Geotechnical Site Investigation

Sage Property

Camas, Washington

January 4, 2021

Geotechnical ▪ Environmental ▪ Special Inspections

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GEOTECHNICAL SITE INVESTIGATION
SAGE PROPERTY
CAMAS, WASHINGTON

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Site Location:	**1811 NW Hood Street** **Parcel Nos. 127415000, 127440000** **Camas, Washington**
Prepared By:	**Columbia West Engineering, Inc.** **11917 NE 95th Street** **Vancouver, Washington 98682** **Phone: 360-823-2900** **Fax: 360-823-2901**
Date Prepared:	**January 4, 2021**

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1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Modern NW, Inc. to conduct a geotechnical site investigation for the proposed Sage Property single-family residential project located in Camas, 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. This report also addresses potential geologic hazard areas in accordance with *Camas Municipal Code, Section 16.59, Geologically Hazardous Areas*. The specific scope of services was outlined in a proposal contract dated August 13, 2020. 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 at 1811 NW Hood Street in Camas, Washington. The site is comprised of tax parcels 127415000 and 127440000 totaling approximately 6.08 acres. The approximate latitude and longitude are N 45° 35’ 30” and W 122° 26’ 37”, and the legal description is a portion of the NE ¼ of Section 09, T1N, R3E, Willamette Meridian. The regulatory jurisdictional agency is the City of Camas, Washington.

1.2 Proposed Development

Correspondence with the client indicates that proposed development includes construction of a single-family residential subdivision with approximately 15 building lots, paved public roadways, underground utilities, and stormwater management facilities. The preliminary site plan is indicated on Figure 2A. Columbia West has not reviewed preliminary grading plans but understands that cut and fill will likely 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 eastern portion of the Portland/Vancouver Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide.

According to the *Geologic Map of the Camas Quadrangle, Clark County, Washington, and Multnomah County, Oregon* (Russell C. Evarts, USGS Geological Survey, 2008), near-surface soils on the eastern portion of the subject site are expected to consist of Holocene-aged, unconsolidated loess deposits of silt and fine sand (Qlo). Mapped QTc exposures on the western portion of the property indicate that loess deposits may be underlain by Pleistocene- to Pliocene-aged, unconsolidated to cemented, pebble- to boulder-sized sedimentary conglomerate.

The *Web Soil Survey* (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2020 Website) identifies surface soils as Powell silt loam. Powell series soils are generally fine-textured clays and silts with low permeability, moderate water capacity, and low shear strength. Powell soils are generally moisture sensitive, somewhat compressible, and described as having low 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.

<u>Portland Hills Fault Zone</u>

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 18 miles west 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 two miles east 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, eight test pits (TP-1 through TP-8), and three infiltration tests was conducted at the site on November 20, 2020. Test pits were explored with a track-mounted excavator. Subsurface soil profiles were 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. Exploration locations are indicated on Figure 2. Subsurface exploration logs are 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 approximate 6.08-acre subject site consists of two tax parcels located at 1811 NW Hood Street in Camas, Washington. The site is bounded by residential development to the north and west, NW 16th Avenue to the south, and NW Hood Street to the east. Observed site structures included an existing single-family home and an agricultural outbuilding located along the eastern property boundary. Site vegetation primarily consisted of grass in open areas with trees and shrubs concentrated around existing structures. Site elevations range from approximately 642 to 730 feet above mean sea level (amsl) respectively between the east and west site boundaries.

Field reconnaissance and review of site topographic mapping indicate rolling to gently sloped terrain with grades of 10 to 15 percent characterizing the site. Slope grades exceed 15 percent in localized areas and are identified as potential landslide hazards according to *Clark County Maps Online*. Discussion related to slope geometry, geomorphic features, and stability are discussed later in Section 5.0, *Geologically Hazardous Areas*.

4.2 Subsurface Exploration and Investigation

Test pits were explored to a maximum depth of 14 feet below ground surface (bgs). Exploration locations were selected to observe subsurface soil characteristics in proximity to proposed development areas and are indicated on Figure 2.

4.2.1 Soil Type Description

The field investigation indicated the presence of approximately 12 to 16 inches of sod and topsoil in the observed locations. Underlying the topsoil layer, subsurface soils resembling native USDA Powell soil series descriptions were encountered. Subsurface lithology was reasonably consistent at explored locations and may generally be described by soil types identified in the following text. Detailed 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 represents existing fill and was observed to consist of tan to dark brown, moist to wet, medium stiff silt. Soil Type 1 was observed below the topsoil layer in test pit TP-5 and extended to an observed depth of 5 feet bgs. Additional discussion and recommendations pertaining to Soil Type 1 are discussed in Section 6.1.1, *Existing Fill.*

Soil Type 2 – Lean CLAY

Soil Type 2 was observed to primarily consist of tan to brown moist to wet, medium stiff to stiff lean CLAY. Soil Type 2 was observed below Soil Type 1 in test pit TP-5 and below the topsoil layer in all other test pits. Soil Type 2 extended to observed depths of 6.5 to 13 feet bgs where it was typically underlain by Soil Type 3.

Analytical laboratory testing conducted upon representative soil samples obtained from test pits TP-1, TP-3, and TP-6 indicated approximately 85 to 87 percent by weight passing the No. 200 sieve and in situ moisture contents ranging from 25 to 30 percent. Atterberg Limits analysis conducted on tested samples of Soil Type 2 indicated liquid limits ranging from 31 to 41 percent and plasticity indices ranging from 11 to 21 percent. The laboratory tested samples of Soil Type 2 are classified CL according to USCS specifications and A-6(8), A-6(16), and A-7-6(18) according to AASHTO specifications.

Soil Type 3 – Sandy Elastic SILT

Soil Type 3 was observed to primarily consist of brown to orange/red-brown, moist to wet, medium stiff to stiff sandy elastic SILT. Portions of the soil type contained trace to some subrounded gravels, cobbles, and boulders which may represent initial transition from unconsolidated regolith to mapped sedimentary conglomerate (Evarts, 2008). With the exception of TP-3, Soil Type 3 was observed below Soil Type 2 in all test pit explorations and extended to the maximum depth of exploration.

Analytical laboratory testing conducted upon a representative soil sample obtained from test pit TP-7 indicated approximately 53 percent by weight passing the No. 200 sieve and an in situ moisture content of approximately 42 percent. Atterberg Limits analysis indicated a liquid limit of 58 percent and a plasticity index of 25 percent. The laboratory tested sample of Soil

Type 3 is classified MH according to USCS specifications and A-7-5(11) according to AASHTO specifications.

4.2.2 Groundwater

Groundwater seeps and springs were observed within test pit explorations TP-3 through TP-8 at depths ranging from 2 to 8 feet bgs. Review of nearby well logs obtained from the State of Washington Department of Ecology indicates that static groundwater levels in the area may vary significantly. Variations in ground water elevations likely reflect the screened interval depth of these wells, changes in ground surface elevation, and the presence of multiple aquifers and confining units. Mitigation of shallow groundwater within proposed development areas is discussed in greater detail in Section 6.8, *Dewatering* and Section 6.12, *Drainage*.

Groundwater levels are often subject to seasonal variance and may rise during extended periods of increased precipitation or flooding. Perched groundwater may also be present in localized areas. 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.

5.0 GEOLOGICALLY HAZARDOUS AREAS

Camas Municipal Code, Section 16.59 defines geologic hazard requirements for proposed development in areas subject to City of Camas jurisdiction. Three potential geologic hazards are identified: (1) erosion hazard areas, (2) landslide hazard areas, and (3) seismic hazard areas. As previously indicated, hazard mapping obtained from *Clark County Maps Online* indicates potential landslide hazard areas (slopes greater than 15 percent) within portions of the property.

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

5.1 Erosion Hazards

Camas Municipal Code, Section 16.59.020.A defines an erosion hazard as areas where slope grades meet or exceed 40 percent. Based upon review of slope grade mapping published by *Clark County Maps Online,* maximum slope grades of 15 to 25 percent are mapped in the central and western portions of the site. Therefore, site slopes do not meet the definition of an erosion hazard according to *Camas Municipal Code*.

5.2 Landslide Hazards

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

5.2.1 Geologic Literature Review

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

Columbia West also reviewed the *Geologic Map of the Camas Quadrangle, Clark County, Washington, and Multnomah County, Oregon* (Russell C. Evarts, USGS Geological Survey, 2008) and the *Landslide Inventory Map of the Northwest Quarter of the Camas Quadrangle, Multnomah County, Oregon, and Clark County, Washington* (William Burns, et al., 2012) which indicates that no active landslides or historic landslide deposits are mapped at the subject site or in the surrounding vicinity.

5.2.2 Slope Reconnaissance and Slope Stability Assessment

To observe geomorphic conditions, Columbia West personnel conducted visual and physical reconnaissance of slopes on the property. Test pits TP-1 through TP-8 were explored in sloped areas. Subsurface native soils at the locations tested generally consisted of medium stiff to stiff lean clay and sandy elastic silt with trace to some gravels, cobbles, and boulders. Soil horizons appeared firm and well developed.

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 east to west with no apparent toe or crest observed on the property or adjacent parcels. The maximum grade change between the east and west property boundaries is approximately 88 feet. Slope grades of 10 to 15 percent characterize the property with localized areas approaching 15 to 25 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 explored site soils and groundwater seeps or springs within the face of the slopes were not observed.

Camas Municipal Code defines a landslide hazard as slopes mapped by Fiksdal as ‘areas of potential instability’ or areas meeting all three of the following characteristics: 1) slopes steeper than 15 percent; 2) hillsides intersecting geologic contacts with permeable sediment overlying low permeability sediment or bedrock, and; 3) any springs or groundwater seepage. The above-mentioned criteria were not observed during our field investigation or site research. Based upon the results of slope reconnaissance, subsurface exploration, and site research, slopes on the subject site do not appear to meet the definition of a landslide hazard according to *Camas 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 site is mapped as very low susceptibility for liquefaction. Liquefaction, defined as the transformation of the behavior of a granular material from a solid to a liquid due to increased pore-water pressure and reduced effective stress, may occur when granular materials quickly compact under cyclic stresses caused by a seismic event. The effects of liquefaction may include immediate ground settlement, lateral spreading, and differential compaction.

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

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

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 may be represented by Site Class C as defined in *2015 IBC Section 1613.3.2*. A designation of Site Class C indicates that minor amplification of seismic energy may occur during a seismic event due to subsurface conditions. However, this is typical for many areas within Clark County, does not represent a geologic hazard in Columbia West's opinion, and will not prohibit development if properly accounted for during the design process. Additional seismic information is presented in Section 6.10, *Seismic Design Considerations*.

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 existing fill, drainage, shallow groundwater, and fine-textured soil. 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 sod and highly organic topsoil is anticipated to vary between approximately 12 and 16 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 foundations, basement walls, utilities, associated soft soils, and debris. Excavation areas should be backfilled with engineered structural fill.

Test pits excavated during site exploration were backfilled loosely with onsite soils. These test pits 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 *2015 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.

6.1.1 Existing Fill

As previously discussed and indicated on Figure 2, existing fill was observed within test pit TP-5 and extended to an observed depth of 5 feet bgs. Observed fill material generally consisted of tan to dark brown, moist to wet, medium stiff silt. As presented in Appendix D, *Photo Log*, review of 1998 aerial imagery published by *Clark County Maps Online* indicates previous site disturbance and potential earthwork activity in the vicinity of test pit TP-5.

Existing fill and other previously disturbed soils or debris are not suitable for bearing structures in their current state and should be removed completely and thoroughly from structural areas. In some areas, existing fill may directly overlie vegetation and the original topsoil layer. This material should also be removed completely. Upon removal of existing fill, Columbia West should observe the exposed subgrade to verify adequate support conditions.

Based upon Columbia West's investigation, existing fill soils as described appear to be acceptable for reuse as structural fill, provided materials are observed to exhibit index properties similar to those observed during this investigation and that construction adheres to the specifications presented in this report. Portions of existing fill found to contain highly organic soils, debris, or other deleterious material should be removed. Note that the limited scope of exploration conducted for this investigation cannot wholly eliminate uncertainty regarding the presence of unsuitable soils in areas not explored. Final recommendations regarding the suitability of reusing existing fill soils as structural fill material should be provided in the field by Columbia West during construction.

6.2 Engineered Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in the preceding text. Surface soils should then 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 standard Proctor moisture-density relationship test (ASTM D698), is recommended for structural fill placement. Engineered structural fill placed on sloped grades should be benched to provide a horizontal surface for compaction.

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 (Soil Types 2 and 3) may be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native soils with a plasticity index greater than 25, if encountered, should be evaluated and approved by Columbia West prior to use as structural fill. Boulders and large cobbles exceeding approximately six inches in diameter should be removed from proposed native fill soils prior to placement. Native soils may require addition of moisture during periods of dry weather. Compacted fill soils should be covered shortly after placement.

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 drain pipe 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 20 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

Residential foundations are anticipated to consist of shallow continuous perimeter or column spread footings. 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. Footings should be designed by a licensed structural engineer and conform to the recommendations below.

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 (Soil Types 2 or 3) 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 existing fill or disturbed soil (Soil Type 1). 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 residential structures may have slab-on-grade floors. Slabs should be supported on firm, competent, in situ soil (Soil Types 2 or 3) or engineered structural fill. Disturbed soils and unsuitable fills in proposed slab locations should be removed and replaced with structural fill.

Preparation and compaction 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. For lightly loaded slabs not exceeding 200 psf, the modulus of subgrade reaction is estimated to be 100 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. Explosive blasting is not anticipated, however, difficult excavation conditions associated with bouldery or cemented soils will require appropriately-sized equipment and potential specialized excavation techniques to construct site improvements.

Groundwater seeps and springs were encountered within test pit explorations TP-3 through TP-8 at depths ranging from 2 to 8 feet below ground surface. Recommendations as 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 in no case should excavation be conducted in excess of 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

If retaining walls are proposed, lateral earth pressures should be carefully considered in the design process. Hydrostatic pressure and additional surcharge loading should also be considered. 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. 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 (Soil Type 2)	61 pcf	42 pcf	319 pcf	115 pcf	28°
Undisturbed native Sandy Elastic SILT (Soil Type 3)	62 pcf	42 pcf	346 pcf	120 pcf	29°
Approved Structural Backfill Material WSDOT 9-03.12(2) compacted aggregate backfill	52 pcf	32 pcf	568 pcf	135 pcf	38°

**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 for the site, 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. If seismic design is required for restrained walls, seismic forces may be calculated by superimposing a uniform lateral force of $25H^2$ pounds per lineal foot of wall. 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 drain pipe 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 6.12, *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	0.397 g
0.2 sec Spectral Acceleration	0.922 g
1.0 sec Spectral Acceleration	0.382 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 Fa, Fv, F_{PGA} as defined in *ASCE 7-10, 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. Seismic site class was discussed previously in Section 5.3, *Seismic Hazard Areas*.

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 *2015 IBC*, the potential for peak ground accelerations in excess of the adjusted and amplified values should be understood.

6.11 Infiltration Testing Results and Hydrologic Soil Group Classification

To facilitate design of stormwater management infrastructure and classify tested soils into a representative hydrologic soil group, Columbia West conducted in situ infiltration testing within test pits TP-1 through TP-3 at a depth of approximately two feet bgs. Results of in situ infiltration testing are presented in Table 3. Infiltration rates are presented as a coefficient of permeability (k) and have been reported without application of a factor of safety.

Table 3. Infiltration Test Results and Hydrologic Soil Group Classifications

Test Number	Location	Approximate Test Depth (feet bgs)	Approximate Depth to Groundwater on 11-20-20 (feet bgs)	USCS Soil Type (*Indicates Visual Classification)	Clark County WWHM Soil Group**	Passing No. 200 Sieve (%)	Infiltration Rate (Coefficient of Permeability, k) (inches/hour)
IT-1.1	TP-1	2.0	Not observed to 11.0	CL, Lean CLAY	4	86.8	**< 0.06**
IT-2.1	TP-2	2.0	Not observed to 13.0	CL, Lean CLAY*	4	--	**< 0.06**
IT-3.1	TP-3	2.0	2.0	CL, Lean CLAY	4	86.1	**< 0.06**

** WWHM classifications are based upon subsurface investigation and infiltration testing conducted at the locations indicated.

Single-ring, falling head infiltration tests were performed by inserting standpipes into the soil at the noted depths, filling the pipes with water, and measuring time relative to changes in hydraulic head. Using Darcy's Law for saturated flow in homogenous media, the coefficient of permeability (k) was then calculated. Soils in the tested locations were observed and sampled where appropriate to adequately characterize the subsurface profile. Tested native soils are classified as lean CLAY (CL) according to USCS specifications.

Columbia West classified tested near-surface soils within test pits TP-1 through TP-3 into representative soil groups based upon site-specific infiltration test results and review of published literature. As indicated in Table 3, observed near-surface infiltration rates were less than 0.06 inches per hour in the tested locations. Based upon review of USDA hydrologic soil group criteria (USDA, 2007), Appendix 2-A of the *2015 Clark County Stormwater Manual,* and the *Clark County WWHM Soil Groupings Memorandum* (Otak, 2010), measured infiltration rates generally meet the criteria for WWHM Soil Group 4. Therefore, based upon site-specific infiltration testing and review of published literature, tested near-surface soils in the locations of TP-1 through TP-3 may be appropriately classified as presented in Table 3.

Due to the presence of shallow groundwater and fine-textured, low permeability soils at the site, subsurface disposal of concentrated stormwater is likely infeasible and is not recommended without further study.

6.12 Drainage

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. Drainage design in general should conform to City of Camas regulations. Finished site grading should be conducted with positive drainage away from structures. 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 drain pipe 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.

6.13 Bituminous Asphalt and Portland Cement Concrete

Review of Figure 2A indicates that proposed development will include new asphalt-paved public roadways. Unless a site-specific pavement design is conducted, Columbia West recommends adherence to City of Camas 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 6.14, *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.

Crushed aggregate base should be compacted and tested in accordance with the specifications outlined 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 Washington Department of Transportation and City of Camas specifications.

Portland cement concrete curbs and sidewalks should be installed in accordance with City of Camas specifications. Curb and sidewalk aggregate base should be observed and proof-rolled by Columbia West. 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.

6.14 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.

Crushed aggregate base should 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. 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.

6.15 Erosion Control Measures

As indicated previously in Section 5.1, *Erosion Hazards*, 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.

6.16 Soil Shrink/Swell Potential

Based upon laboratory analysis, tested near-surface soils contain as much as 87 percent by weight passing the No. 200 sieve and exhibit a plasticity index ranging from 11 to 25 percent. This indicates the potential for soil shrinking or swelling and underscores the importance of proper moisture conditioning during fill placement. Medium to high plasticity soils, if approved for use as structural fill, should be placed and compacted at a moisture content approximately two percent above optimum as determined by laboratory analysis.

6.17 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

1-4-21

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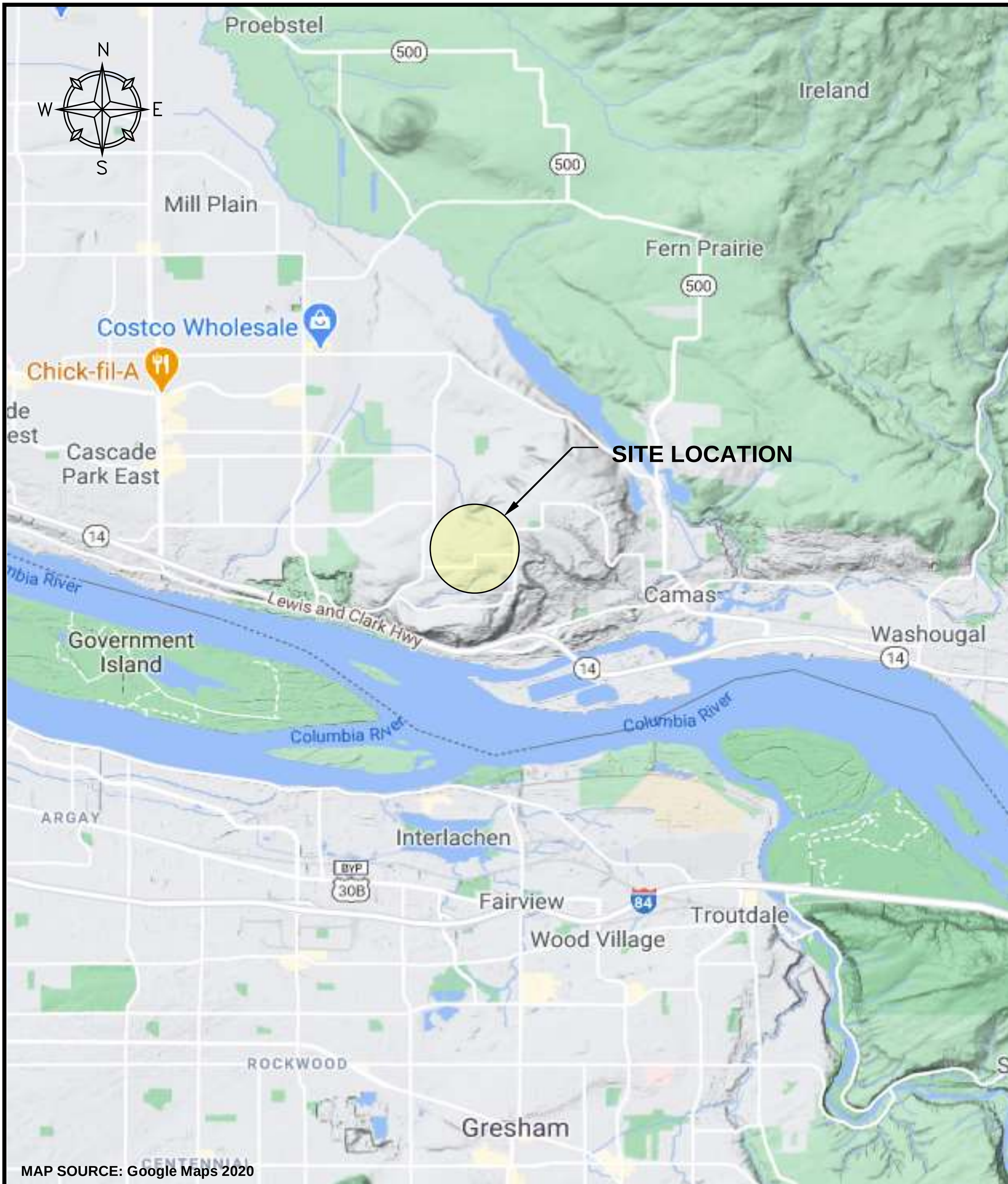
FIGURES



Geotechnical • Environmental • Special Inspections Columbia West Engineering, Inc	Job No: 20287 Date: 12/21/20 Drawn: GLW Checked: LVL	SITE LOCATION MAP SAGE PROPERTY CAMAS, WASHINGTON	FIGURE 1

Infiltration Test Results

Test Number	Location	Approximate Test Depth (feet bgs)	Groundwater Depth on 11-20-20 (feet bgs)	USCS Soil Type (*Visual Classification)	Clark County WWHM Soil Group	Passing No. 200 Sieve (%)	Infiltration Rate (Coefficient of Permeability, k) (inches/hour)
IT-1.1	TP-1	2.0	Not observed to 11.0	CL, Lean CLAY	4	86.8	< 0.06
IT-2.1	TP-2	2.0	Not observed to 13.0	CL, Lean CLAY*	4	-	< 0.06
IT-3.1	TP-3	2.0	2.0	CL, Lean CLAY	4	86.1	< 0.06

APPROXIMATE LOCATION OF TEST PIT EXPLORATION

APPROXIMATE LOCATION OF INFILTRATION TEST

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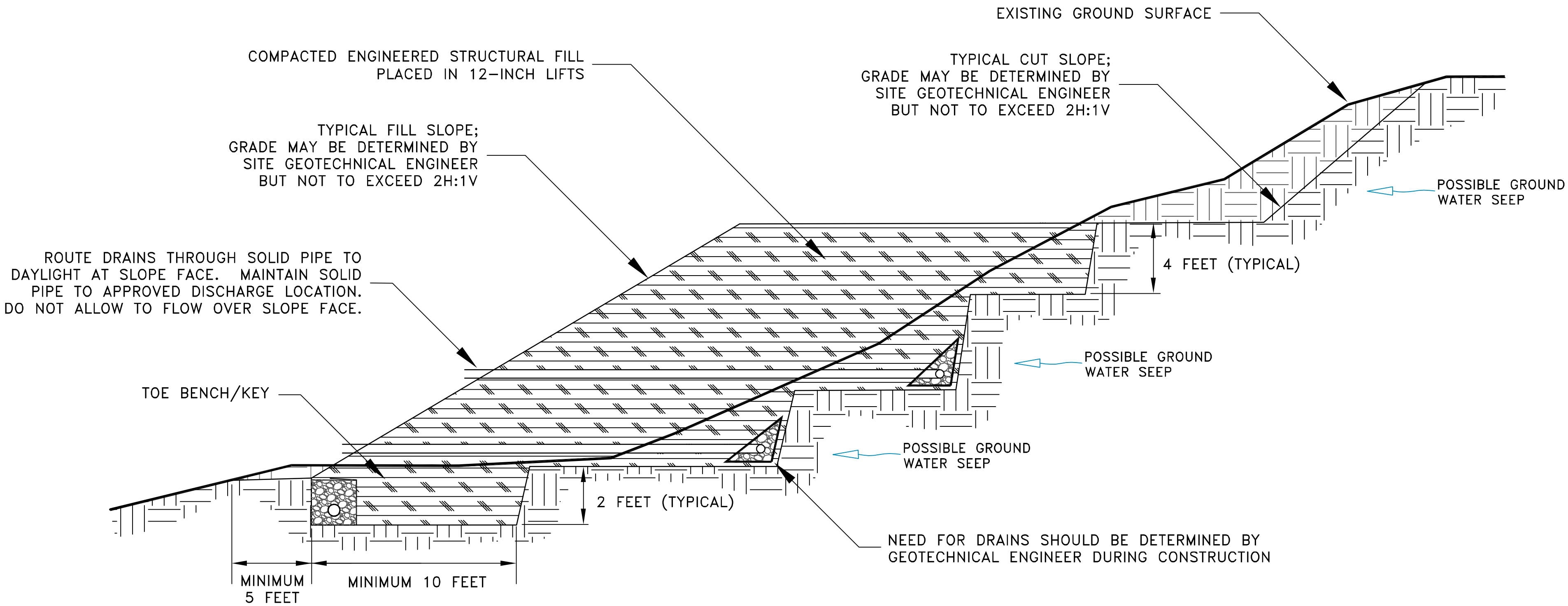
Date: 12/21/20
Drawn: GLW
Checked: LVL

EXPLORATION LOCATION MAP
SAGE PROPERTY

NOTES:
1. SITE LOCATION: 1811 NW HOOD STREET, CAMAS, WASHINGTON.
2. SITE CONSISTS OF TAX PARCELS 127415000 AND 127440000 TOTALING APPROXIMATELY 6.08 ACRES.
3. AERIAL PHOTO SOURCED FROM CLARK COUNTY GIS, 2020.
4. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
5. TEST PIT EXPLORATIONS BACKFILLED LOOSELY WITH ONSITE SOILS ON NOVEMBER 20, 2020.

FIGURE 2

Geotechnical ▪ Environmental ▪ Special Inspections Columbia West Engineering, Inc	Job No: 20287 Date: 12/21/20 Drawn: GLW Checked: LVL	PRELIMINARY SITE PLAN SAGE PROPERTY	NOTES: 1. SITE PLAN PROVIDED BY OLSON ENGINEERING, INC.	FIGURE 2A

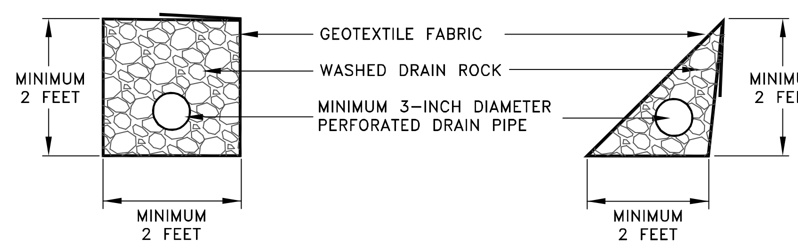


DRAIN SPECIFICATIONS

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.

TYPICAL DRAIN SECTION DETAIL



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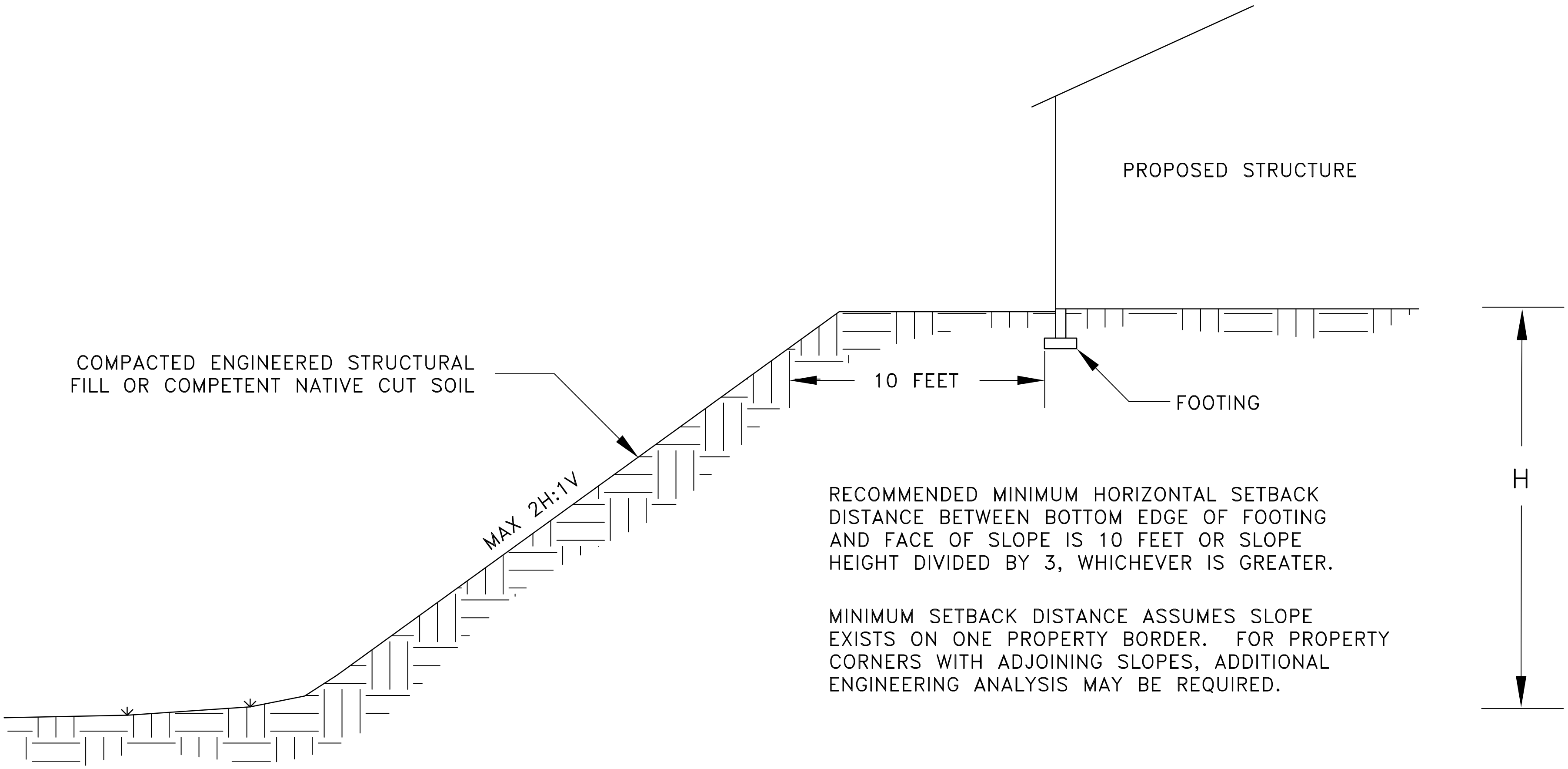
TYPICAL CUT AND FILL
SLOPE CROSS SECTION

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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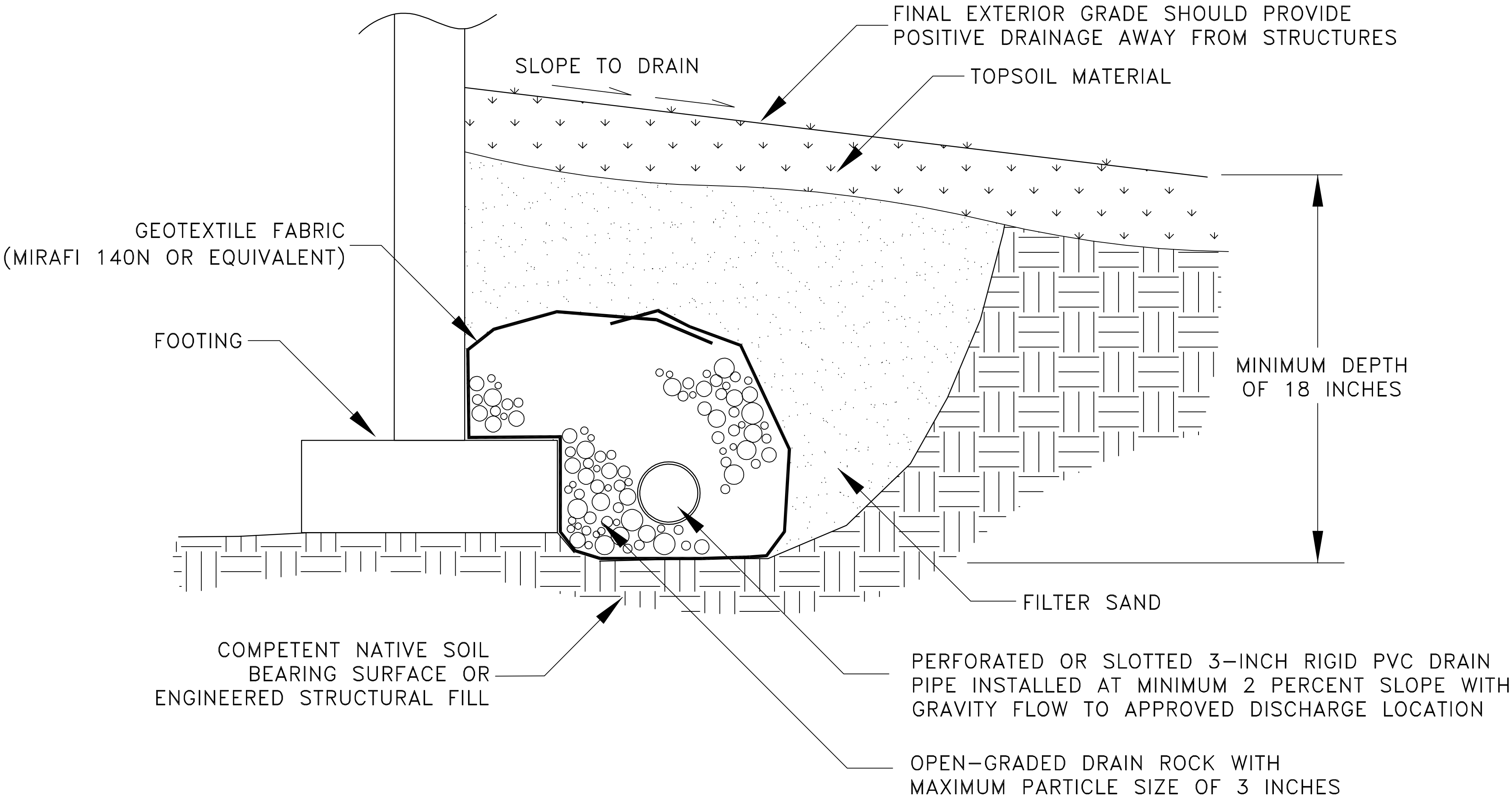
TYPICAL MINIMUM FOUNDATION SLOPE SETBACK DETAIL

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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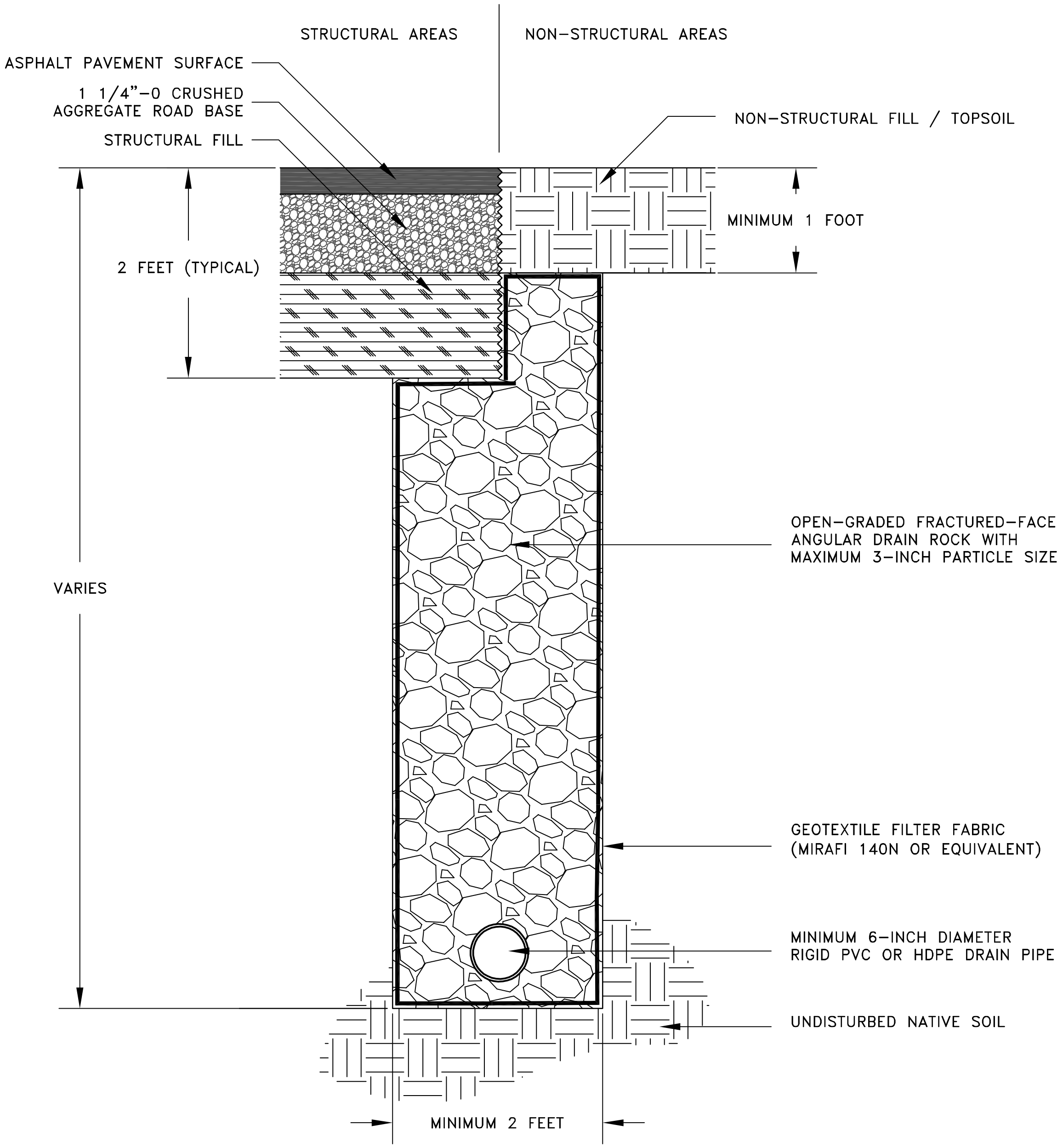
TYPICAL PERIMETER FOOTING DRAIN DETAIL

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.

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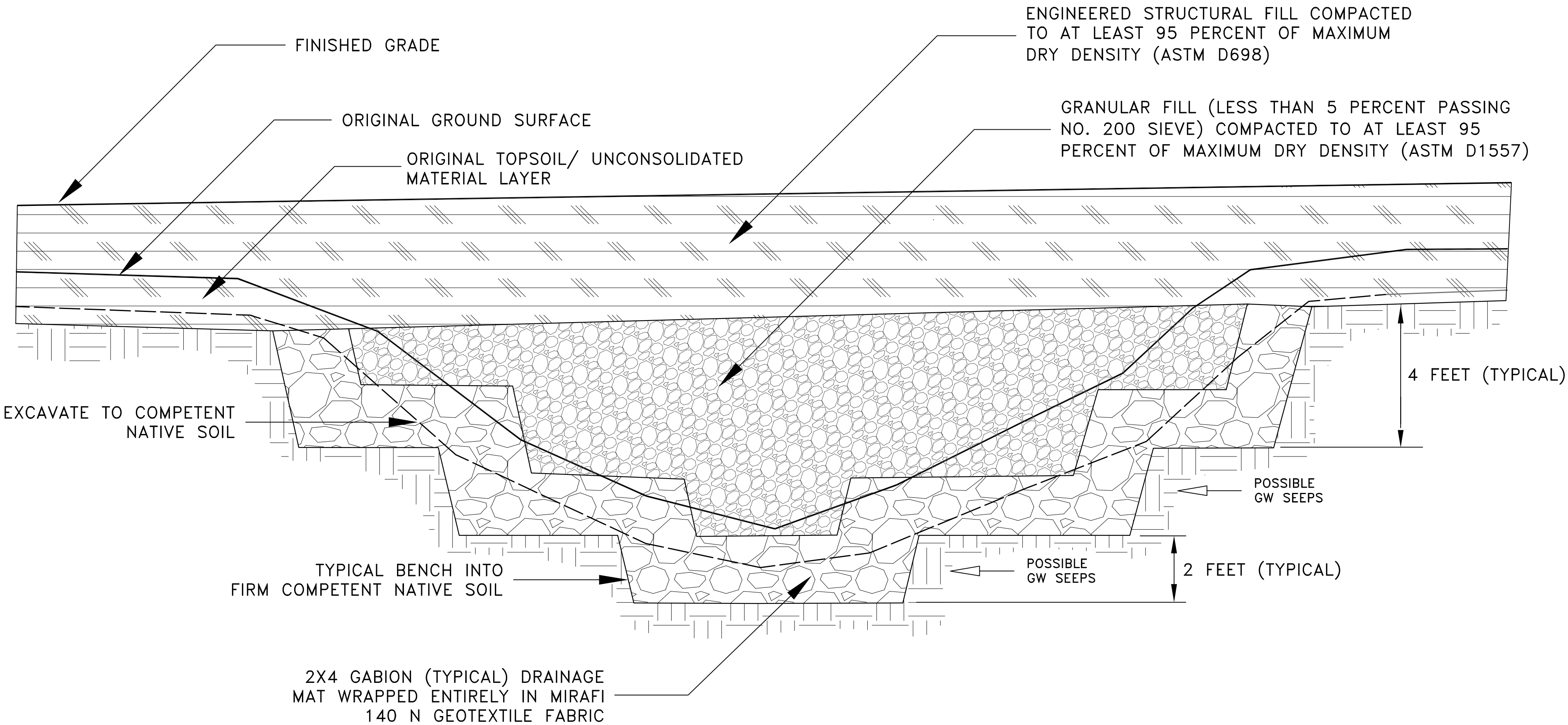
TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL

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



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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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Phone: 360-823-2900, Fax: 360-823-2901
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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sage Property Camas, Washington	Modern NW, Inc. 8101 NE Glisan Street Portland, Oregon 97213	20287	S20-1663
		REPORT DATE: 12/08/20	FIELD ID: TP1.1
		DATE SAMPLED: 11/20/20	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Test Pit TP-01 depth = 2 feet	CL, Lean Clay

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-6(16)

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

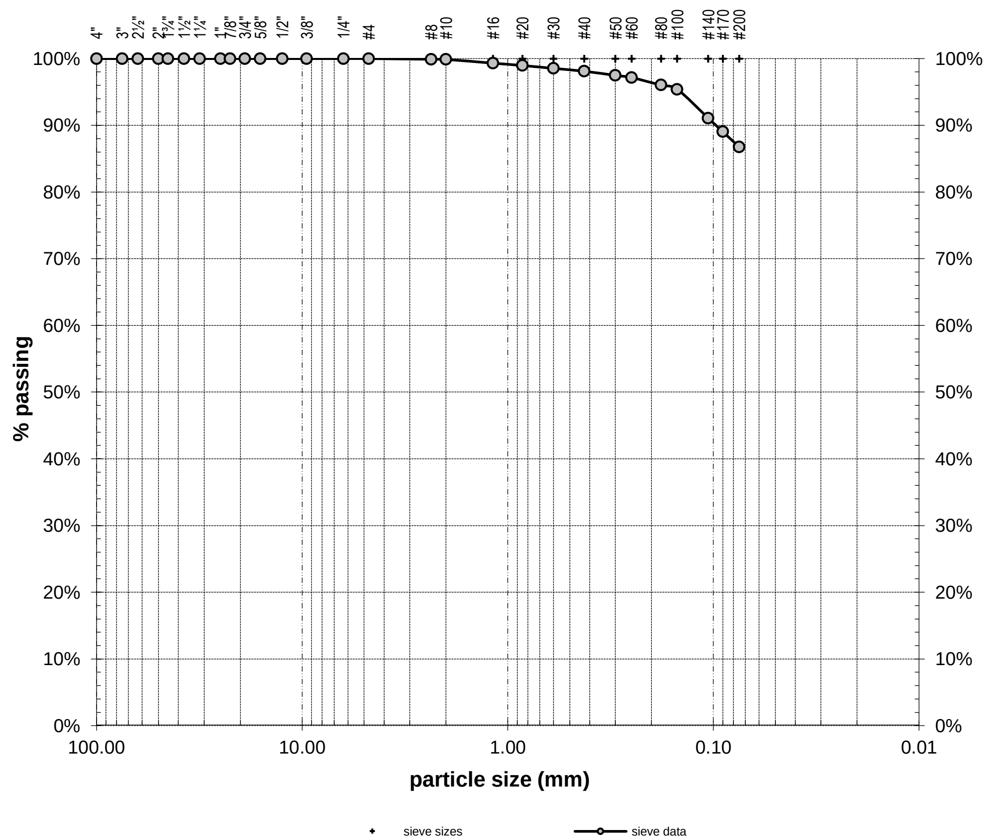
initial dry mass (g) =	138.17		
as-received moisture content =	24.0%	coefficient of curvature, C_C =	n/a
liquid limit =	38	coefficient of uniformity, C_U =	n/a
plastic limit =	20	effective size, $D_{(10)}$ =	n/a
plasticity index =	18	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 13.2%
% silt and clay = 86.8%

	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		99%		
	#20	0.850	99%			
	#30	0.600		99%		
	#40	0.425	98%			
	#50	0.300		97%		
	#60	0.250	97%			
	#80	0.180		96%		
	#100	0.150	95%			
	#140	0.106		91%		
	#170	0.090		89%		
	#200	0.075	87%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
11/30/20	BTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sage Property Camas, Washington	Modern NW, Inc. 8101 NE Glisan Street Portland, Oregon 97213	20287	S20-1663
		REPORT DATE: 12/08/20	FIELD ID: TP1.1
		DATE SAMPLED: 11/20/20	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Test Pit TP-01 depth = 2 feet	CL, Lean Clay

LABORATORY TEST DATA

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

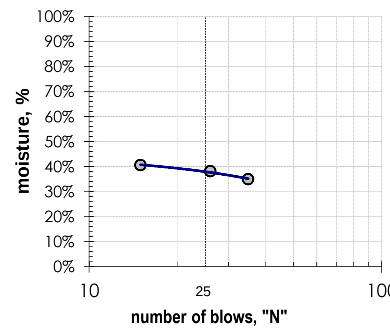
ATTERBERG LIMITS

liquid limit = 38
plastic limit = 20
plasticity index = 18

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	30.58	32.22	33.02	
dry soil + pan weight, g =	28.08	29.09	29.55	
pan weight, g =	20.91	20.89	20.98	
N (blows) =	35	26	15	
moisture, % =	34.9 %	38.2 %	40.5 %	

LIQUID LIMIT



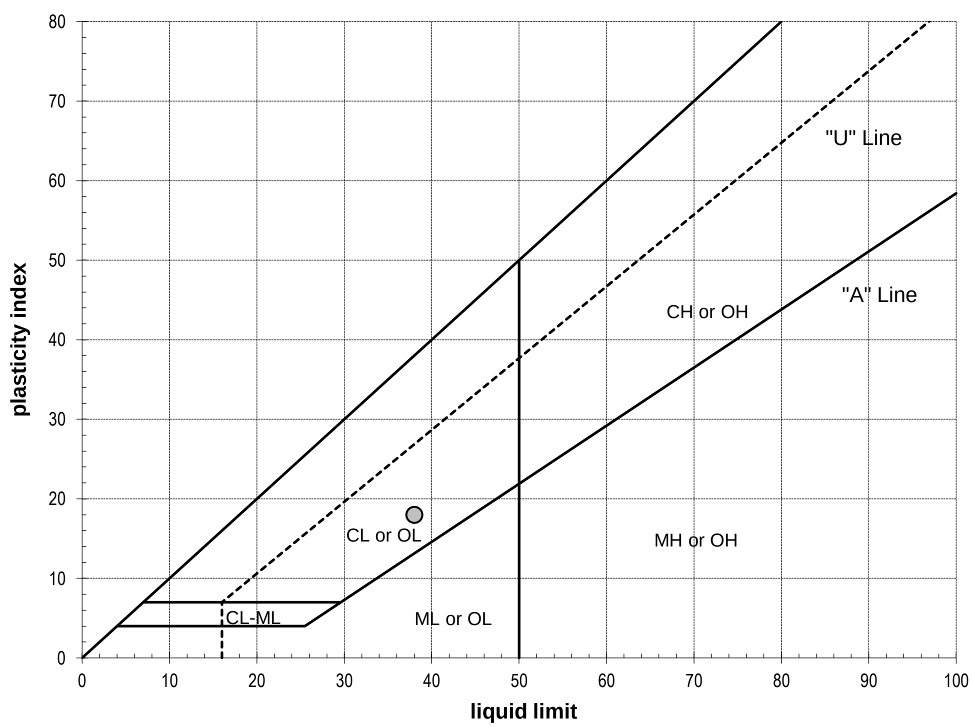
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.42	27.27		
dry soil + pan weight, g =	26.31	26.17		
pan weight, g =	20.93	20.74		
moisture, % =	20.6 %	20.3 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 13.2%
% silt and clay = 86.8%
% silt = n/a
% clay = n/a
moisture content = 24.0%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sage Property Camas, Washington	Modern NW, Inc. 8101 NE Glisan Street Portland, Oregon 97213	20287	S20-1664
		REPORT DATE: 12/08/20	FIELD ID: TP3.1
		DATE SAMPLED: 11/20/20	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Test Pit TP-03 depth = 2 feet	CL, Lean Clay

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-7-6(18)

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

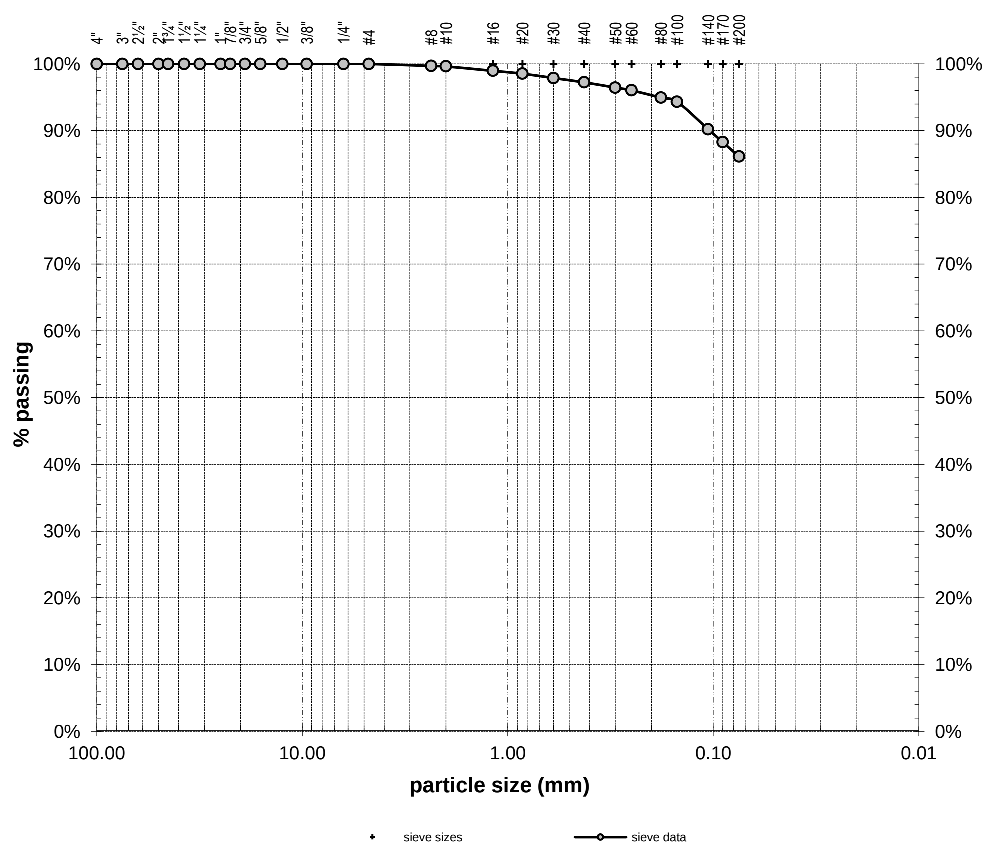
initial dry mass (g) =	143.87		
as-received moisture content =	29.7%	coefficient of curvature, C_C =	n/a
liquid limit =	41	coefficient of uniformity, C_U =	n/a
plastic limit =	20	effective size, $D_{(10)}$ =	n/a
plasticity index =	21	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 13.9%
% silt and clay = 86.1%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	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		99%		
	#20	0.850	99%			
	#30	0.600		98%		
	#40	0.425	97%			
	#50	0.300		96%		
	#60	0.250	96%			
	#80	0.180		95%		
	#100	0.150	94%			
	#140	0.106		90%		
	#170	0.090		88%		
	#200	0.075	86%			

GRAIN SIZE DISTRIBUTION



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sage Property Camas, Washington	Modern NW, Inc. 8101 NE Glisan Street Portland, Oregon 97213	20287	S20-1664
		REPORT DATE: 12/08/20	FIELD ID: TP3.1
		DATE SAMPLED: 11/20/20	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Test Pit TP-03 depth = 2 feet	CL, Lean Clay

LABORATORY TEST DATA

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

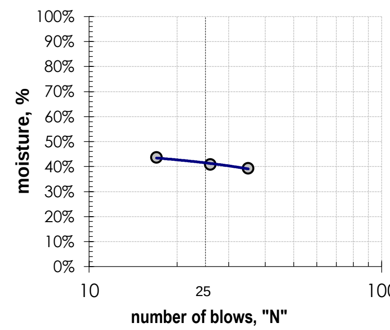
ATTERBERG LIMITS

liquid limit = 41
plastic limit = 20
plasticity index = 21

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.32	31.24	31.63	
dry soil + pan weight, g =	28.38	28.24	28.44	
pan weight, g =	20.90	20.89	21.13	
N (blows) =	35	26	17	
moisture, % =	39.3 %	40.8 %	43.6 %	

LIQUID LIMIT



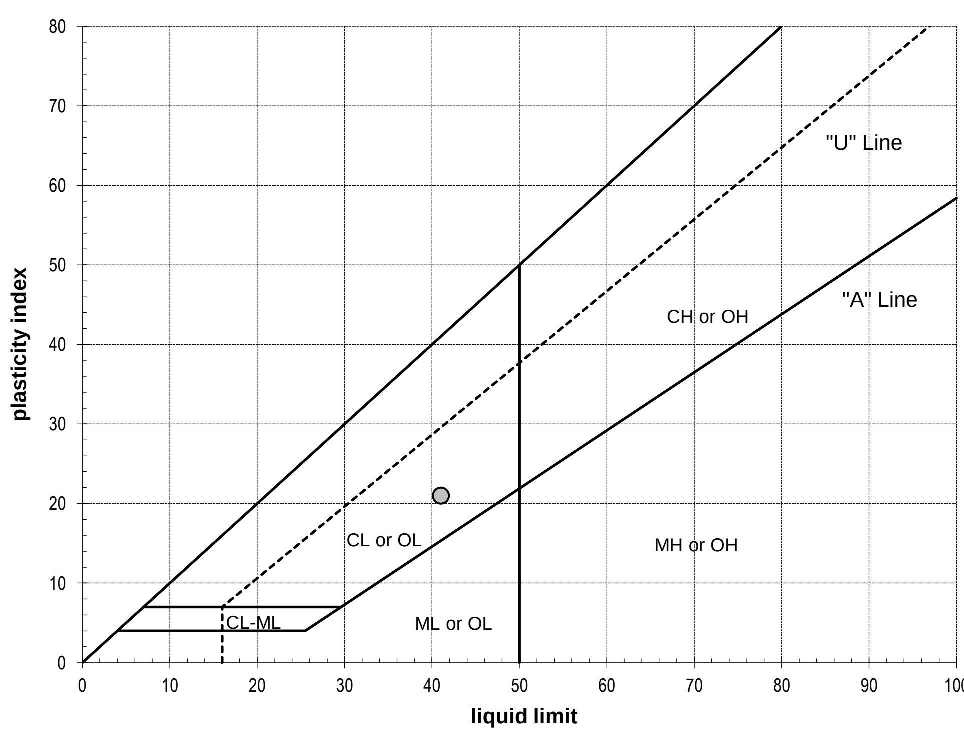
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.12	27.46		
dry soil + pan weight, g =	26.02	26.33		
pan weight, g =	20.61	20.80		
moisture, % =	20.3 %	20.4 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 13.9%
% silt and clay = 86.1%
% silt = n/a
% clay = n/a
moisture content = 29.7%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sage Property Camas, Washington	Modern NW, Inc. 8101 NE Glisan Street Portland, Oregon 97213	20287	S20-1665
		REPORT DATE: 12/08/20	FIELD ID: TP6.1
		DATE SAMPLED: 11/20/20	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Test Pit TP-06 depth = 5 feet	CL, Lean Clay

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

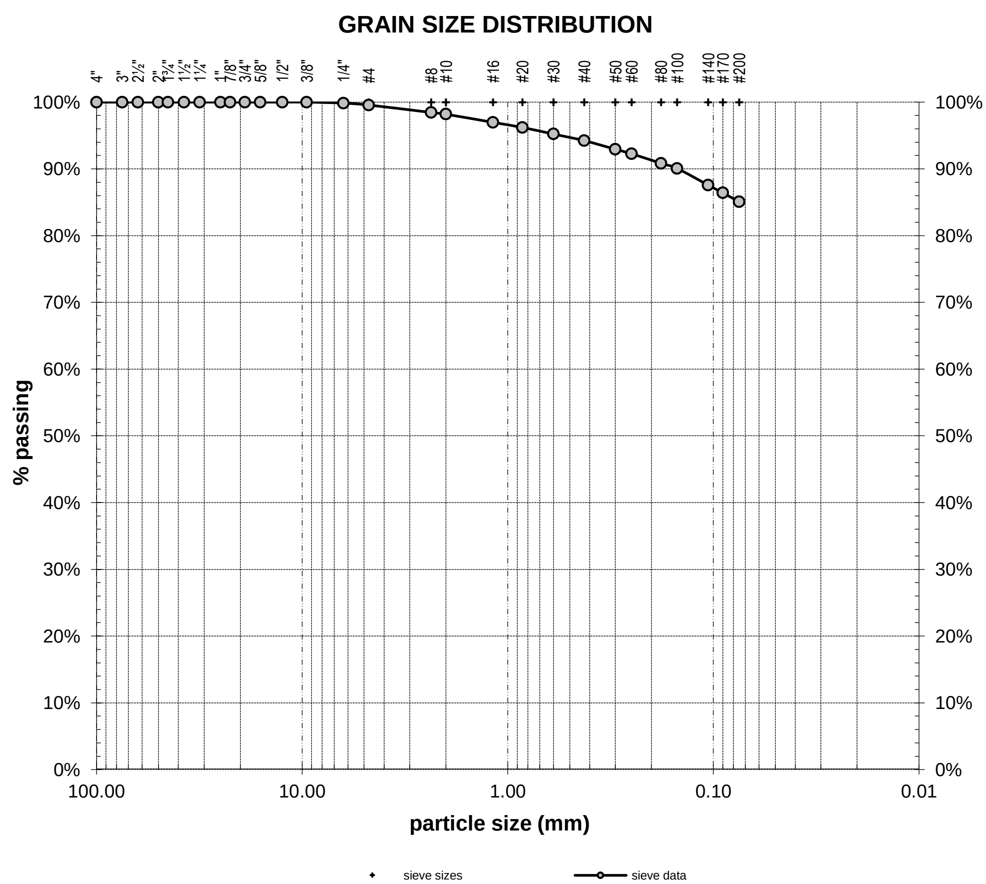
ADDITIONAL DATA

initial dry mass (g) =	163.31		
as-received moisture content =	25.3%	coefficient of curvature, C_C =	n/a
liquid limit =	31	coefficient of uniformity, C_U =	n/a
plastic limit =	20	effective size, $D_{(10)}$ =	n/a
plasticity index =	11	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.4%
% sand = 14.5%
% silt and clay = 85.1%

	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		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		98%		
	#10	2.00	98%			
	#16	1.18		97%		
	#20	0.850	96%			
	#30	0.600		95%		
	#40	0.425	94%			
	#50	0.300		93%		
	#60	0.250	92%			
	#80	0.180		91%		
	#100	0.150	90%			
	#140	0.106		88%		
	#170	0.090		86%		
	#200	0.075	85%			



DATE TESTED	TESTED BY
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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sage Property Camas, Washington	Modern NW, Inc. 8101 NE Glisan Street Portland, Oregon 97213	20287	S20-1665
		REPORT DATE: 12/08/20	FIELD ID: TP6.1
		DATE SAMPLED: 11/20/20	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Test Pit TP-06 depth = 5 feet	CL, Lean Clay

LABORATORY TEST DATA

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

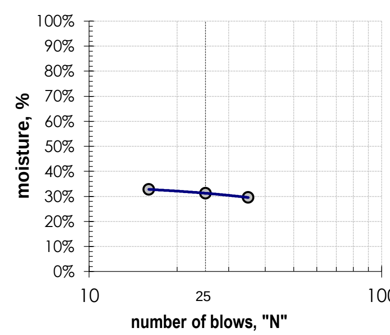
ATTERBERG LIMITS

liquid limit =	31
plastic limit =	20
plasticity index =	11

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	35.61	33.97	31.62	
dry soil + pan weight, g =	32.29	30.83	28.90	
pan weight, g =	21.05	20.79	20.60	
N (blows) =	35	25	16	
moisture, % =	29.5 %	31.3 %	32.8 %	

LIQUID LIMIT



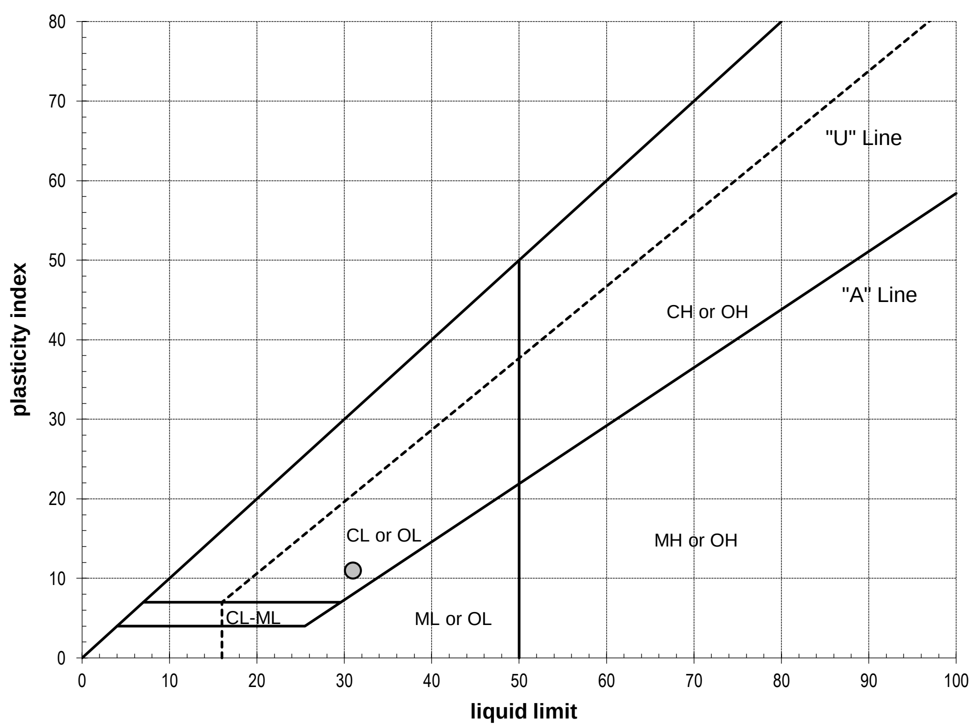
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.54	27.85		
dry soil + pan weight, g =	26.41	26.65		
pan weight, g =	20.79	20.75		
moisture, % =	20.1 %	20.3 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.4%
% sand =	14.5%
% silt and clay =	85.1%
% silt =	n/a
% clay =	n/a
moisture content =	25.3%

DATE TESTED	TESTED BY
11/30/20	RTT

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sage Property Camas, Washington	Modern NW, Inc. 8101 NE Glisan Street Portland, Oregon 97213	20287	S20-1666
		REPORT DATE	**FIELD ID**
		12/08/20	TP7.1
		DATE SAMPLED	**SAMPLED BY**
		11/20/20	MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Elastic SILT	Test Pit TP-07 depth = 12 feet	MH, Sandy Elastic Silt
SPECIFICATIONS		**AASHTO CLASSIFICATION**
none		A-7-5(11)

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) =	14552.7		
as-received moisture content =	42.2%	coefficient of curvature, C_C =	n/a
liquid limit =	58	coefficient of uniformity, C_U =	n/a
plastic limit =	33	effective size, $D_{(10)}$ =	n/a
plasticity index =	25	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	0.127 mm

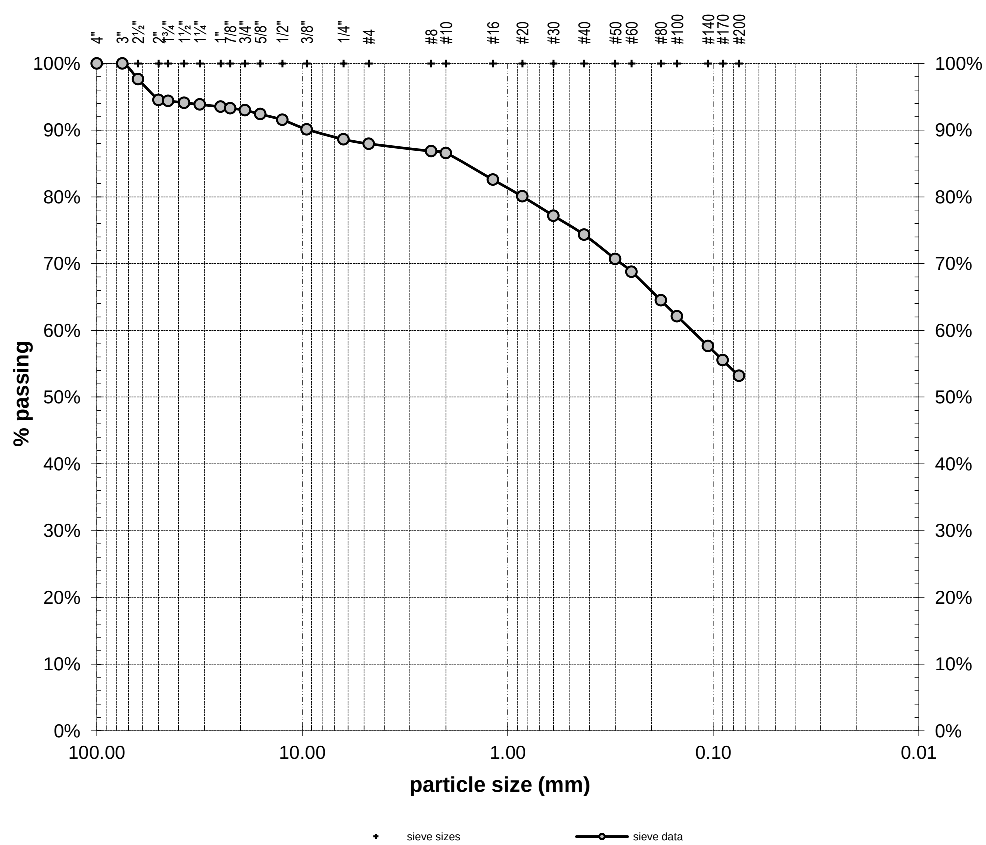
NOTE: Entire sample used for analysis; did not meet minimum size required.

SIEVE DATA

% gravel = 12.0%
% sand = 34.8%
% silt and clay = 53.2%

	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		98%		
	2.00"	50.0	95%			
	1.75"	45.0		94%		
	1.50"	37.5		94%		
	1.25"	31.5		94%		
	1.00"	25.0	93%			
	7/8"	22.4		93%		
	3/4"	19.0	93%			
	5/8"	16.0		92%		
	1/2"	12.5	92%			
	3/8"	9.50	90%			
	1/4"	6.30	89%			
	#4	4.75	88%			
SAND	#8	2.36		87%		
	#10	2.00	87%			
	#16	1.18		83%		
	#20	0.850	80%			
	#30	0.600		77%		
	#40	0.425	74%			
	#50	0.300		71%		
	#60	0.250	69%			
	#80	0.180		65%		
	#100	0.150	62%			
	#140	0.106		58%		
	#170	0.090		56%		
	#200	0.075	53%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sage Property Camas, Washington	Modern NW, Inc. 8101 NE Glisan Street Portland, Oregon 97213	20287	S20-1666
		REPORT DATE: 12/08/20	FIELD ID: TP7.1
		DATE SAMPLED: 11/20/20	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Elastic SILT	Test Pit TP-07 depth = 12 feet	MH, Sandy Elastic Silt

LABORATORY TEST DATA

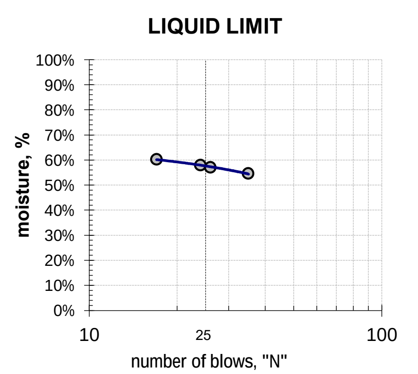
LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

ATTERBERG LIMITS

liquid limit =	58
plastic limit =	33
plasticity index =	25

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.79	33.00	32.50	33.14
dry soil + pan weight, g =	28.59	28.60	28.22	28.58
pan weight, g =	20.89	20.89	20.84	21.01
N (blows) =	35	26	24	17
moisture, % =	54.6 %	57.1 %	58.0 %	60.2 %

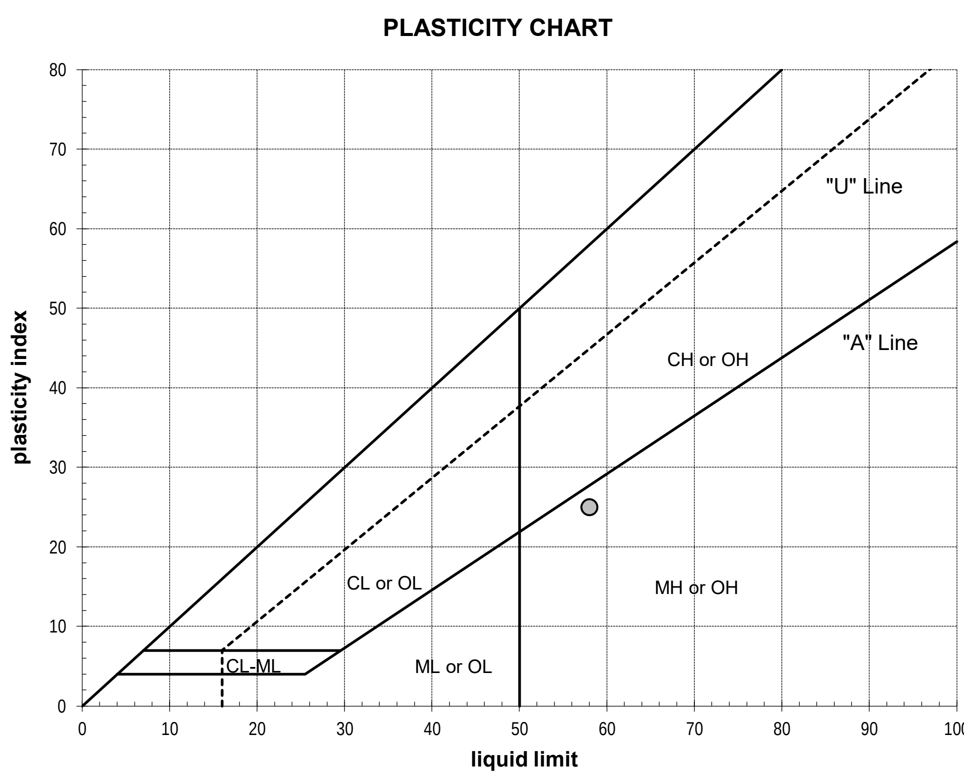


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.49	27.31		
dry soil + pan weight, g =	26.59	25.74		
pan weight, g =	20.77	20.98		
moisture, % =	32.7 %	33.0 %		



ADDITIONAL DATA

% gravel =	12.0%
% sand =	34.8%
% silt and clay =	53.2%
% silt =	n/a
% clay =	n/a
moisture content =	42.2%

DATE TESTED	TESTED BY
12/04/20	KMS

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sage Property	Modern NW, Inc.		20287	TP-1
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN MCK	DATE 11/20/20
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 650 ft amsl	GROUNDWATER DEPTH Not Observed	START TIME 0819	FINISH TIME 1230

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						Approximately 12 inches of grass and topsoil					
		Powell silt loam		CL		Tan to brown, moist, medium stiff to stiff lean CLAY. [Soil Type 2]					
	TP1.1		A-6(16)				24.0	86.8	38	18	IT1.1 D = 2.0-ft k < 0.06 in/hr
5											
			A-7	MH		Tan to orange-brown, weathered, moist, medium stiff to stiff sandy elastic SILT. [Soil Type 3]					
10						Rounded to subrounded cobbles and boulders observed at approximately 10 feet bgs.					
						Bottom of test pit at 11.0 feet bgs. Groundwater not observed to 11.0 feet bgs on 11/20/20.					
15											

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TEST PIT LOG

PROJECT NAME: Sage Property	CLIENT: Modern NW, Inc.		PROJECT NO.: 20287	TEST PIT NO.: TP-2
PROJECT LOCATION: Camas, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	TECHNICIAN: MCK	DATE: 11/20/20
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 694 ft amsl	GROUNDWATER DEPTH: Not Observed	START TIME: 0853	FINISH TIME: 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						Approximately 12 inches of grass and topsoil					
		Powell silt loam	A-6	CL		Brown to tan, moist, medium stiff lean CLAY. [Soil Type 2]					IT2.1 D = 2.0-ft k < 0.06 in/hr
5											
			A-7	MH		Red-brown, weathered, moist, medium stiff to stiff sandy elastic SILT. [Soil Type 3]					
10											
						Rounded to subrounded cobbles and boulders observed at approximately 12 feet bgs.					
						Bottom of test pit at 13.0 feet bgs. Groundwater not observed to 13.0 feet bgs on 11/20/20.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sage Property	Modern NW, Inc.		20287	TP-3
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	MCK	11/20/20
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	670 ft amsl	2.0 feet bgs	0930	1300

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						Approximately 12 inches of grass and topsoil					
		Powell silt loam		CL		Brown to tan, mottled, moist to wet, medium stiff lean CLAY. [Soil Type 2]					
▼ (2.0)	TP3.1		A-7-6(18)				29.7	86.1	41	21	IT3.1 D = 2.0-ft k < 0.06 in/hr
5											
10											
13						Bottom of test pit at 13.0 feet bgs. Groundwater observed at approximately 2.0 feet bgs on 11/20/20.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sage Property	Modern NW, Inc.		20287	TP-4
PROJECT LOCATION: Camas, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	TECHNICIAN: MCK	DATE: 11/20/20
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 676 ft amsl	GROUNDWATER DEPTH: 2.0 feet bgs	START TIME: 1005	FINISH TIME: 1028

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						Approximately 12 inches of grass and topsoil.					
		Powell silt loam	A-6	CL		Brown to tan, moist to wet, medium stiff lean CLAY. [Soil Type 2]					
5											
10											
			A-7	MH		Tan to orange-brown, weathered, wet, medium stiff to stiff sandy elastic SILT with trace to some subrounded gravel. [Soil Type 3]					
						Bottom of test pit at 13.0 feet bgs. Groundwater observed at approximately 2.0 feet bgs on 11/20/20.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sage Property	Modern NW, Inc.		20287	TP-5
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	MCK	11/20/20
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	700 ft amsl	2.0 feet bgs	1030	1059

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						Approximately 12 inches of grass and topsoil.					
						FILL. Tan to dark brown, moist to wet, medium stiff silt. [Soil Type 1]					
5		Powell silt loam	A-6	CL		Tan, mottled, wet, medium stiff lean CLAY. [Soil Type 2]					
10											
			A-7	MH		Tan to orange-brown, weathered, wet, stiff sandy elastic SILT with trace to some subrounded gravel. [Soil Type 3]					
15						Bottom of test pit at 14.0 feet bgs. Groundwater observed at approximately 2.0 feet bgs on 11/20/20.					

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TEST PIT LOG

PROJECT NAME: Sage Property	CLIENT: Modern NW, Inc.	PROJECT NO.: 20287	TEST PIT NO.: TP-6
PROJECT LOCATION: Camas, Washington	CONTRACTOR: L&S Contractors; EQUIPMENT: Excavator	TECHNICIAN: MCK	DATE: 11/20/20
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 702 ft amsl; GROUNDWATER DEPTH: 8.0 feet bgs	START TIME: 1100	FINISH TIME: 1122

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						Approximately 12 inches of grass and topsoil.					
		Powell silt loam		CL		Brown to tan, moist, medium stiff lean CLAY. [Soil Type 2]					
5	TP6.1		A-6(8)				25.3	85.1	31	11	
			A-7	MH		Brown to red-brown, weathered, wet, stiff sandy elastic SILT. [Soil Type 3]					
10											
						Rounded to subrounded cobbles and boulders at approximately 13 feet bgs.					
						Bottom of test pit at 14.0 feet bgs. Groundwater observed at approximately 8.0 feet bgs on 11/20/20.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sage Property	Modern NW, Inc.		20287	TP-7
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	MCK	11/20/20
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	696 ft amsl	6.5 feet bgs	1124	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						Approximately 14 to 16 inches of grass and topsoil.					
		Powell silt loam	A-6	CL		Brown to tan, mottled, moist to wet, medium stiff lean CLAY. [Soil Type 2]					
5											
				MH		Red-brown to orange-brown, weathered, wet, medium stiff to stiff sandy elastic SILT. [Soil Type 3]					
10											
	TP7.1		A-7-5(11)				42.2	53.2	58	25	
						Bottom of test pit at 13.0 feet bgs. Groundwater observed at approximately 6.5 feet bgs on 11/20/20.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sage Property	Modern NW, Inc.		20287	TP-8
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	MCK	11/20/20
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	674 ft amsl	3.0 feet bgs	1153	1225

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						Approximately 12 inches of grass and topsoil					
		Powell silt loam	A-6	CL		Brown to tan, moist to wet, medium stiff lean CLAY. [Soil Type 2]					
5											
			A-7	MH		Brown to red-brown, weathered, wet, medium stiff to stiff sandy elastic SILT. [Soil Type 3]					
10											
						Bottom of test pit at 12.0 feet bgs. Groundwater observed at approximately 3.0 feet bgs on 11/20/20.					
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.

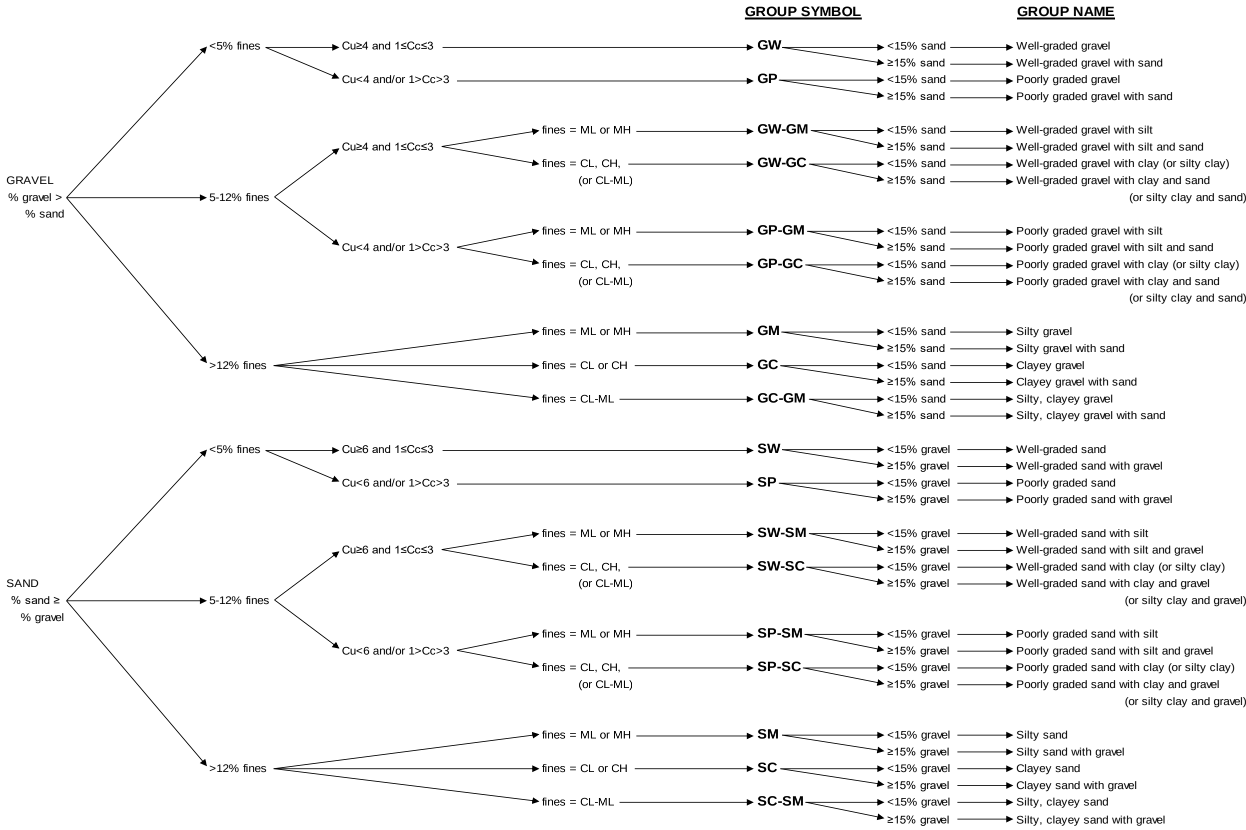
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

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			

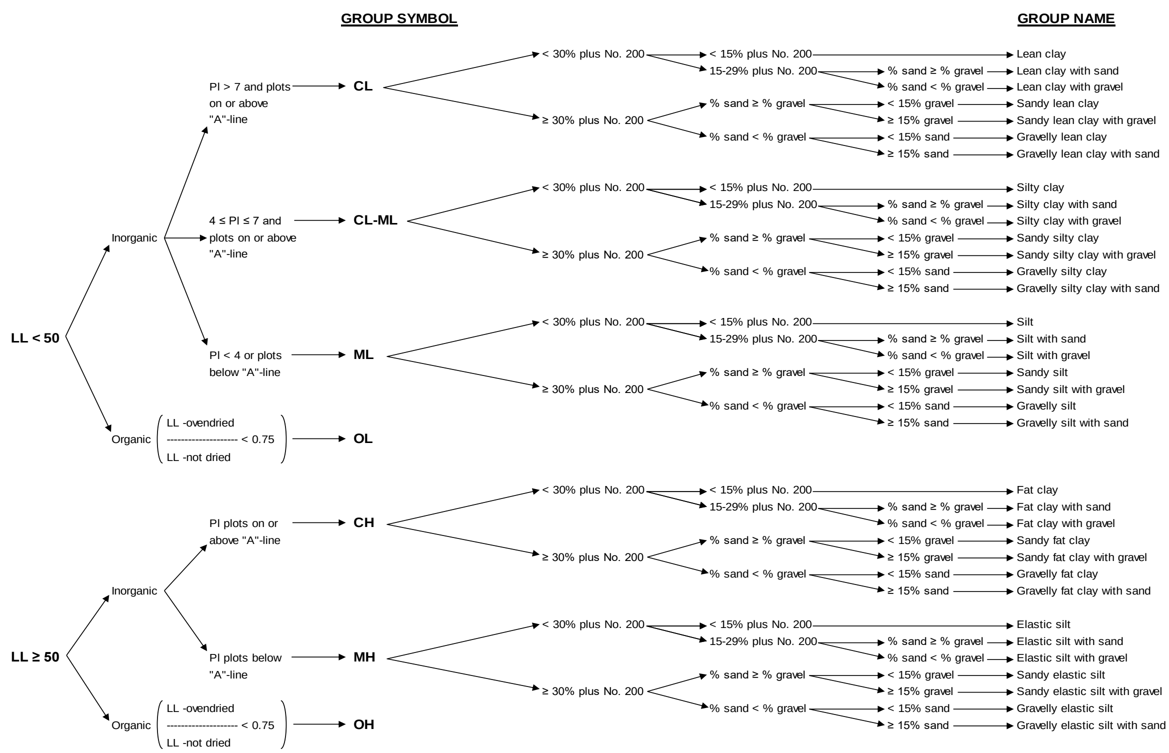
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

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

Sage Property

November, 2020

Camas, Washington

Northern Site Area, Facing East from TP-8

Sage Property

November, 2020

Camas, Washington

Central Site Area, Facing West from TP-5

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Sage Property

November, 2020

Camas, Washington

Typical Test Pit Profile, TP-1

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Typical Test Pit Profile, TP-4

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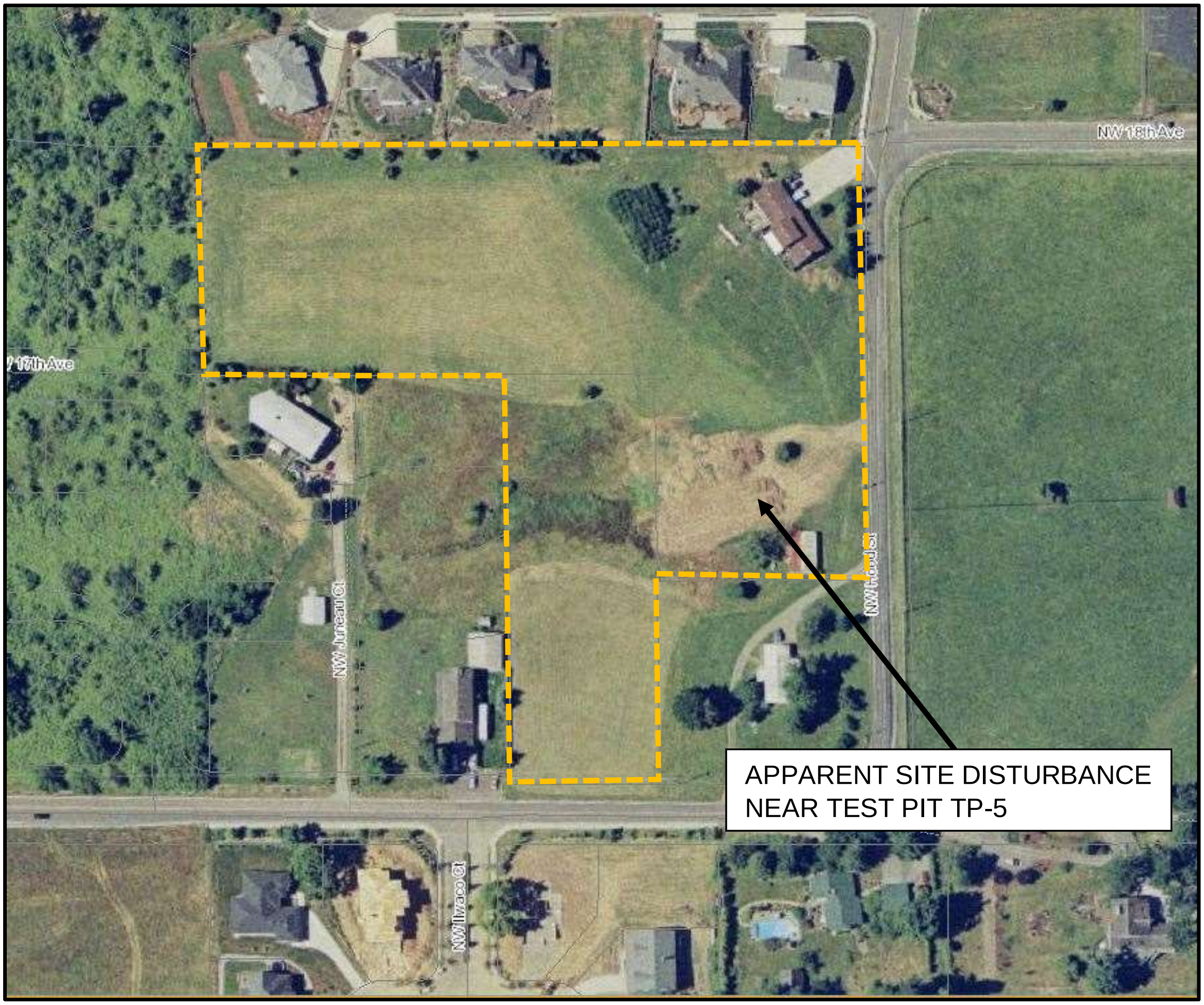
Sage Property

November, 2020

Camas, Washington



1998 Aerial Photography

(Clark County Maps Online, Accessed December, 2020)

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Geotechnical ■ Environmental ■ Special Inspections
Columbia West Engineering, Inc

Date: January 4, 2021
Project: Sage Property
Camas, 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.