Geotechnical Site Investigation

La Center Flow Station #1

La Center, Washington

March 8, 2022

Geotechnical ■ Environmental ■ Special Inspections

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GEOTECHNICAL SITE INVESTIGATION
LA CENTER FLOW STATION #1
LA CENTER, WASHINGTON

Prepared For:	**Clark Public Utilities** **Attn: Barry Lovingood** **PO Box 8900** **Vancouver, Washington 98668**
Approximate Site Location:	**32100 NW La Center Road** **Ridgefield, Washington**
Prepared By:	**Columbia West Engineering, Inc.** **11917 NE 95th Street** **Vancouver, Washington 98682** **Phone: 360-823-2900** **W.O. No. 21300**
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1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Clark Public Utilities to conduct a geotechnical site investigation for the proposed Flow Station #1 project located in La Center, Washington. The purpose of the investigation was to observe and assess subsurface soil conditions at specific locations and provide geotechnical engineering analyses, planning, and design recommendations for proposed development. The specific scope of services was outlined in a proposal contract dated December 3, 2021. This report summarizes the investigation and provides field assessment documentation and laboratory analytical test reports. This report is subject to the limitations expressed in Section 7.0, *Conclusion and Limitations*, and Appendix E.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located in the right-of-way bordering the north side of the intersection of NW La Center Road and NW Timmen Road in La Center, Washington. The approximate latitude and longitude are N 45° 51' 12" and W 122° 40' 37", and the legal description is a portion of the SW ¼ of Section 03, T4N, R1E, Willamette Meridian. The current regulatory jurisdictional agency is the City of La Center.

1.2 Proposed Development

Correspondence with the client and review of the preliminary site plan indicates that proposed development includes the construction of a new 10-foot by 16-foot precast concrete building and flow station. Columbia West has not reviewed preliminary grading plans but understands that cut and fill may be proposed at the subject site. This report is based upon proposed development as described above and may not be applicable if modified.

2.0 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.

According to the *Geologic Map of the Ridgefield Quadrangle, Clark and Cowlitz Counties, Washington* (Russell C. Evarts, USGS Geological Survey Scientific Investigation Map 2844, 2004), onsite near-surface soils are primarily mapped as Pleistocene-aged, unconsolidated, rhythmically bedded, periglacial clay, silt, and fine- to medium-textured sand deposits derived from catastrophic outburst floods of Glacial Lake Missoula (Qfs). Minor Holocene-aged, unconsolidated, soil, sand, gravel, and rock artificial fill and modified land deposits (Af) are mapped in the southwest area of the site. Fine-textured flood deposits are underlain by Pleistocene to Pliocene-aged, unconsolidated to cemented, deeply weathered, pebble to boulder sedimentary conglomerate (QTc).

The Web Soil Survey (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2022 Website) identifies surface soils as Gee silt loam. Although soil conditions may vary from the broad USDA descriptions, Gee series soils are generally fine-textured clays and silts with very low permeability, moderate to high water capacity, and low shear strength. Gee soils are generally moisture sensitive, somewhat compressible, and described as having low to moderate shrink-swell potential. The erosion hazard is slight primarily based upon slope grade.

3.0 REGIONAL SEISMOLOGY

Recent research and subsurface mapping investigations within the Pacific Northwest appear to suggest the historic potential risk for a large earthquake event with strong localized ground movement may be underestimated. Past earthquakes in the Pacific Northwest appear to have caused landslides and ground subsidence, in addition to severe flooding near coastal areas. Earthquakes may also induce soil liquefaction, which occurs when elevated horizontal ground acceleration and velocity cause soil particles to interact as a fluid as opposed to a solid. Liquefaction of soil can result in lateral spreading and temporary loss of bearing capacity and shear strength.

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

Portland Hills Fault Zone

The Portland Hills Fault Zone consists of several northwest-trending faults located along the northeastern margin of the Tualatin Mountains, also known as the Portland Hills, and the southwest margin of the Portland Basin. The fault zone is approximately 25 to 30 miles in length and is located approximately 12 miles southwest of the site. According to *Seismic Design Mapping, State of Oregon* (Geomatrix Consultants, 1995), there is no definitive consensus among geologists as to the zone fault type. Several alternate interpretations have been suggested.

According to the *USGS Earthquake Hazards Program*, the fault was originally mapped as a down-to-the-northeast normal fault but has also been mapped as part of a regional-scale zone of right-lateral, oblique slip faults, and as a steep escarpment caused by asymmetrical folding above a south-west dipping, blind thrust fault. The Portland Hills fault offsets Miocene Columbia River Basalts, and Miocene to Pliocene sedimentary rocks of the Troutdale Formation. No fault scarps on surficial Quaternary deposits have been described along the fault trace, and the fault is mapped as buried by the Pleistocene-aged Missoula flood deposits.

However, evidence suggests that fault movement has impacted shallow Holocene deposits and deeper Pleistocene sediments. Seismologists recorded a M3.2 earthquake thought to be associated with the fault zone near Kelly Point Park in November 2012, a M3.9 earthquake thought to be associated with the fault zone near Kelly Point Park in April 2003, and a M3.5 earthquake possibly associated with the fault zone approximately 1.3 miles east of the fault in 1991. Therefore, the Portland Hills Fault Zone is generally thought to be potentially active and capable of producing possible damaging earthquakes.

Gales Creek-Newberg-Mt. Angel Fault Zone

Located approximately 32 miles southwest of the site, the northwest-striking, approximately 50-mile long Gales Creek-Newberg-Mt. Angel Structural Zone forms the northwestern boundary between the Oregon Coast Range and the Willamette Valley, and consists of a series of discontinuous northwest-trending faults. The southern end of the fault zone forms the southwest margin of the Tualatin basin. Possible late-Quaternary geomorphic surface deformation may exist along the structural zone (Geomatrix Consultants, 1995).

According to the *USGS Earthquake Hazards Program*, the Mount Angel fault is mapped as a high-angle, reverse-oblique fault, which offsets Miocene rocks of the Columbia River Basalts, and Miocene and Pliocene sedimentary rocks. The fault appears to have controlled emplacement of the Frenchman Spring Member of the Wanapum Basalts, and thus must have a history that predates the Miocene age of these rocks. No unequivocal evidence of deformation of Quaternary deposits has been described as a thick sequence of sediments deposited by the Missoula floods covers much of the southern part of the fault trace.

Although no definitive evidence of impacts to Holocene sediments have clearly been identified, the Mount Angel fault appears to have been the location of minor earthquake swarms in 1990 near Woodburn, Oregon, and a M5.6 earthquake in March 1993 near Scotts Mills, approximately four miles south of the mapped extent of the Mt. Angel fault. It is unclear if the earthquake occurred along the fault zone or a parallel structure. Therefore, the Gales Creek-Newberg-Mt. Angel Structural Zone is considered potentially active.

Lacamas Lake-Sandy River Fault Zone

The northwest-trending Lacamas Lake Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 21 miles southeast of the site, and form part of the northeastern margin of the Portland basin. According to *Geology and Groundwater Conditions of Clark County Washington* (USGS Water Supply Paper 1600, Mundorff, 1964) and the *Geologic Map of the Lake Oswego Quadrangle* (Oregon DOGAMI Series GMS-59, 1989), the Lacamas Lake fault zone consists of shear contact between the Troutdale Formation and underlying Oligocene andesite-basalt bedrock. Secondary shear contact associated with the fault zone may have produced a series of prominent northwest-southeast geomorphic lineaments in proximity to the site.

According to the *USGS Earthquake Hazards Program* the fault has been mapped as a normal fault with down-to-the-southwest displacement and has also been described as a steeply northeast or southwest-dipping, oblique, right-lateral, slip-fault. The trace of the Lacamas Lake fault is marked by the very linear lower reach of Lacamas Creek. No fault scarps on Quaternary surficial deposits have been described. The Lacamas Lake fault offsets Pliocene-aged sedimentary conglomerates generally identified as the Troutdale formation, and Pliocene- to Pleistocene-aged basalts generally identified as the Boring Lava formation.

Recent seismic reflection data across the probable trace of the fault under the Columbia River yielded no unequivocal evidence of displacement underlying the Missoula flood deposits, however, recorded mild seismic activity during the recent past indicates this area may be potentially seismogenic.

Cascadia Subduction Zone

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

4.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance and one test pit (TP-1) was conducted at the site on January 3, 2022. The test pit was explored with a track-mounted excavator. The subsurface soil profile was logged in accordance with Unified Soil Classification System (USCS) specifications. Disturbed soil samples were collected from relevant soil horizons and submitted for laboratory analysis. Analytical laboratory test results are presented in Appendix A. The exploration location is indicated on Figure 2. A subsurface exploration log is presented in Appendix B. Soil descriptions and classification information are provided in Appendix C. A photo log is presented in Appendix D.

4.1 Surface Investigation and Site Description

The subject site is located in the right-of-away bordering the north side of the intersection of NW La Center Road and NW Timmen Road in La Center, Washington. Site observations during exploration indicate the site is generally open and vegetated with grass and brush. The site is bounded by NW La Center Road to the south and east and residential development to the north and west. Field reconnaissance and review of site topographic mapping indicate the presence of north- and northwest-facing slopes with grades ranging from 5 to 80 percent located approximately 100 to 200 feet northeast to west of the proposed development area with vertical slope heights of approximately 100 feet as measured from toe to top-of-slope break. Site elevations in the proposed development area range from approximately 156 feet above mean see level (amsl) at the southwest property corner to 145 feet amsl at the northeast property corner with grades ranging from 5 to 15 percent. Slope geometry and geomorphic features are discussed in greater detail in Section 5.2.2, *Slope Reconnaissance and Slope Stability Assessment*.

4.2 Subsurface Exploration and Investigation

The test pit was explored to a maximum depth of approximately 14 feet below ground surface (bgs). The exploration location was selected to observe subsurface soil characteristics in proximity to proposed development areas and is indicated on Figure 2.

4.2.1 Soil Type Description

The field investigation indicated the presence of approximately 2 feet of apparent disturbed fill soils with trace organics in the observed location. Underlying the existing fill layer, subsurface soils resembling geologically mapped outburst floods of Glacial Lake Missoula (Qfs) and native USDA Gee soil series description were encountered. Subsurface lithology may generally be described by soil types identified in the following text. Field logs and observed stratigraphy for the encountered materials are presented in Appendix B, *Subsurface Exploration Logs*.

Soil Type 1 – Existing FILL

Soil Type 1 was observed to primarily consist of light brown to brown/gray, moist, apparent disturbed and re-worked native soils and trace organic debris. Soil Type 1 was observed at the ground surface in TP-1, extending to an apparent depth of approximately two feet bgs.

Soil Type 2 – Lean CLAY with Sand

Soil Type 2 was observed to consist of brown, moist to wet, medium stiff, lean CLAY with sand. Soil Type 2 was observed below Soil Type 1 and extended to an observed depth of approximately nine feet bgs where it was underlain by Soil Type 3.

Soil Type 3 – SILT

Soil Type 3 was observed to consist of brown/grey, wet, SILT. Soil Type 3 was observed below Soil Type 2 and extended to the maximum explored depth of approximately 14 feet bgs.

4.2.2 Groundwater

Groundwater seeps were encountered within test pit exploration TP-1 at 3 feet bgs on January 3, 2022. Groundwater levels are often subject to seasonal variance and may rise during extended periods of increased precipitation or flooding.

Seeps and springs may become evident during site grading, primarily along slopes or in areas cut below existing grade. Structures, roads, and drainage design should be planned accordingly. Piezometer installation and long-term monitoring beyond the scope of this investigation, would be necessary to provide more detailed groundwater information.

5.0 GEOLOGIC HAZARDS

City of La Center Municipal Code (*LCMC Development Code Section 18.300*) defines geologic hazard requirements for proposed development in areas subject to the City of La Center jurisdiction. Three potential geologic hazards are identified: (1) erosion hazard areas, (2) landslide hazard and steep slope areas, and (3) seismic hazard areas.

Columbia West conducted a geologic hazard review to assess whether a geologic hazard is present in the vicinity of proposed development, and if so, to provide mitigation recommendations. The geologic hazard review was based upon physical and visual reconnaissance, subsurface exploration, and review of maps and other published technical literature. The results of the geologic hazard review for potential geologic hazards are discussed in the following sections.

5.1 Erosion Hazard Areas

According to *Clark County Maps Online*, and the *Soil Survey of Clark County, Washington* an erosion hazard is mapped approximately 130 feet northwesterly of the area of proposed development at a mapped contact of surficial soil units consisting of Gee silt loam and Rough Broken Land. This mapped erosion hazard is not anticipated to adversely effect proposed development and no erosion hazard is mapped on the development site. Therefore, according to the *City of La Center Development Code,* a soil erosion hazard is not present at the site. However, if there are erosion concerns, erosion can be successfully mitigated by preparation and adherence to a site-specific erosion control plan that identifies BMPs to be utilized 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 6.14, *Erosion Control Measures*.

5.2 Landslide Hazard and Steep Slope Areas

To evaluate steep slope areas and assess whether landslide hazards are present in the vicinity of proposed development, Columbia West conducted a review of literature, subsurface exploration, and physical slope reconnaissance. As mentioned previously, slope grades of up to 80 percent were observed west and northwest of the site.

5.2.1 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 and neighboring slopes are mapped as (1) stable areas – no slides or unstable slopes.

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.

5.2.2 Slope Reconnaissance and Slope Stability Assessment

Review of topographic mapping published by *Clark County Maps Online* indicates that the subject site is located in an area that slopes regionally downgradient from south to north approximately 100 to 200 feet east to south from neighboring slope crests and 200 to 400 feet from slope toes.

The maximum grade change of neighboring slopes is approximately 100 feet, with slope grades generally ranging from 5 to 80 percent. Slopes appear planar with no observed evidence of instability. There was no observed direct evidence of large-scale, mass slope movements or historic landslides. No landslide debris was observed within subsurface soils explored onsite and no slope face groundwater seeps or springs were observed.

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

5.3 Seismic Hazard Areas

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

5.3.1 Soil Liquefaction and Dynamic Settlement

According to the *Liquefaction Susceptibility Map of Clark County Washington* (Washington State Department of Natural Resources, 2004), the area of proposed development is mapped as very low to moderate susceptibility for liquefaction. Neighboring slopes are primarily mapped as very low susceptibility for liquefaction with areas mapped as high nearing the toe of the northeast trending slope and the wetland. These areas are not anticipated to adversely affect proposed development as Columbia West understand it. Liquefaction, defined as the transformation of the behavior of a granular material from a solid to a liquid due to increased pore-water pressure and reduced effective stress, may occur when granular materials quickly compact under cyclic stresses caused by a seismic event. The effects of liquefaction may include immediate ground settlement and lateral spreading.

Soils most susceptible to liquefaction are generally saturated, cohesionless, loose to medium-dense sands within 50 feet of the ground surface. Recent research has also indicated that low plasticity silts and clays may also be subject to sand-like liquefaction behavior if the plasticity index determined by the Atterberg Limits analysis is less than 8. Potentially liquefiable soils located above the existing, historic, or expected ground water levels do not generally pose a liquefaction hazard. It is important to note that changes in perched ground water elevation may occur due to project development or other factors not observed at the time of investigation.

Based upon results of literature review, site-specific testing, and laboratory analysis, the potential for soil liquefaction in the location of proposed development is considered to be low.

5.3.2 Ground Shaking Amplification

Review of the *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), indicates that site soils in the location of proposed development may be represented by Site Class C to D as defined by the *ASCE 7, Chapter 20, Table 20.3-1*. A designation of Site Class D 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 Columbia West's opinion, and will not prohibit development if properly accounted for during the design process. Soils nearing the toe of the neighboring northeast trending slope and the wetland at its base are mapped as Site Class E but are not expected to adversely affect proposed development.

5.3.3 Fault Rupture

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

6.0 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 utilized and incorporated into the design and construction processes. The primary geotechnical concerns associated with the site are shallow groundwater and fine-textured soils. Design recommendations are presented in the following text sections.

6.1 Site Preparation and Grading

Vegetation, organic material, unsuitable fill, and deleterious material that may be encountered should be cleared from areas identified for structures and site grading. Vegetation, other 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 stripping depth for topsoil and disturbed fill soils is anticipated to be approximately 24 inches. The required stripping depth may increase in areas of existing fill, heavy organics, or previously existing structures. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed. The post-construction maximum depth of landscape fill placed or spread at any location onsite should not exceed one foot.

Previously disturbed soil, debris, or unconsolidated fill encountered during grading or construction activities should be removed completely and thoroughly from structural areas. This includes old remnant foundations, basement walls, utilities, associated soft soils, and debris. These materials and associated disturbed soils should also be completely removed from structural areas. Excavation areas should be backfilled with engineered structural fill.

The test pit excavated during site exploration was backfilled loosely with onsite soils. The test pit should be located and properly backfilled with structural fill during site improvements construction. Trees, stumps, and associated roots should also be removed from structural areas, individually and carefully. Resulting cavities and excavation areas should be backfilled with engineered structural fill.

Site grading activities should be performed in accordance with requirements specified in the *2018 International Building Code* (IBC), Chapter 18 and Appendix J, with exceptions noted in the text herein. Site preparation, soil stripping, and grading activities should be observed and documented by Columbia West.

6.2 Engineered Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in the preceding text. Surface soils should be scarified and compacted prior to additional fill placement. Engineered structural fill should be placed in loose lifts not exceeding 12 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within two percentage points of optimum conditions. A field density at least equal to 95 percent of the maximum dry density, obtained from the modified Proctor moisture-density relationship test (ASTM D1557), is recommended for structural fill placement and scarified and recompacted subgrade.

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. Engineered fill placement should be observed by Columbia West.

Engineered structural fill placement activities should be performed during dry summer months if possible. Most clean native soils may be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native soils may require addition of moisture during periods of dry weather. Compacted fill soils should be covered shortly after placement. Native soils will likely expand during excavation and transport, and consolidate during compaction. Development of site-specific expansion and consolidation factors is beyond the scope of this investigation. Columbia West can provide site-specific factors upon request.

Because they are moisture-sensitive, fine-textured soils are often difficult to excavate and compact during wet weather conditions. If adequate compaction is not achievable with clean native soils, import structural fill consisting of granular fill meeting WSDOT specifications for *Gravel Borrow 9-03.14*(*1*) is recommended.

Representative samples of proposed engineered structural fill should be submitted for laboratory analysis and approval by Columbia West prior to placement. Laboratory analyses should include particle-size gradation and standard Proctor moisture-density analysis.

6.3 Cut and Fill Slopes

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 3. 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 4.

Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Fill slopes should be constructed by placing fill material in maximum 12-inch level lifts, compacting as described in Section 6.2, *Engineered Structural Fill* and horizontally benching where appropriate. 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.

6.4 Foundations

Foundations for proposed structures are anticipated to consist of shallow continuous perimeter or column spread footings. Footings should be designed by a licensed structural engineer and conform to the recommendations below. Typical building loads are not expected to exceed approximately 3 kips per foot for perimeter footings or 10 kips per column. If actual loading exceeds anticipated loading, additional analysis should be conducted for the specific load conditions and proposed footing dimensions.

The existing ground surface should be prepared as described in Section 6.1, *Site Preparation and Grading*, and Section 6.2, *Engineered Structural Fill*. Foundations should bear upon firm native soil or engineered structural fill.

To evaluate bearing capacity for proposed structures, serviceability and reliability of shear resistance for subsurface soils was considered. Allowable bearing capacity is typically a function of footing dimension and subsurface soil properties, including settlement and shear resistance. Based upon in situ field testing and laboratory analysis, the estimated allowable bearing capacity for well-drained foundations prepared as described above is 1,500 psf. Bearing capacity may be increased by one-third for transient lateral forces such as seismic or wind. The estimated coefficient of friction between in situ compacted native soil or engineered structural fill and in-place poured concrete is 0.35. Lateral forces may also be resisted by an assumed passive soil equivalent fluid pressure of 250 psf/f against embedded footings. The upper six inches of soil should be neglected in passive pressure calculations.

Footings should extend to a depth at least 18 inches below lowest adjacent grade to provide adequate bearing capacity and protection against frost heave. Foundations constructed during wet weather conditions will require over-excavation of saturated subgrade soils and granular structural backfill prior to concrete placement. Over-excavation recommendations should be provided by Columbia West during foundation excavation and construction. Excavations adjacent to foundations should not extend within a 2H:1V angle projected down from the outside bottom footing edge without additional geotechnical analysis.

Foundations should not be permitted to bear upon undocumented fill or disturbed soil. Columbia West should observe foundation excavations prior to placing forms or reinforcing bar to verify subgrade support conditions are as anticipated in this report.

6.5 Slabs on Grade

Proposed structures may have slab-on-grade floors. Slabs should be supported on firm, competent, in situ soil or engineered structural fill. Disturbed soils and unsuitable fills in proposed slab locations should be removed and replaced with structural fill.

Preparation beneath slabs should be performed in accordance with the recommendations presented in Section 6.1, *Site Preparation and Grading* and Section 6.2, *Engineered Structural Fill*. Slabs should be underlain by at least 6 inches of 1 ¼”-0 crushed aggregate meeting *WSDOT 9-03.9*(*3*). Geotextile filter fabric conforming to *WSDOT 2010 Standard Specification M 41-10, 9-33.2*(*1*), *Geotextile Properties, Table 3: Geotextile for Separation or Soil Stabilization* may be used below the crushed aggregate to increase subgrade support. Base aggregate should be compacted to at least 95 percent of maximum dry density determined by the modified Proctor moisture-density relationship test (ASTM D1557).

For lightly loaded slabs not exceeding 200 psf, the modulus of subgrade reaction is estimated to be 150 psi/inch. Columbia West should be contacted for additional analysis if slab loading exceeds 200 psf. If desired, a moisture barrier may be constructed beneath the slabs. Slabs should be appropriately waterproofed in accordance with the desired type of finished flooring. Slab thickness and reinforcement should be designed by an experienced structural engineer in accordance with anticipated loads.

6.6 Static Settlement

Total long-term static footing displacement for shallow foundations constructed as described in this report is not anticipated to exceed approximately 1 inch. Differential settlement between comparably loaded footing elements is not expected to exceed approximately ½ inch over a span of 50 feet. The resulting vertical displacement after loading may be due to elastic distortion, dissipation of excess pore pressure, or soil creep.

6.7 Excavation

Soils at the site were explored to a maximum depth of 14 feet using a track-mounted excavator. Bedrock was not encountered and blasting or specialized rock-excavation techniques are not anticipated. Perched groundwater layers may exist at shallow depths depending on seasonal fluctuations in the water table. Recommendations presented in Section 6.8, *Dewatering* should be considered where below-grade construction intersects the shallow groundwater table.

Based upon laboratory analysis and field testing, near-surface soils may be Washington State Industrial Safety and Health Administration (WISHA) Type C. For temporary open-cut excavations deeper than four feet, but less than 20 feet in soils of these types, the maximum allowable slope is 1.5H:1V. WISHA soil type should be confirmed during field construction activities by the contractor. Soil is often anisotropic and heterogeneous, and it is possible that WISHA soil types determined in the field may differ from those described above.

Site-specific shoring design 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 soldier piles and timber lagging, sheet pile walls, tiebacks and shotcrete, or pre-fabricated hydraulic shoring. Because lateral earth pressure distributions acting on below-grade structures are dependent upon the type of shoring system used, Columbia West should be contacted to conduct additional analysis when shoring type, excavation depths, and locations are known.

The contractor should be held responsible for site safety, sloping, and shoring. Columbia West is not responsible for contractor activities and excavation should be conducted in accordance with all applicable local, state, and federal laws.

6.8 Dewatering

Groundwater elevation and hydrostatic pressure should be carefully considered during design of utilities, retaining walls, or other structures that require below-grade excavation. Utility trenches in shallow groundwater areas or excavations and cuts that remain open for even short periods of time may undermine or collapse due to groundwater effects. Placement of layers of riprap or quarry spalls in localized areas on shallow excavation side slopes may be required to limit instability. Over-excavation and stabilization of pipe trenches or other excavations with imported crushed aggregate or gabion rock may also be necessary to provide adequate subgrade support.

Significant pumping and dewatering may be required to temporarily reduce the groundwater elevation to allow construction of proposed below-grade structures, installation of utilities, or placement of structural fills. 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. Well pumps should remain functioning at all times during the excavation and construction period. Suitable back-up pumps and power supplies should be available to prevent unanticipated shut-down of dewatering equipment. Failure to operate pumps full-time may result in flooding of the excavation zones, resulting in damage to forms, slopes, or equipment.

6.9 Lateral Earth Pressure

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 6.4, *Foundations*. Retained material may include engineered structural backfill or undisturbed native soil. Structural wall backfill should consist of imported granular material meeting *Section 9-03.12*(2) of *WSDOT Standard Specifications*. Backfill should be prepared and compacted to at least 95 percent of maximum dry density as determined by the modified Proctor test *(ASTM D1557).* Recommended parameters for lateral earth pressures for retained soils and engineered structural backfill consisting of imported granular fill meeting WSDOT specifications for *Gravel Backfill for Walls 9-03.12*(2) are presented in Table 1.

Table 1. Recommended Lateral Earth Pressure Parameters for Level Backfill

Retained Soil	Equivalent Fluid Pressure for Level Backfill			Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive		
Undisturbed native Lean CLAY with Sand (Soil Type 2)	60 pcf	41 pcf	293 pcf	110 pcf	27°
Undisturbed native SILT (Soil Type 3)	61 pcf	42 pcf	319 pcf	115 pcf	28°
Approved Structural Backfill Material WSDOT 9-03.12(2) compacted aggregate backfill	56 pcf	35 pcf	520 pcf	135 pcf	36°

**The upper 6 inches of soil should be neglected in passive pressure calculations. If exterior grade from top or toe of retaining wall is sloped, Columbia West should be contacted to provide location-specific lateral earth pressures.*

The design parameters presented in Table 1 are valid for static loading cases only and are based upon in situ undisturbed native soils or compacted granular fill. The recommended earth pressures do not include surcharge loads, dynamic loading, hydrostatic pressure, or seismic design. If sloped backfill conditions are proposed, Columbia West should be contacted for additional analysis and associated recommendations.

If seismic design is required for unrestrained walls, seismic forces may be calculated by superimposing a uniform lateral force of $10H^2$ pounds per lineal foot of wall, where H is the total wall height in feet. The resultant force should be applied at 0.6H from the base of the wall.

A continuous one-foot-thick zone of free-draining, washed, open-graded 1-inch by 2-inch drain rock and a 4-inch perforated gravity drainpipe is assumed behind retaining walls. Geotextile filter fabric should be placed between the drain rock and backfill soil. Specifications for drainpipe design are presented in Section 6.11, *Drainage*. If walls cannot be gravity drained, saturated base conditions and/or applicable hydrostatic pressures should be assumed.

Final retaining wall design should be reviewed and approved by Columbia West. Retaining wall subgrade and backfill activities should also be observed and tested for compliance with recommended specifications by Columbia West during construction.

6.10 Seismic Design Considerations

According to the *ASCE 7 Hazard Tool,* the anticipated peak ground and maximum considered earthquake spectral response accelerations resulting from seismic activity for the subject site are summarized in Table 2.

Table 2. Approximate Probabilistic Ground Motion Values for 'firm rock' sites based on subject property longitude and latitude

	2% Probability of Exceedance in 50 yrs
Peak Ground Acceleration	0.363 g
0.2 sec Spectral Acceleration	0.802 g
1.0 sec Spectral Acceleration	0.378 g

The listed probabilistic ground motion values are based upon "firm rock" sites with an assumed shear wave velocity of 2,500 ft/s in the upper 100 feet of soil profile. These values should be adjusted for site class effects by applying site coefficients F_a and F_v and F_{PGA} as defined by *ASCE 7-16 and associated ASCE 7-16 Supplement 1, dated December 12, 2018, Tables 11.4-1, 11.4-2, and 11.8-1*. The site coefficients are intended to more accurately characterize estimated peak ground and respective earthquake spectral response accelerations by considering site-specific soil characteristics and index properties.

Localized peak ground accelerations exceeding the adjusted values may occur in some areas in direct proximity to an earthquake's origin. This may be a result of amplification of seismic energy due to depth to competent bedrock, compression and shear wave velocity of bedrock, presence and thickness of loose, unconsolidated alluvial deposits, soil plasticity, grain size, and other factors.

Identification of specific seismic response spectra is beyond the scope of this investigation. If site structures are designed in accordance with recommendations specified in the *2018 IBC*, the potential for peak ground accelerations in excess of the adjusted and amplified values should be understood.

6.11 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. Depressions or shallow areas that may retain ponding water should be avoided. Roof drains, low-point drains, and perimeter foundation drains are recommended for structures. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from foundations into an approved discharge location.

Perimeter foundation drains should consist of 3-inch perforated PVC pipe surrounded by a minimum of 1 ft^3 of clean, washed drain rock per linear foot of pipe and wrapped with geotextile filter fabric. Open-graded drain rock with a maximum particle size of 3 inches and less than 2 percent passing the No. 200 sieve is recommended. Geotextile filter fabric should consist of Mirafi 140N or approved equivalent, with AOS between No. 70 and No. 100 sieve. The water permittivity should be greater than 1.5/sec. Figure 5 presents a typical foundation drain. Perimeter drains may limit increased hydrostatic pressure beneath footings and assist in reducing potential perched moisture areas.

Subdrains should also be considered if portions of the site are cut below surrounding grades. Shallow groundwater, springs, or seeps should be conveyed via drainage channel or perforated pipe into an approved discharge. Recommendations for design and installation of perforated drainage pipe may be performed on a case-by-case basis by Columbia West during construction. Failure to provide adequate surface and sub-surface drainage may result in soil slumping or unanticipated settlement of structures exceeding tolerable limits. A typical perforated drainpipe trench detail is presented in Figure 6.

Site improvements construction in some areas may occur at or near the shallow groundwater table, particularly if work is conducted during wet-weather conditions. Dewatering may be necessary, and a drainage mat may be required to achieve sufficient elevation for fill placement. A typical drainage mat is shown on Figure 7. Columbia West should determine drainage mat location, extent, and thickness when subsurface conditions are exposed. Drainage mats may need to be constructed in conjunction with subdrains to convey captured water to an approved discharge location.

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. Columbia West should be consulted to provide appropriate recommendations.

6.12 Bituminous Asphalt and Portland Cement Concrete

Based upon correspondence with the client, proposed development may include new public asphalt-paved right-of-way improvements. Columbia West recommends adherence to City of La Center paving guidelines for roadway improvements in the public right-of-way.

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 pavement construction is discussed in Section 6.13, *Wet Weather Construction Methods and Techniques*. Subgrade conditions should be evaluated and tested by Columbia West prior to placement of crushed aggregate base. Subgrade evaluation should include nuclear gauge density testing and wheel proof-roll observations conducted with a loaded 12-cubic yard, double-axle dump truck or equivalent. Nuclear gauge density testing should be conducted at 150-foot intervals or as determined by the onsite geotechnical engineer. Subgrade soil should be compacted to at least 95 percent of the modified Proctor dry density, as determined by *ASTM D1557*. Areas of observed deflection or rutting during proof-roll evaluation should be excavated to a firm surface and replaced with compacted crushed aggregate.

Aggregate base should consist of 1 ¼”-0 crushed aggregate meeting *WSDOT 9-03.9*(*3*) and be compacted to at least 95 percent of maximum dry density as determined by *ASTM D1557*. Aggregate base should also be subject to proof-roll observations as described above. Asphalt concrete pavement should be compacted to at least 91 percent of maximum Rice density. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with WSDOT and City of La Center specifications.

Portland cement concrete curbs and sidewalks should be installed in accordance with City of La Center specifications. Curb and sidewalk aggregate base should consist of 1 ¼"-0 crushed aggregate meeting *WSDOT 9-03.9*(*3*) and be compacted to at least 95 percent of maximum dry density as determined by *ASTM D1557*. Curb and sidewalk base should also be subject to proof-roll observations as described above. Soft areas that deflect or rut should be stabilized prior to pouring concrete. Concrete should be tested during installation in accordance with *ASTM C171, C138, C231, C143, C1064, and C31*. This includes casting of cylinder specimen at a frequency of four cylinders per 100 cubic yards of poured concrete. Recommended field concrete testing includes slump, air entrainment, temperature, and unit weight.

6.13 Wet Weather Construction Methods and Techniques

Wet weather construction often results in significant shear strength reduction and soft areas that may rut or deflect. Installation of granular working layers may be necessary to provide a firm support base and sustain construction equipment. Granular layers should consist of all-weather gravel, 2x4-inch gabion, or other similar material (six-inch maximum size with less than five percent passing the No. 200 sieve).

Construction equipment traffic across exposed soil should be minimized. Equipment traffic induces dynamic loading, which may result in weak areas and significant reduction in shear strength for wet soils. Wet weather construction may also result in generation of significant excess quantities of soft wet soil. This material should be removed from the site or stockpiled in a designated area.

Construction during wet weather conditions may require increased base thickness. Over-excavation of subgrade soils or subgrade amendment with lime and/or cement may be necessary to provide a firm base upon which to place crushed aggregate. Geotextile filter fabric is also recommended. If soil amendment with lime or cement is considered, Columbia West should be contacted to provide appropriate recommendations based upon observed field conditions and desired performance criteria.

Crushed aggregate base should be installed in a single lift with trucks end-dumping from an advancing pad of granular fill. During extended wet periods, stripping activities may also need to be conducted from an advancing pad of granular fill. Once installed, the crushed aggregate base should be compacted with several passes from a static drum roller. A vibratory compactor is not recommended because it may further disturb the subgrade. Subdrains may also be necessary to provide subgrade drainage and maintain structural integrity.

Aggregate base should consist of 1 ¼"-0 crushed aggregate meeting *WSDOT 9-03.9*(*3*) and be compacted to at least 95 percent of maximum dry density according to the modified Proctor density test *(ASTM D1557).* Compaction should be verified by nuclear gauge density testing, conducted at 150-foot intervals or as determined by the onsite geotechnical engineer. Observation of a proof-roll with a loaded dump truck is also recommended as an indication of the compacted aggregate's performance.

It should be understood that wet weather construction is risky and costly. Columbia West should observe and document wet weather construction activities. Proper construction methods and techniques are critical to overall project integrity.

6.14 Erosion Control Measures

Based upon field observations and laboratory testing, the erosion hazard for site soils in flat to shallow-gradient portions of the property is likely to be low. The potential for erosion generally increases in sloped areas. Therefore, disturbance to vegetation in sloped areas should be minimized during construction activities. Soil is also prone to erosion if unprotected and unvegetated during periods of increases precipitation. Erosion can be minimized by performing construction activities during dry summer months.

Site-specific erosion control measures should be implemented to address the maintenance of exposed areas. This may include silt fence, biofilter bags, straw wattles, or other suitable methods. During construction activities, exposed areas should be well-compacted and protected from erosion with visqueen, surface tackifier, or other means, as appropriate. Temporary slopes or exposed areas may be covered with straw, crushed aggregate, or riprap in localized areas to minimize erosion. Erosion and water runoff during wet weather conditions may be controlled by application of strategically placed channels and small detention depressions with overflow pipes.

After grading, exposed surfaces should be vegetated as soon as possible with erosion-resistant native vegetation. Jute mesh or straw may be applied to enhance vegetation. Once established, vegetation should be properly maintained. Disturbance to existing native vegetation and surrounding organic soil should also be minimized during construction activities.

6.15 Soil Shrink/Swell Potential

Based upon laboratory analysis, near-surface soils contain as much as approximately 93 percent by weight passing the No. 200 sieve and exhibit a plasticity index ranging from 9 to 11 percent. This indicates the potential for soil shrinking or swelling and underscores the importance of proper moisture conditioning during fill placement.

6.16 Utility Installation

Utility installation may require subsurface excavation and trenching. Excavation, trenching and shoring should conform to federal (Occupational Safety and Health Administration) (OSHA) (29 CFR, Part 1926) and WISHA (WAC, Chapter 296-155) regulations. Site soils may slough when cut vertically and sudden precipitation events or perched groundwater may result in accumulation of water within excavation zones and trenches.

Utilities should be installed in general accordance with manufacturer's recommendations. Utility trench backfill 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). The remaining backfill should be compacted to at least 95 percent of maximum dry density as determined by the standard Proctor moisture-density test *(ASTM D698).* Clean, free-draining, fine bedding sand is recommended for use in the pipe zone. With exception of the pipe zone, backfill should be placed in loose lifts not exceeding 12 inches in thickness.

Compaction of utility trench backfill material should be verified by nuclear gauge field compaction testing performed in accordance with *ASTM D6938*. Field compaction testing should be performed at 200-foot intervals along the utility trench centerline at the surface and midpoint depth of the trench. Compaction frequency and specifications may be modified for non-structural areas in accordance with recommendations of the site geotechnical engineer.

7.0 CONCLUSION AND LIMITATIONS

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

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

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

This report was prepared solely for the client and is not to be reproduced without prior authorization from Columbia West. Final engineering plans and specifications for the project should be reviewed and approved by Columbia West as they relate to geotechnical and grading issues prior to final design approval. Columbia West is not responsible for independent conclusions or recommendations made by other parties based upon information presented in this report. Unless a particular service was expressly included in the scope, it was not performed and there should be no assumptions based upon services not provided. Additional report limitations and important information about this document are presented in Appendix E. This information should be carefully read and understood by the client and other parties reviewing this document.

Sincerely,

COLUMBIA WEST ENGINEERING, Inc.

Lance V. Lehto, PE, GE
President

LANCE V. LEHTO
STATE OF WASHINGTON
36301
REGISTERED
PROFESSIONAL ENGINEER

3-8-2022

REFERENCES

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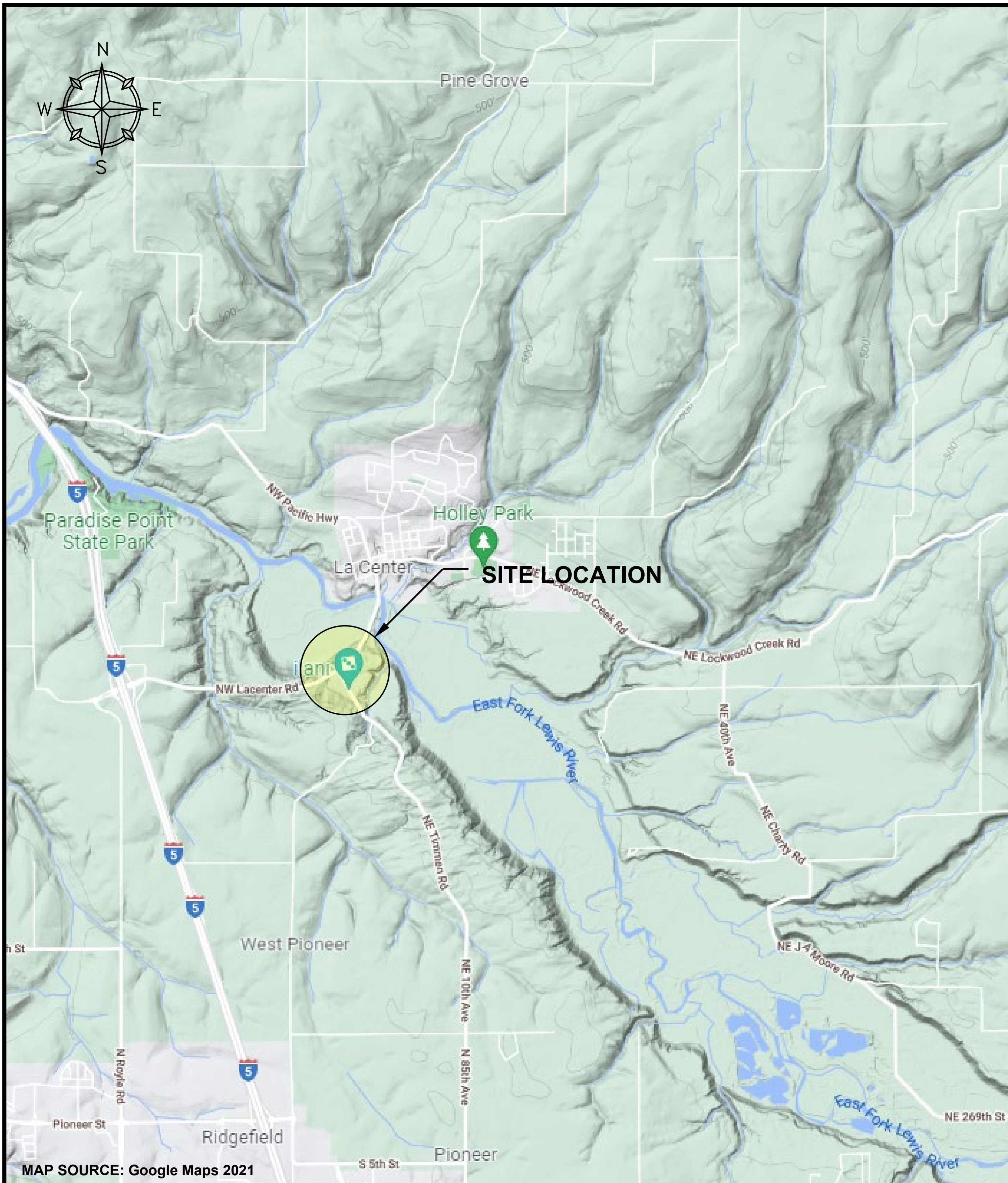
FIGURES



Geotechnical ▪ Environmental ▪ Special Inspections **Columbia West** Engineering, Inc	Job No: 21300 Date: 1/25/22 Drawn: EMU Checked: CWS	SITE LOCATION MAP LA CENTER FLOW STATION #1 LA CENTER, WASHINGTON	FIGURE 1

LOCATION OF TEST PIT

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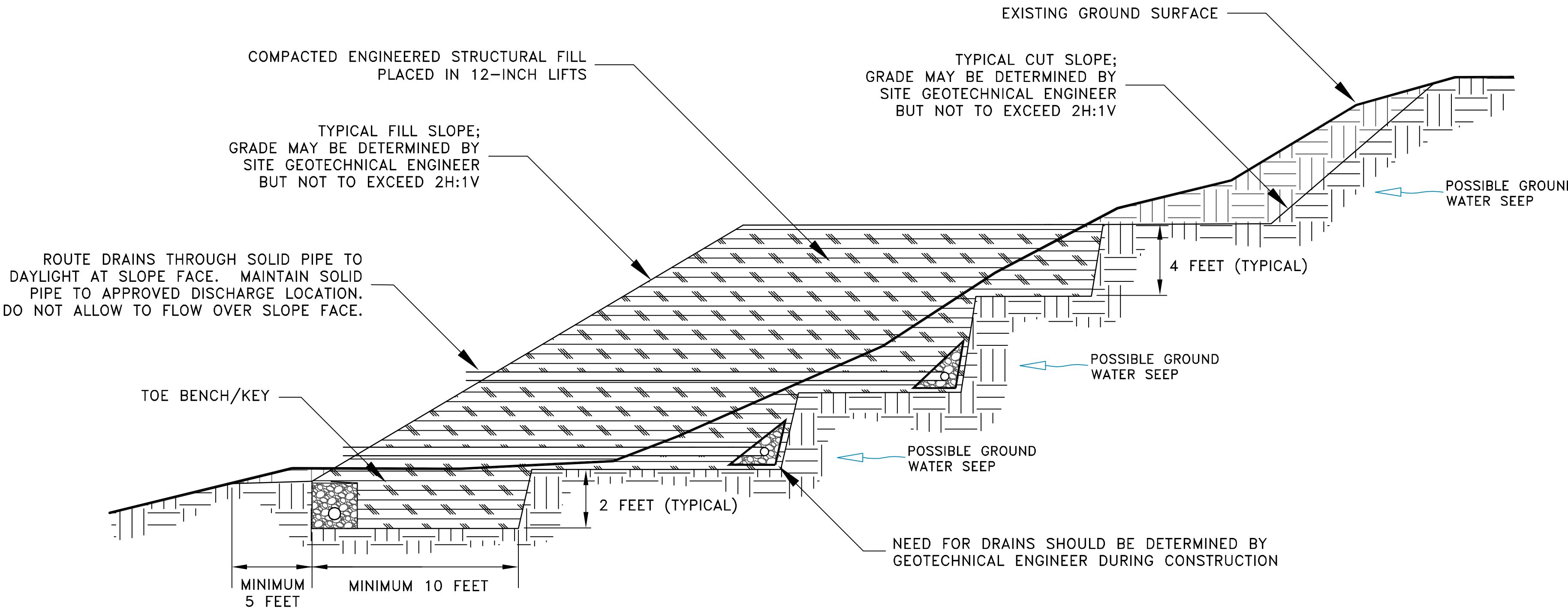
Columbia West Engineering, Inc

Job No: 21300
Date: 01/26/22
Drawn: EMU
Checked: CWS

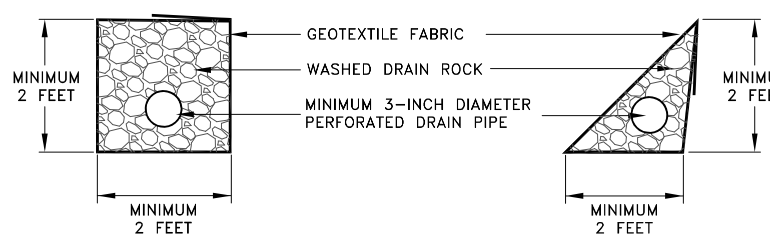
EXPLORATION LOCATION MAP
LA CENTER FLOW STATION #1

NOTES:
1. SITE LOCATION: 32100 NW LA CENTER ROAD IN RIDGEFIELD, WASHINGTON.
2. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
3. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
4. TEST PITS BACKFILLED LOOSELY WITH ONSITE SOILS ON JANUARY 3, 2022.

FIGURE 2



TYPICAL DRAIN SECTION DETAIL



DRAIN SPECIFICATIONS

GEOTEXTILE FABRIC SHALL CONSIST OF MIRAFI 140N OR APPROVED EQUIVALENT WITH AOS BETWEEN No. 70 AND No. 100 SIEVE.

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 3 INCHES.

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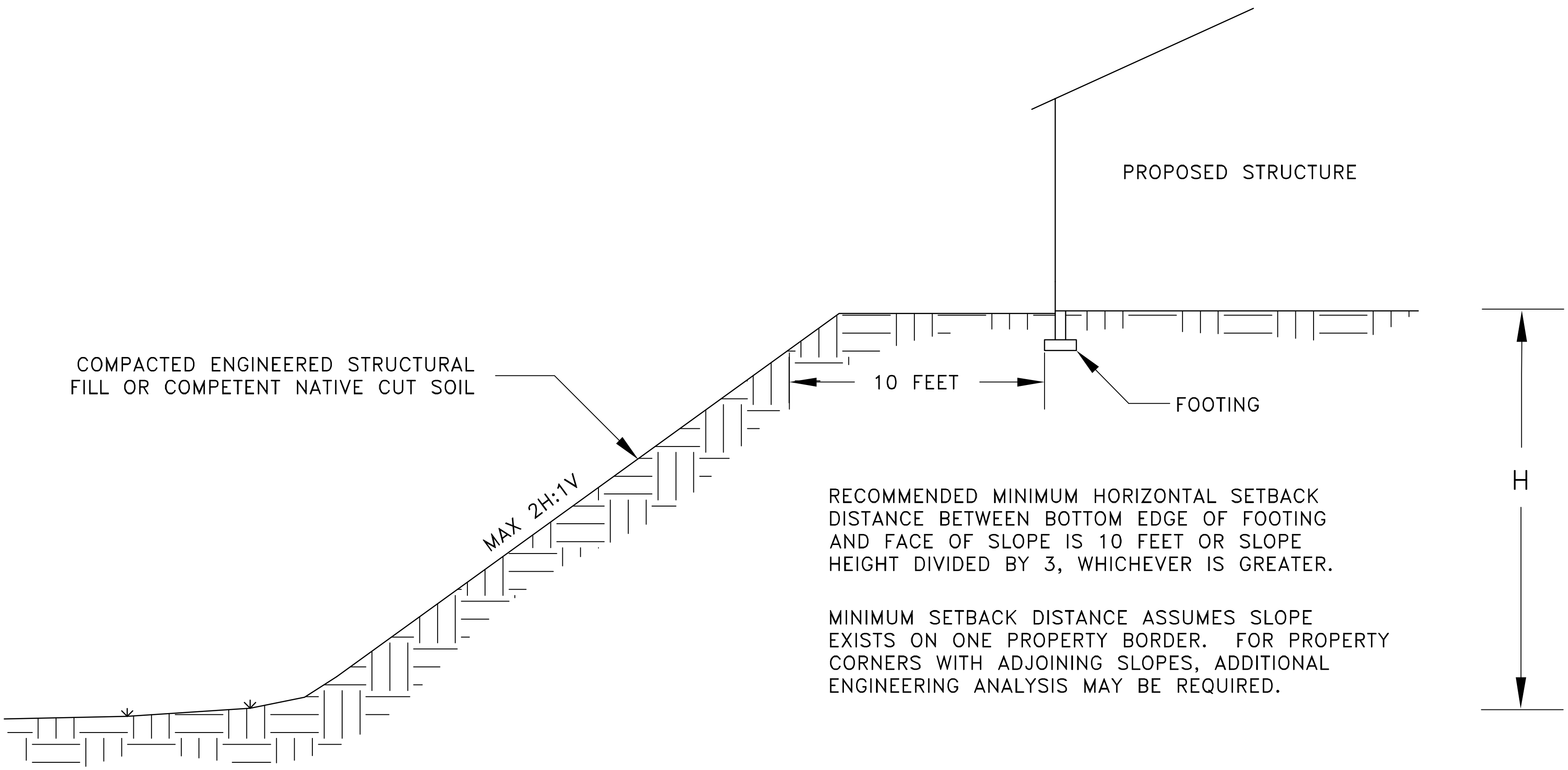
TYPICAL CUT AND FILL SLOPE CROSS SECTION

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

FIGURE 3



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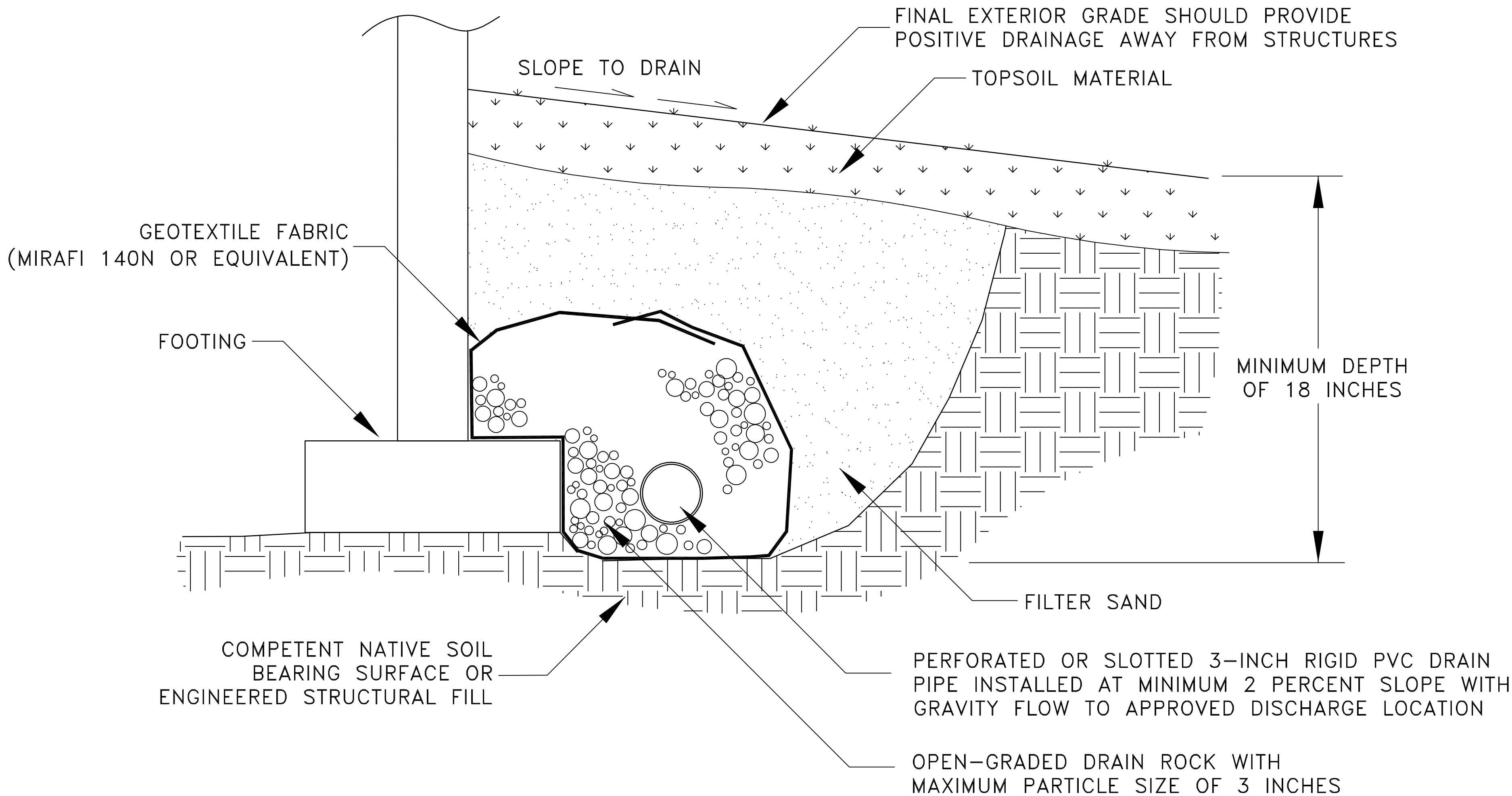
TYPICAL MINIMUM FOUNDATION SLOPE SETBACK DETAIL

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 4



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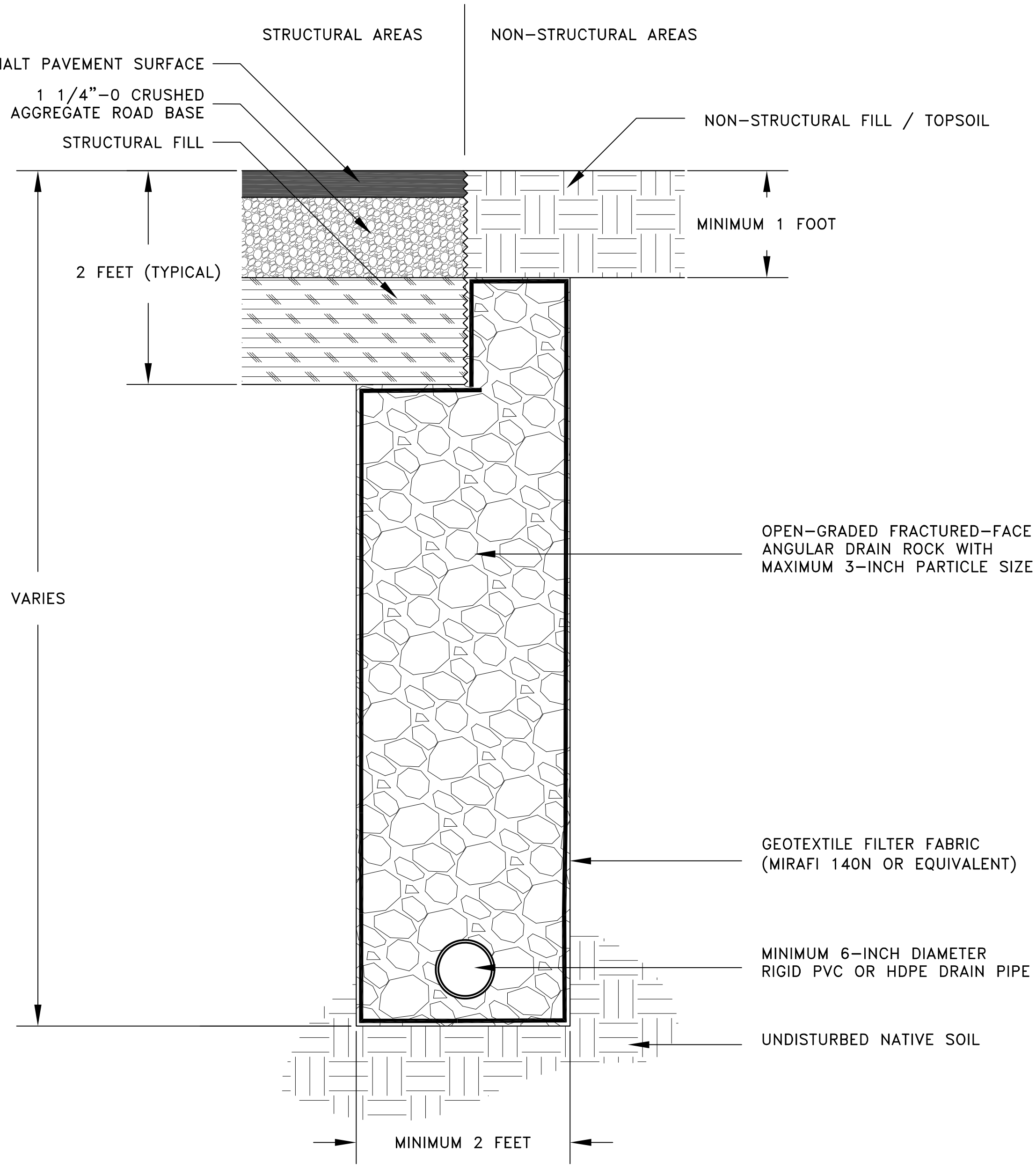
TYPICAL PERIMETER FOOTING DRAIN DETAIL

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

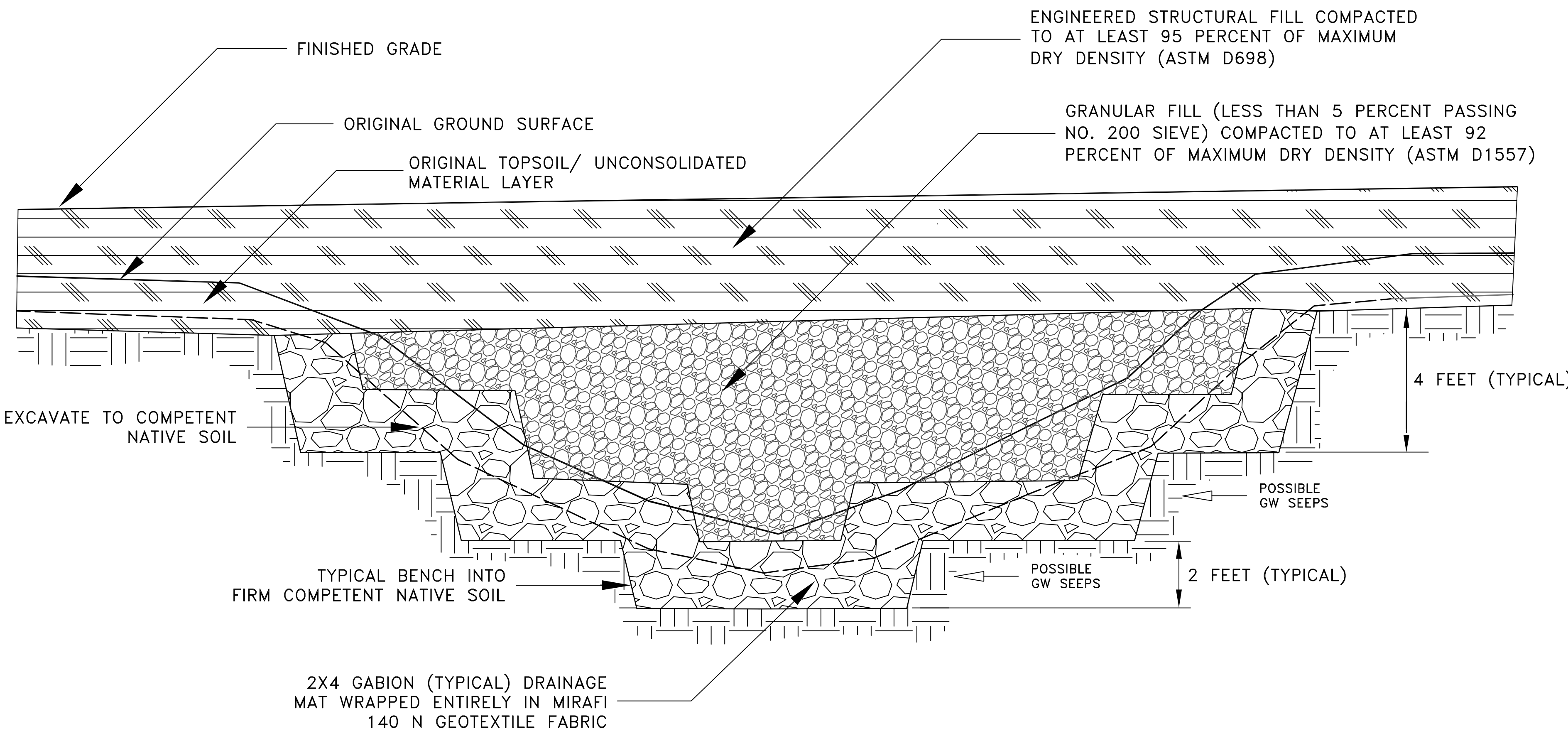
FIGURE 5



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 6

TYPICAL DRAINAGE MAT CROSS-SECTION



Geotechnical ▪ Environmental ▪ Special Inspections Columbia West Engineering, Inc	TYPICAL DRAINAGE MAT CROSS SECTION	NOTES: 1. DRAWING IS NOT TO SCALE. 2. DRAWING REPRESENTS TYPICAL DRAINAGE MAT SECTION AND MAY NOT BE SITE-SPECIFIC.	FIGURE 7

APPENDIX A
LABORATORY TEST RESULTS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
La Center Flow Station #1 La Center, Washington	Clark Public Utilities PO Box 8900 Vancouver, Washington 98668	21300	S22-0020
		REPORT DATE: 01/24/22	FIELD ID: TP1.1
		DATE SAMPLED: 01/03/22	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-01 depth = 3 feet	CL, Lean Clay with Sand

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-6(8)

LABORATORY TEST DATA

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

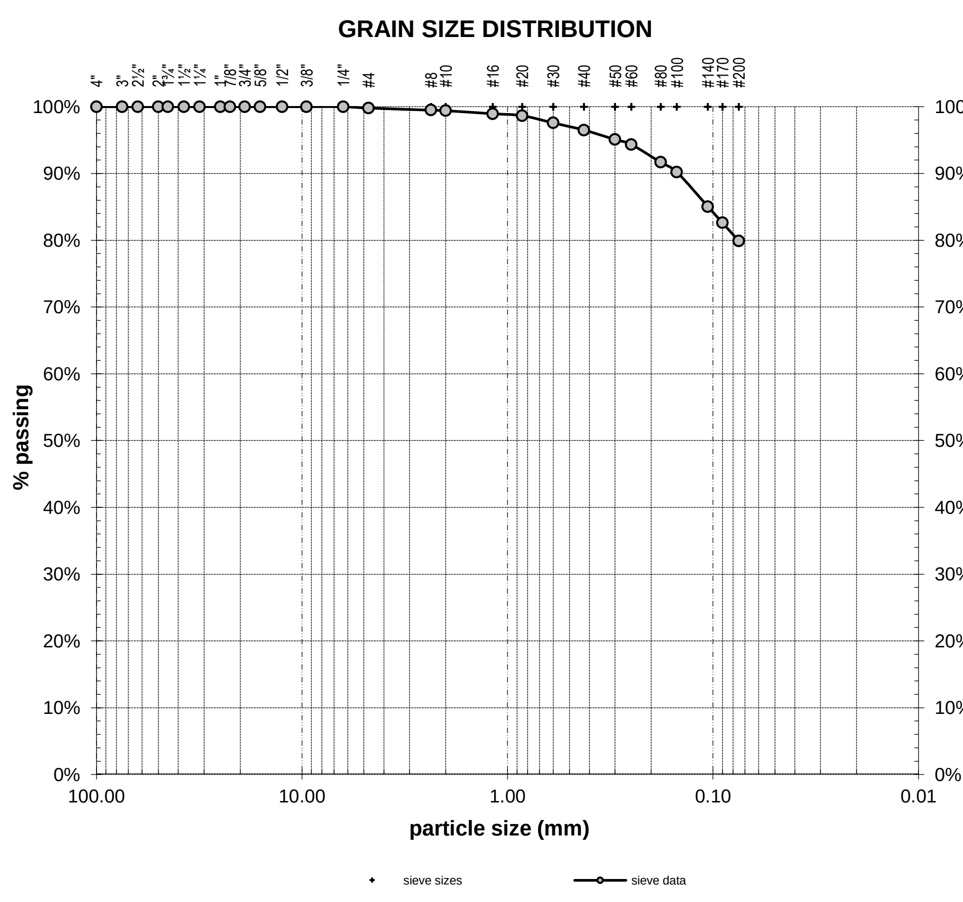
ADDITIONAL DATA

initial dry mass (g) =	204.20		
as-received moisture content =	31.6%	coefficient of curvature, C_C =	n/a
liquid limit =	33	coefficient of uniformity, C_U =	n/a
plastic limit =	22	effective size, $D_{(10)}$ =	n/a
plasticity index =	11	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.2%
% sand = 19.9%
% silt and clay = 79.9%

	SIEVE SIZE US	mm	PERCENT PASSING SIEVE act.	interp.	SPECS max	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		99%		
	#10	2.00	99%			
	#16	1.18		99%		
	#20	0.850	99%			
	#30	0.600		98%		
	#40	0.425	96%			
	#50	0.300		95%		
	#60	0.250	94%			
	#80	0.180		92%		
	#100	0.150	90%			
	#140	0.106		85%		
	#170	0.090		83%		
	#200	0.075	80%			



DATE TESTED	TESTED BY
01/21/22	BTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
La Center Flow Station #1 La Center, Washington	Clark Public Utilities PO Box 8900 Vancouver, Washington 98668	21300	S22-0020
		REPORT DATE: 01/24/22	FIELD ID: TP1.1
		DATE SAMPLED: 01/03/22	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-01 depth = 3 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

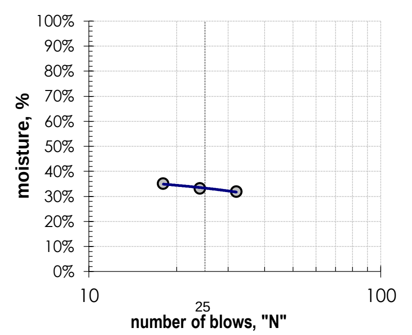
ATTERBERG LIMITS

liquid limit = 33
plastic limit = 22
plasticity index = 11

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.92	33.18	33.08	
dry soil + pan weight, g =	29.25	30.12	29.91	
pan weight, g =	20.87	20.90	20.87	
N (blows) =	32	24	18	
moisture, % =	31.9 %	33.2 %	35.1 %	

LIQUID LIMIT



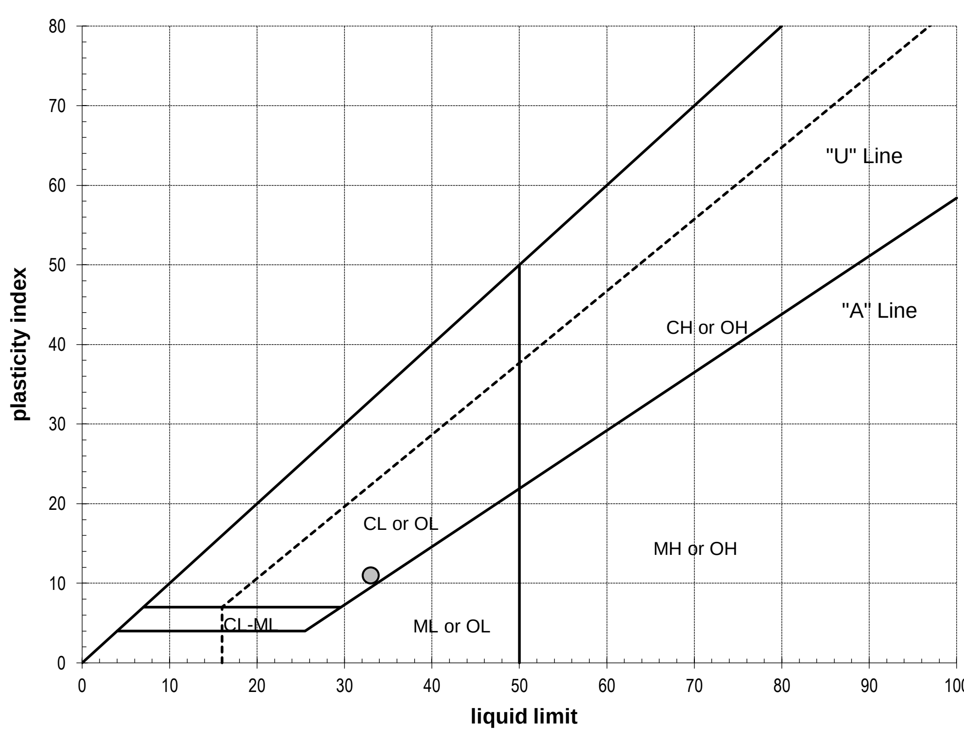
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.03	27.41		
dry soil + pan weight, g =	25.96	26.23		
pan weight, g =	20.96	20.78		
moisture, % =	21.4 %	21.7 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.2%
% sand = 19.9%
% silt and clay = 79.9%
% silt = n/a
% clay = n/a
moisture content = 31.6%

DATE TESTED	TESTED BY
01/21/22	MJR

COLUMBIA WEST ENGINEERING, INC. authorized signature

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
La Center Flow Station #1 La Center, Washington	Clark Public Utilities PO Box 8900 Vancouver, Washington 98668	21300	S22-0021
		REPORT DATE: 01/24/22	FIELD ID: TP1.2
		DATE SAMPLED: 01/03/22	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
SILT	Test Pit TP-01 depth = 10 feet	ML, Silt

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-4(10)

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

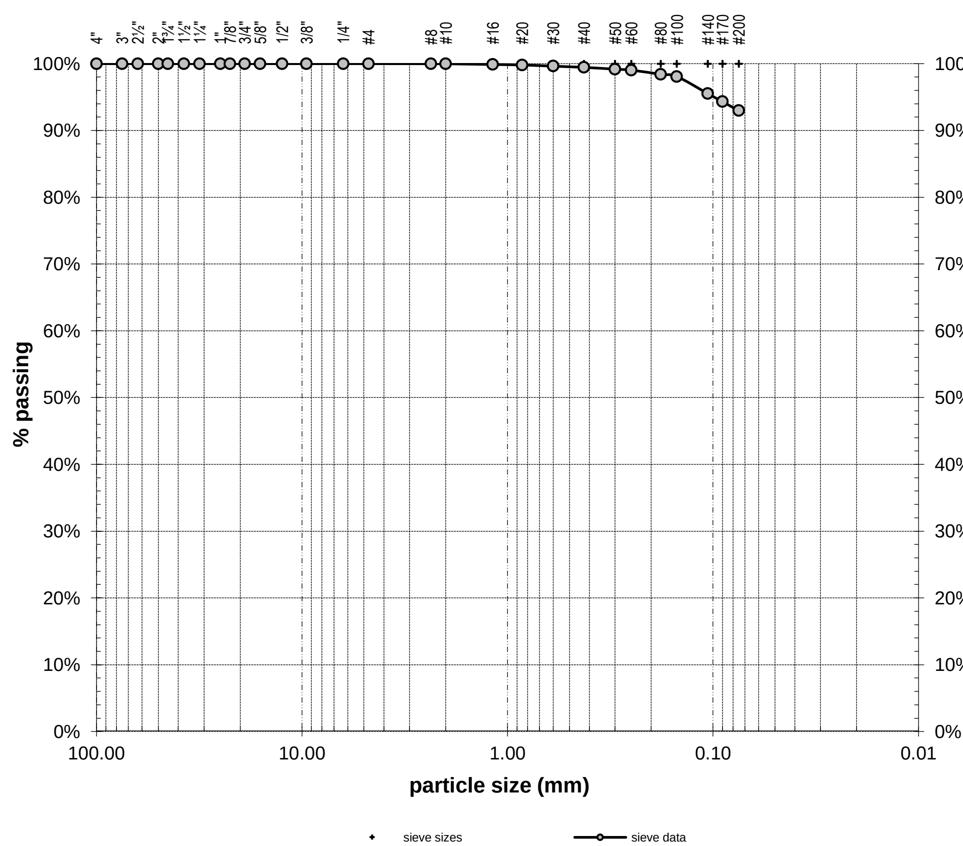
initial dry mass (g) =	200.79		
as-received moisture content =	38.6%	coefficient of curvature, C_C =	n/a
liquid limit =	37	coefficient of uniformity, C_U =	n/a
plastic limit =	28	effective size, $D_{(10)}$ =	n/a
plasticity index =	9	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 7.0%
% silt and clay = 93.0%

	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		100%		
	#40	0.425	99%			
	#50	0.300		99%		
	#60	0.250	99%			
	#80	0.180		98%		
	#100	0.150	98%			
	#140	0.106		96%		
	#170	0.090		94%		
	#200	0.075	93%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
01/21/22	BTT

COLUMBIA WEST ENGINEERING, INC. authorized signature

11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
La Center Flow Station #1 La Center, Washington	Clark Public Utilities PO Box 8900 Vancouver, Washington 98668	21300	S22-0021
		REPORT DATE: 01/24/22	FIELD ID: TP1.2
		DATE SAMPLED: 01/03/22	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
SILT	Test Pit TP-01 depth = 10 feet	ML, Silt

LABORATORY TEST DATA

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

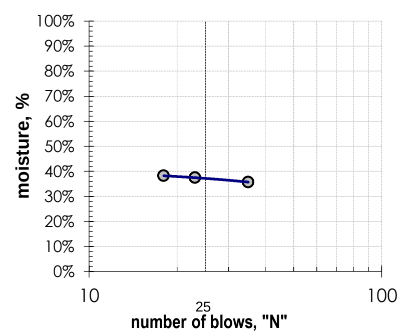
ATTERBERG LIMITS

liquid limit = 37
plastic limit = 28
plasticity index = 9

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	36.18	35.66	32.25	
dry soil + pan weight, g =	32.14	31.56	29.12	
pan weight, g =	20.82	20.62	20.93	
N (blows) =	35	23	18	
moisture, % =	35.7 %	37.5 %	38.2 %	

LIQUID LIMIT



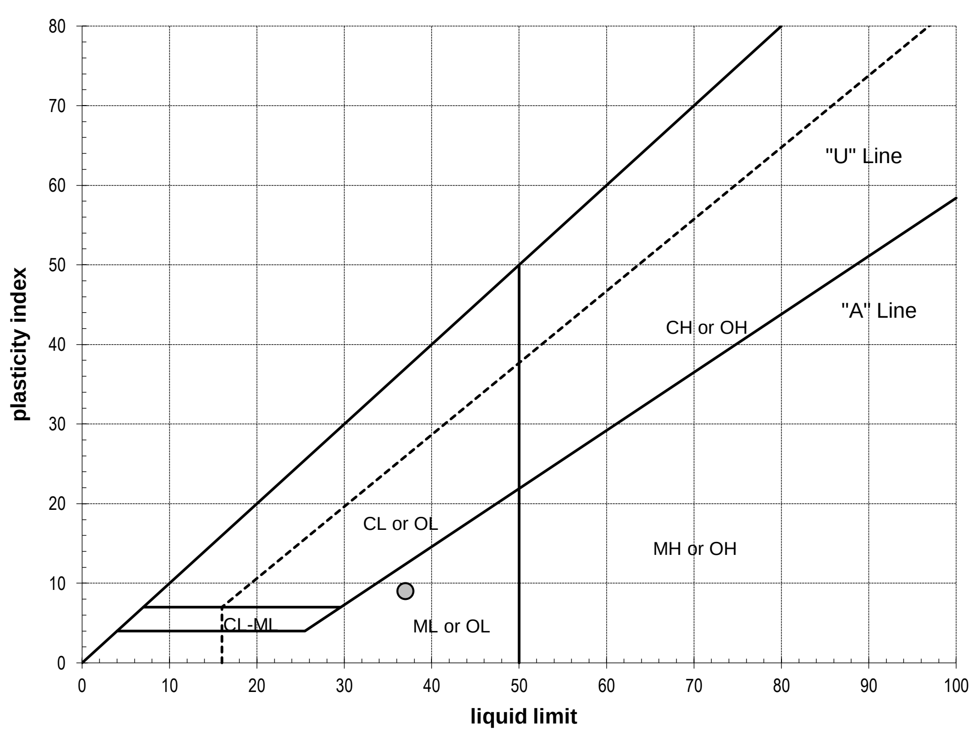
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.09	29.10		
dry soil + pan weight, g =	26.51	27.31		
pan weight, g =	20.80	20.91		
moisture, % =	27.7 %	28.0 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 7.0%
% silt and clay = 93.0%
% silt = n/a
% clay = n/a
moisture content = 38.6%

DATE TESTED	TESTED BY
01/21/22	KMS

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

11917 NE 95TH Street, Vancouver, Washington 98682
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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
La Center Flow Station #1	Clark Public Utilities		21300	TP-1
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**ENGINEER/GEOLOGIST**	**DATE**
La Center, Washington	L&S Contractors	Excavator	CWS	01/03/22
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	148 ft amsl	3 feet bgs	0819	0842

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						FILL. Brown, moist, apparent native disturbed soils with trace organics [Soil Type 1].					
				CL		Brown, moist to wet, medium stiff, lean CLAY with sand [Soil Type 2].					
	TP1.1		A-6(8)			PP=0.75 TSF TV=1.0 TSF	31.6	79.9	33	11	
5											
				ML		Brown/grey, wet, SILT [Soil Type 3].					
10	TP1.2		A-4(10)				38.6	93	37	9	
						Bottom of test pit at 14 feet bgs. Groundwater seeps observed at 3 feet bgs on 01/03/22.					
15											

APPENDIX C
SOIL CLASSIFICATION INFORMATION

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

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

Consistency for Cohesive Soil

CONSISTENCY	SPT N-VALUE (BLOWS PER FOOT)	POCKET PENETROMETER (UNCONFINED COMPRESSIVE STRENGTH, tsf)
Very Soft	2	less than 0.25
Soft	2 to 4	0.25 to 0.50
Medium Stiff	4 to 8	0.50 to 1.0
Stiff	8 to 15	1.0 to 2.0
Very Stiff	15 to 30	2.0 to 4.0
Hard	30 to 60	greater than 4.0
Very Hard	greater than 60	-

Relative Density for Granular Soil

RELATIVE DENSITY	SPT N-VALUE (BLOWS PER FOOT)
Very Loose	0 to 4
Loose	4 to 10
Medium Dense	10 to 30
Dense	30 to 50
Very Dense	more than 50

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.

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing .075 mm)			Silt-Clay Materials (More than 35 Percent Passing 0.075)			
Group Classification	A-1	A-3	A-2	A-4	A-5	A-6	A-7
Sieve analysis, percent passing:							
2.00 mm (No. 10)	-	-	-				
0.425 mm (No. 40)	50 max	51 min	-	-	-	-	-
0.075 mm (No. 200)	25 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)							
Liquid limit				40 max	41 min	40 max	41 min
Plasticity index	6 max	N.P.		10 max	10 max	11 min	11 min
General rating as subgrade	Excellent to good			Fair to poor			

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

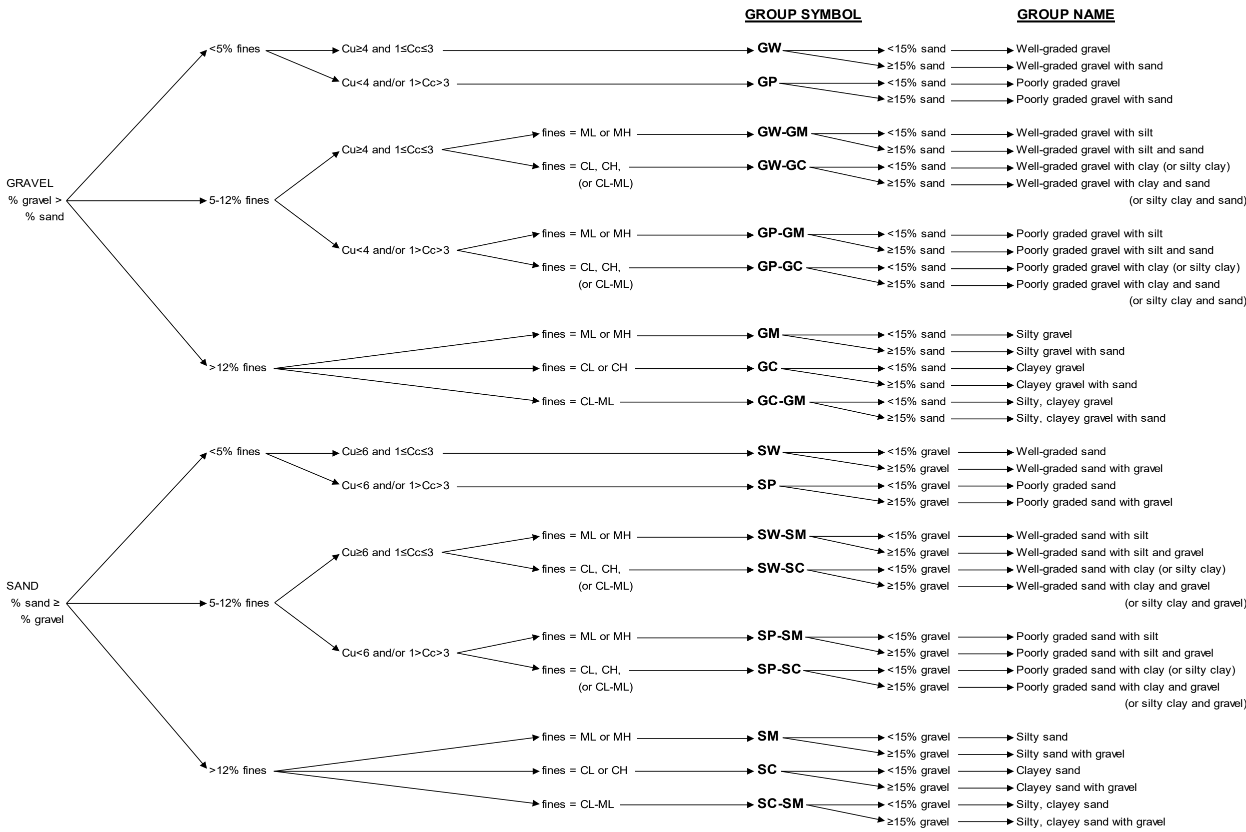
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 mm (No. 10)	50 max	-	-	-	-	-	-	-	-	-	-
0.425 mm (No. 40)	30 max	50 max	51 min	-	-	-	-	-	-	-	-
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11min
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General ratings as subgrade	Excellent to Good							Fair to poor			

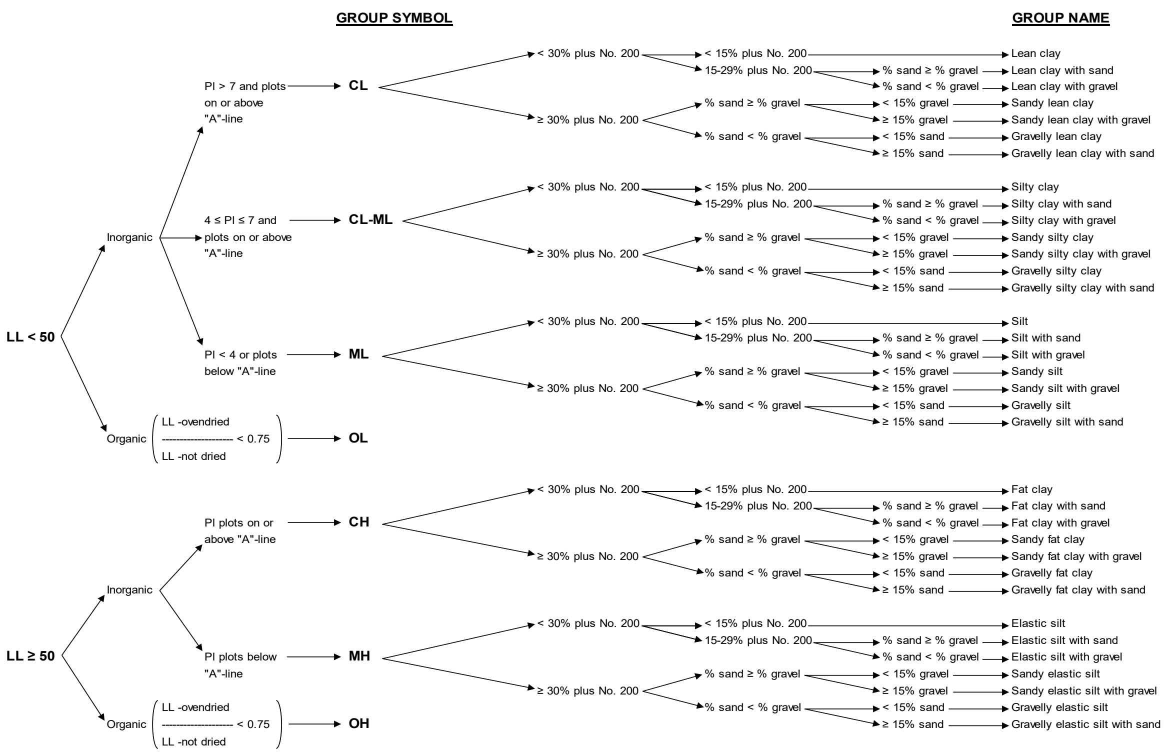
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

USCS 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)

APPENDIX D
PHOTO LOG

LA CENTER FLOW STATION #1

JANUARY, 2022

LA CENTER, WASHINGTON

Central Site View, Facing North

LA CENTER FLOW STATION #1

JANUARY, 2022

LA CENTER, WASHINGTON

Western Site Area, Facing East

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LA CENTER FLOW STATION #1

JANUARY, 2022

LA CENTER, WASHINGTON

Test Pit Profile, TP-1

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Date: March 8, 2022
Project: La Center Flow Station #1
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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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.