Bottom of test pit at 14.0 feet. No groundwater observed on 6/30/23.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Peterson Subdivision	MJS Investors		23264	TP-5
PROJECT LOCATION: La Center, Washington	CONTRACTOR: L&S Excavating	EQUIPMENT: Excavator	ENGINEER: MAC	DATE: 6/30/23
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: Not Surveyed	GROUNDWATER DEPTH: See Page Notes	START TIME: 0845	FINISH TIME: 0930

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Grass and 3- to 4-inch-thick root zone. Fine rootlets and low-organic till zone extends to approximately 18 inches.					
		Gee silt loam		CL		Brown to brown/gray mottled lean CLAY with sand, moist, stiff to very stiff, low to medium plasticity.					
5											
	TP-5.1										
10				SC		Weathered brown to black sedimentary CONGLOMERATE of semi-consolidated clayey sand with trace gravel, moist to wet, dense to very dense, medium plasticity fines.					
	TP-5.2										
						Bottom of test pit at 14.0 feet. Groundwater seep observed at 13.0 feet bgs.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Peterson Subdivision	MJS Investors		23264	TP-6
PROJECT LOCATION La Center, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER MAC	DATE 6/30/23
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION Not Surveyed	GROUNDWATER DEPTH See Page Notes	START TIME 1040	FINISH TIME 1115

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Grass and 3- to 4-inch-thick root zone. Fine rootlets and low-organic till zone extends to approximately 18 inches.					
		Gee silt loam		CL		Brown to brown/gray mottled lean CLAY with sand, moist, stiff to very stiff, low to medium plasticity.					
5											
10				GC		Weathered brown to black sedimentary CONGLOMERATE of semi-consolidated clayey gravel with sand, moist, dense to very dense, medium plasticity fines.					
	TP-6.1		A-2-7(1)				22.0	24.1	48	21	
						Bottom of test pit at 13.0 feet. No groundwater observed on 6/30/23.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Peterson Subdivision	MJS Investors		23264	TP-7
PROJECT LOCATION: La Center, Washington	CONTRACTOR: L&S Excavating	EQUIPMENT: Excavator	ENGINEER: MAC	DATE: 6/30/23
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: Not Surveyed	GROUNDWATER DEPTH: See Page Notes	START TIME: 1155	FINISH TIME: 1220

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Grass and 3- to 4-inch-thick root zone. Fine rootlets and low-organic till zone extends to approximately 18 inches.					
		Gee silt loam		CL		Brown to brown/orange/gray mottled lean CLAY with sand, moist, stiff to very stiff, low to medium plasticity.					
5											
10											
						Bottom of test pit at 14.0 feet. No groundwater observed on 6/30/23.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Peterson Subdivision	MJS Investors		23264	TP-8
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Excavating	Excavator	MAC	6/30/23
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	Not Surveyed	See Page Notes	0930	1010

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Grass and 3- to 4-inch-thick root zone. Fine rootlets and low-organic till zone extends to approximately 18 inches.					
	TP-8.1	Gee silt loam		CL		Brown to brown/gray mottled lean CLAY with sand, moist, stiff to very stiff, low to medium plasticity. Increased fine sand and decreased plasticity with depth.	20.0	66			
5											
10											
	TP-8.2			SC		Weathered brown to black sedimentary CONGLOMERATE of semi-consolidated clayey sand with trace gravel, moist to wet, dense to very dense, medium plasticity fines.					
15						Bottom of test pit at 14.0 feet. Groundwater seep observed at 13.0 feet bgs.					
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Peterson Subdivision	MJS Investors		23264	TP-9
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Excavating	Excavator	MAC	6/30/23
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	Not Surveyed	See Page Notes	1010	1040

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Grass and 3- to 4-inch-thick root zone. Fine rootlets and low-organic till zone extends to approximately 18 inches.					
		Gee silt loam		SM		Brown to brown/gray mottled lean CLAY with sand, moist, stiff to very stiff, low to medium plasticity.					
5											
10				GC		Weathered brown to black sedimentary CONGLOMERATE of semi-consolidated clayey gravel with sand, moist, dense to very dense, medium plasticity fines.					
15						Bottom of test pit at 14.0 feet. No groundwater observed on 6/30/23.					
20											

APPENDIX B
LABORATORY TESTING

CLASSIFICATION

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

MOISTURE CONTENT

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

PARTICLE-SIZE ANALYSIS

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

ATTERBERG LIMITS

The plastic and liquid limits (Atterberg limits) of select soil samples were determined in accordance with ASTM D4318. The testing was conducted to classify fine-grained soil in accordance with United Soil Classification System (USCS) specifications. Results of the Atterberg limits analysis are presented in this appendix.

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Peterson Subdivision La Center, Washington	MJS Investors 11201 SE 8th Street, Suite 116 Bellevue, Washington 98004	23264	S23-0835
		REPORT DATE: 07/19/23	FIELD ID: TP2.1
		DATE SAMPLED: 06/30/23	SAMPLED BY: MAC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
brown Lean CLAY with Sand	Test Pit TP-02 depth = 3 feet	CL, Lean Clay with Sand

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-6(7)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter, moist prep, hand washed, 12" single sieve-set	ASTM D6913, Method A

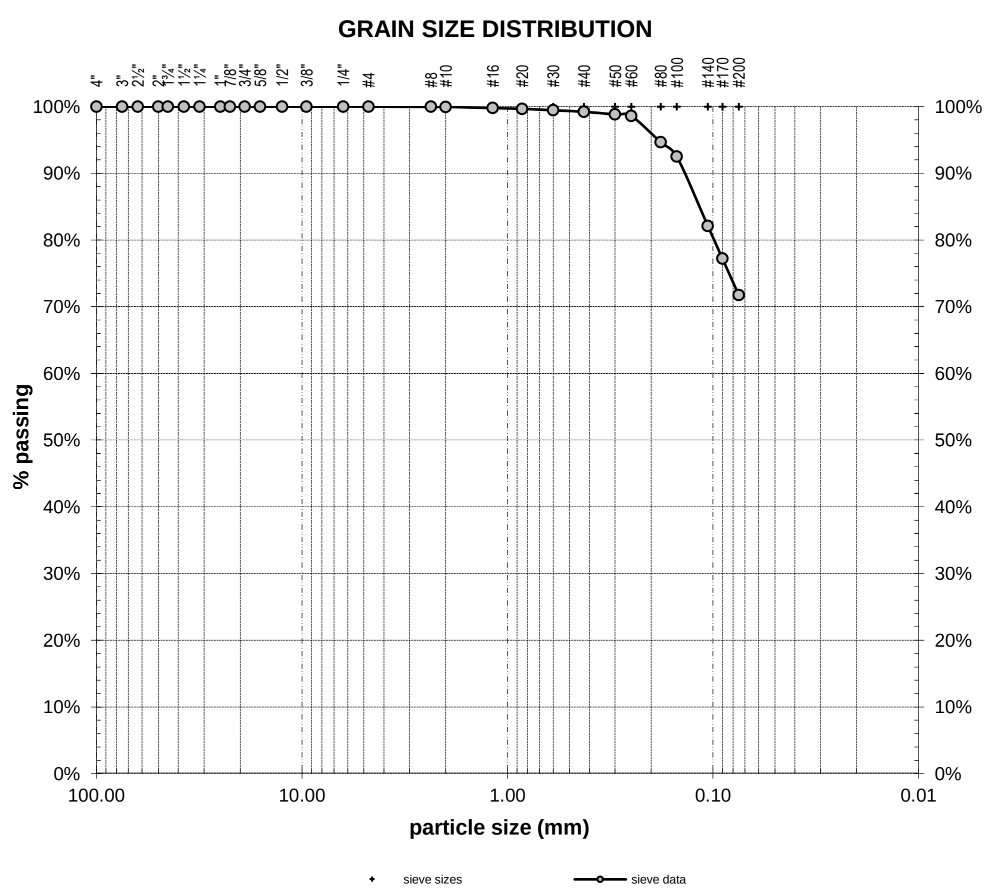
ADDITIONAL DATA

initial dry mass (g) =	151.78		
as-received moisture content =	27%	coefficient of curvature, C_C =	n/a
liquid limit =	35	coefficient of uniformity, C_U =	n/a
plastic limit =	24	effective size, $D_{(10)}$ =	n/a
plasticity index =	11	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 28.3%
% silt and clay = 71.7%

	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		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		100%		
	#20	0.850	100%			
	#30	0.600		99%		
	#40	0.425	99%			
	#50	0.300		99%		
	#60	0.250	99%			
	#80	0.180		95%		
	#100	0.150	92%			
	#140	0.106		82%		
	#170	0.090		77%		
	#200	0.075	72%			



DATE TESTED	TESTED BY
07/14/23	MRS

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Peterson Subdivision La Center, Washington	MJS Investors 11201 SE 8th Street, Suite 116 Bellevue, Washington 98004	23264	S23-0835
		REPORT DATE: 07/19/23	FIELD ID: TP2.1
		DATE SAMPLED: 06/30/23	SAMPLED BY: MAC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
brown Lean CLAY with Sand	Test Pit TP-02 depth = 3 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

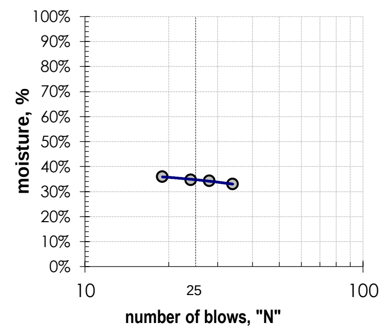
ATTERBERG LIMITS

liquid limit =	35
plastic limit =	24
plasticity index =	11

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	35.08	33.71	34.91	33.89
dry soil + pan weight, g =	31.58	30.46	31.33	30.43
pan weight, g =	20.98	20.97	21.02	20.82
N (blows) =	34	28	24	19
moisture, % =	33.0 %	34.3 %	34.7 %	36.0 %

LIQUID LIMIT



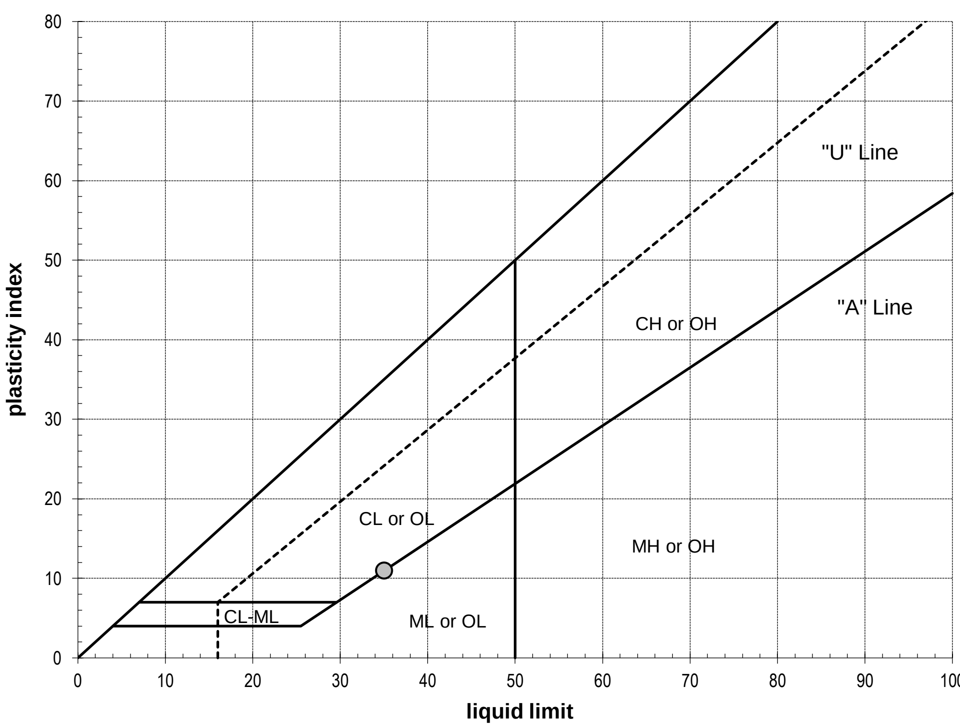
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.37	28.12		
dry soil + pan weight, g =	26.94	26.76		
pan weight, g =	20.87	21.02		
moisture, % =	23.6 %	23.7 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	28.3%
% silt and clay =	71.7%
% silt =	n/a
% clay =	n/a
moisture content =	27%

DATE TESTED	TESTED BY
07/17/23	MRS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Peterson Subdivision La Center, Washington	MJS Investors 11201 SE 8th Street, Suite 116 Bellevue, Washington 98004	23264	S23-0838
		REPORT DATE: 07/19/23	FIELD ID: TP6.1
		DATE SAMPLED: 06/30/23	SAMPLED BY: MAC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
orange/brown/black Clayey GRAVEL with Sand	Test Pit TP-06 depth = 12 feet	GC, Clayey Gravel with Sand
SPECIFICATIONS: none		AASHTO CLASSIFICATION: 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) =	1915.97		
as-received moisture content =	22%	coefficient of curvature, C_C =	n/a
liquid limit =	48	coefficient of uniformity, C_U =	n/a
plastic limit =	27	effective size, $D_{(10)}$ =	n/a
plasticity index =	21	$D_{(30)}$ =	0.173 mm
fineness modulus =	n/a	$D_{(60)}$ =	23.149 mm

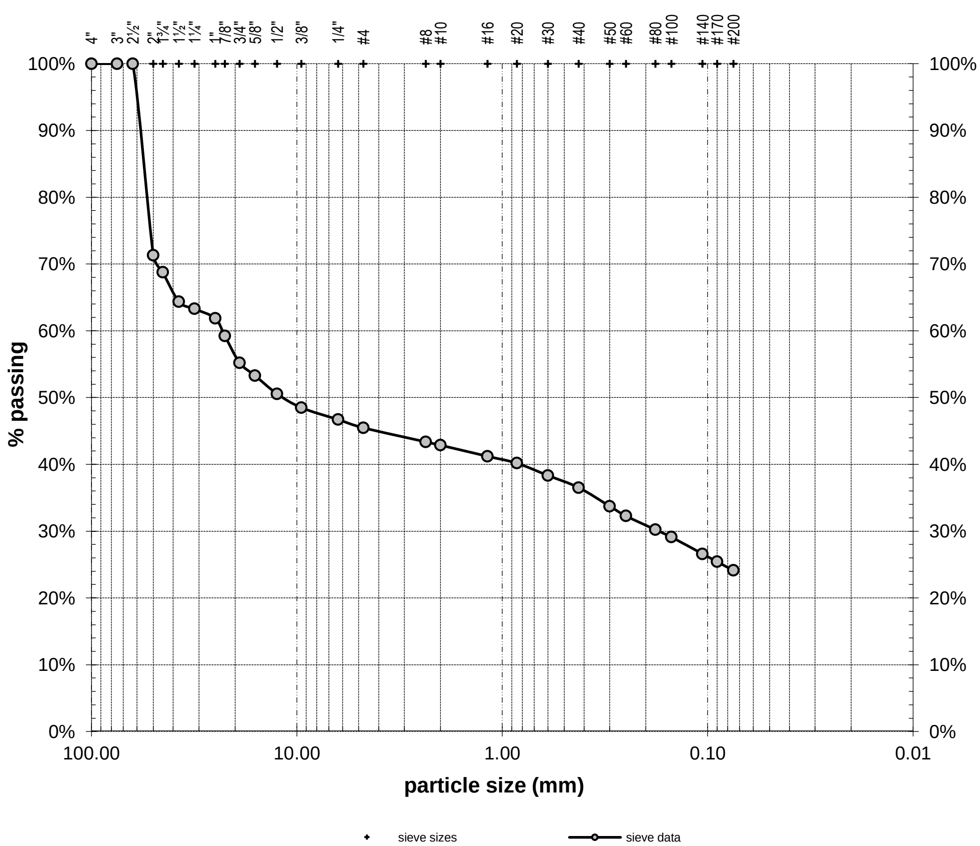
NOTE: Entire sample used for analysis; did not meet minimum size required.

SIEVE DATA

% gravel = 54.5%
% sand = 21.4%
% silt and clay = 24.1%

	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	71%			
	1.75"	45.0		69%		
	1.50"	37.5	64%			
	1.25"	31.5		63%		
	1.00"	25.0	62%			
	7/8"	22.4		59%		
	3/4"	19.0	55%			
	5/8"	16.0		53%		
	1/2"	12.5	51%			
	3/8"	9.50	48%			
	1/4"	6.30		47%		
	#4	4.75	45%			
SAND	#8	2.36		43%		
	#10	2.00	43%			
	#16	1.18		41%		
	#20	0.850	40%			
	#30	0.600		38%		
	#40	0.425	37%			
	#50	0.300		34%		
	#60	0.250	32%			
	#80	0.180		30%		
	#100	0.150	29%			
	#140	0.106		27%		
	#170	0.090		25%		
	#200	0.075	24%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
07/18/23	KMS

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Peterson Subdivision La Center, Washington	MJS Investors 11201 SE 8th Street, Suite 116 Bellevue, Washington 98004	23264	S23-0838
		REPORT DATE 07/19/23	FIELD ID TP6.1
		DATE SAMPLED 06/30/23	SAMPLED BY MAC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
orange/brown/black Clayey GRAVEL with Sand	Test Pit TP-06 depth = 12 feet	GC, Clayey Gravel with Sand

LABORATORY TEST DATA

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

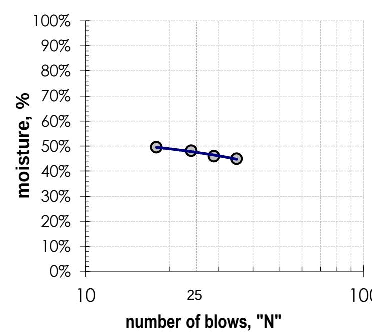
ATTERBERG LIMITS

liquid limit =	48
plastic limit =	27
plasticity index =	21

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.28	31.36	32.91	31.26
dry soil + pan weight, g =	28.61	27.92	29.03	27.72
pan weight, g =	20.44	20.43	20.96	20.56
N (blows) =	35	29	24	18
moisture, % =	44.9 %	45.9 %	48.1 %	49.4 %

LIQUID LIMIT



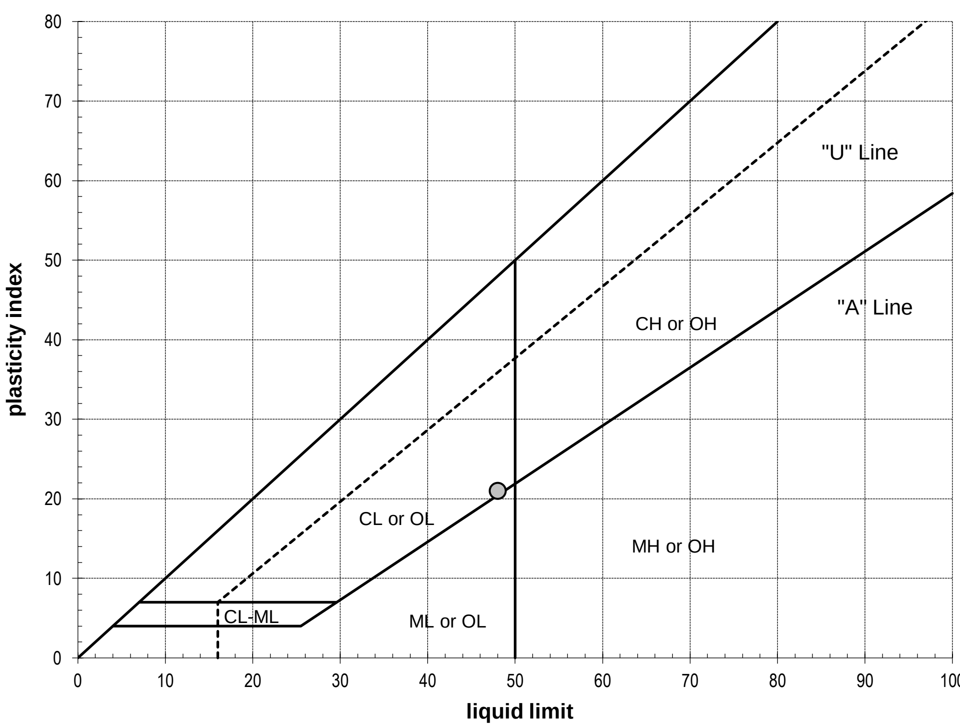
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.68	28.07		
dry soil + pan weight, g =	26.23	26.43		
pan weight, g =	20.93	20.42		
moisture, % =	27.4 %	27.3 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	54.5%
% sand =	21.4%
% silt and clay =	24.1%
% silt =	n/a
% clay =	n/a
moisture content =	22%

DATE TESTED	TESTED BY
07/17/23	MRS

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MOISTURE CONTENT, PERCENT PASSING NO. 200 SIEVE BY WASHING

PROJECT	CLIENT	PROJECT NO.	REPORT DATE
Peterson Subdivision La Center, Washington	MJS Investors 11201 SE 8th Street, Suite 116 Bellevue, Washington 98004	23264	07/19/23
		DATE SAMPLED: 06/30/23	
		SAMPLED BY: MAC	

LABORATORY TEST DATA

TEST PROCEDURE
ASTM D2216 - Method A, ASTM D1140

LAB ID	CONTAINER MASS	MOIST MASS + PAN	DRY MASS + PAN	AFTER WASH DRY MASS + PAN	MATERIAL DESCRIPTION	FIELD ID	SAMPLE DEPTH	MOISTURE CONTENT	PASSING NO. 200 SIEVE
S23-0835	87.20	322.23	272.63	sieved sample	brown Lean CLAY with Sand	TP2.1	3 feet	27%	72%
S23-0836	542.64	832.70	770.44	642.42	brown/gray Sandy SILT	TP2.2	9 feet	27%	56%
S23-0837	541.95	828.20	767.88	599.66	brown/orange/gray CLAY with Sand	TP4.1	13 feet	27%	74%
S23-0838	548.11	2,878.49	2,464.08	sieved sample	brown/orange/black Clayey GRAVEL with Sand	TP6.1	12 feet	22%	24%
S23-0839	556.51	855.50	804.93	640.38	brown Sandy SILT	TP8.1	2 feet	20%	66%

NOTES:
Sample weight received for Lab ID: S23-0838 did not meet the minimum size requirement; entire sample used for analysis.

DATE TESTED	TESTED BY
07/18/23	MRS/BTT

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APPENDIX C
SOIL AND ROCK CLASSIFICATION INFORMATION

SOIL DESCRIPTION AND CLASSIFICATION

Particle-Size Classification

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

Consistency for Cohesive Soil

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

Relative Density for Granular Soil

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

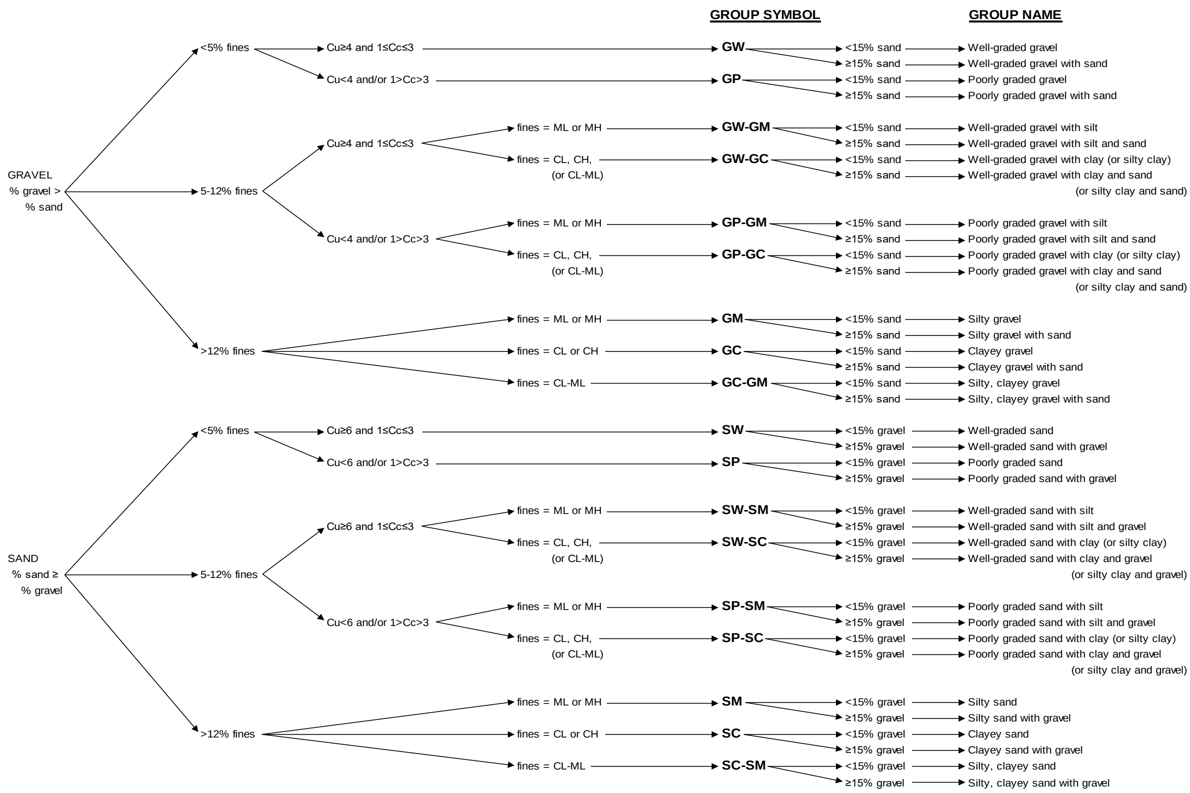
Moisture Designations

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

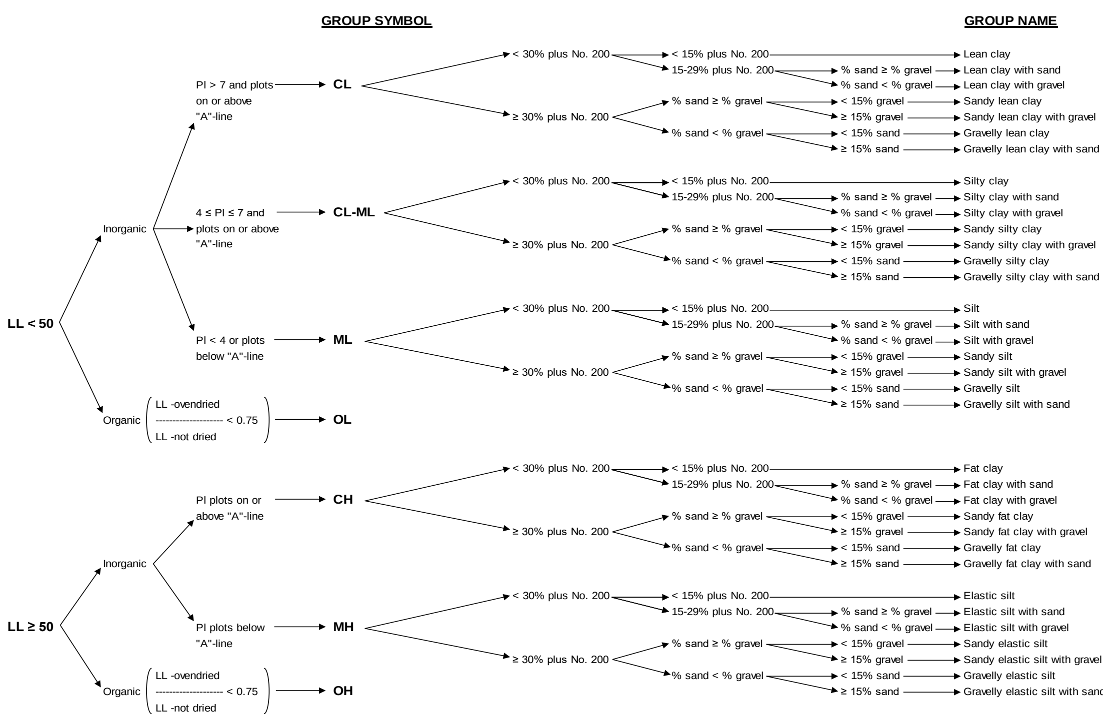
Additional Constituents

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

UNIFIED SOIL CLASSIFICATION SYSTEM



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



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

ROCK CLASSIFICATION SYSTEM

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

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

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

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

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

QUALITY	RQD (%)
Very poor	Less than 25%
Poor	25 to 50%
Fair	51 to 75%
Good	76 to 90%
Excellent	91 to 100%

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

APPENDIX D
PHOTO LOG

Peterson Subdivision

June, 2023

La Center, Washington

Test Pit, TP-1

Typical Soil Mottling, Test Pit, TP-1

Peterson Subdivision

June, 2023

La Center, Washington

Test Pit, TP-2

Test Pit, TP-3

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

Peterson Subdivision

June, 2023

La Center, Washington

Test Pit, TP-4

Test Pit, TP-5

Peterson Subdivision

June, 2023

La Center, Washington

Test Pit, TP-6

Test Pit, TP-7

Peterson Subdivision

June, 2023

La Center, Washington

Test Pit, TP-8

Test Pit, TP-9

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

Peterson Subdivision

June, 2023

La Center, Washington

Facing North From South End of Site

East Slope, Facing South

Peterson Subdivision

June, 2023

La Center, Washington

From East Side of Site, Facing West

From North Side of Site, Facing South

Peterson Subdivision

June, 2023

La Center, Washington

West Side of Site, Facing South

East Side of Site, From North Central Area of Site

Peterson Subdivision

June, 2023

La Center, Washington

Southwest Corner of Site

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

Date: July 31, 2023
Project: Peterson Subdivision
La Center, Washington

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

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

Report Conclusions and Preliminary Nature

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

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

Additional Investigation and Construction QA/QC

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

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Actual subsurface conditions are more readily observed and discerned during the earthwork phase of construction when soils are exposed. Columbia West cannot accept responsibility for deviations from recommendations described in this report or future performance of structural facilities if another consultant is retained during the construction phase or Columbia West is not engaged to provide construction observation to the full extent recommended.

Collected Samples

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

Report Contents

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

Report Limitations for Contractors

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

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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.