

Geotechnical Site Investigation

NW 18th Avenue Subdivision

Camas, Washington

September 15, 2021

Geotechnical ■ Environmental ■ Special Inspections

Columbia West
Engineering, Inc



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NW 18th AVENUE SUBDIVISION
CAMAS, WASHINGTON**

Prepared For:

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Site Location:

**3010 NW 18th Avenue
Parcel Nos. 127356000, 127359000, and
127439000
Camas, Washington**

Prepared By:

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Date Prepared:

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TABLE OF CONTENTS

LIST OF FIGURES	ii
LIST OF APPENDICES	iii
1.0 INTRODUCTION	1
1.1 General Site Information	1
1.2 Proposed Development	1
2.0 REGIONAL GEOLOGY AND SOIL CONDITIONS	1
3.0 REGIONAL SEISMOLOGY	2
4.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION	4
4.1 Surface Investigation and Site Description	4
4.2 Subsurface Exploration and Investigation	5
4.2.1 Soil Type Description	5
4.2.2 Groundwater	5
5.0 GEOLOGIC HAZARDS	5
5.1 Erosion Hazards	6
5.2 Landslide Hazards	6
5.2.1 Geologic Literature Review	6
5.2.2 Slope Reconnaissance and Slope Stability Assessment	6
5.3 Seismic Hazard Areas	7
5.3.1 Soil Liquefaction and Dynamic Settlement	7
5.3.2 Ground Shaking Amplification	7
5.3.3 Fault Rupture	7
6.0 DESIGN RECOMMENDATIONS	8
6.1 Site Preparation and Grading	8
6.2 Engineered Structural Fill	8
6.3 Cut and Fill Slopes	9
6.4 Foundations	9
6.5 Slabs on Grade	10
6.6 Static Settlement	11
6.7 Excavation	11
6.8 Lateral Earth Pressure	11
6.9 Seismic Design Considerations	12
6.10 Infiltration Testing Results	13
6.11 Drainage	14
6.12 Bituminous Asphalt and Portland Cement Concrete	15
6.13 Wet Weather Construction Methods and Techniques	16
6.14 Erosion Control Measures	16
6.15 Utility Installation	17
7.0 CONCLUSION AND LIMITATIONS	17
REFERENCES	
FIGURES	
APPENDICES	

LIST OF FIGURES

<u>Number</u>	<u>Title</u>
1	Site Location Map
2	Exploration Location Map
3	Typical Cut and Fill Slope Cross-Section
4	Minimum Foundation Slope Setback Detail
5	Typical Perimeter Footing Drain Detail
6	Typical Perforated Drainpipe Trench Detail

LIST OF APPENDICES

<u>Number</u>	<u>Title</u>
A	Analytical Laboratory Test Reports
B	Exploration Logs
C	Soil Classification Information
D	Photo Log
E	Report Limitations and Important Information

GEOTECHNICAL SITE INVESTIGATION

NW 18th AVENUE SUBDIVISION

CAMAS, WASHINGTON

1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Modern Dwellings, LLC to conduct a geotechnical site investigation for the proposed NW 18th Avenue Subdivision 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. The specific scope of services was outlined in a proposal contract dated June 21, 2021. This report summarizes the investigation and provides field assessment documentation and laboratory analytical test reports. This report is subject to the limitations expressed in Section 7.0, *Conclusion and Limitations*, and Appendix E.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located at 3010 NW 18th Avenue in Camas, Washington. The site is comprised of tax parcels 127356000, 127359000, and 127439000 totaling approximately 9.82 acres. The approximate latitude and longitude are N 45° 35' 27" and W 122° 26' 25", and the legal description is a portion of the NE ¼ of Section 09, T1N, R3E, Willamette Meridian. The regulatory jurisdictional agency is City of Camas, Washington.

1.2 Proposed Development

Correspondence with the design team and review of the proposed development plan shown on Figure 2A indicates that proposed development at the NW 18th Avenue Subdivision includes the development of the above-referenced tax parcels into 35 new single-family residential lots, private asphalt access drives, public asphalt roadways, underground utilities, and stormwater facilities. Columbia West has not reviewed preliminary grading plans but understands that cut and fill may be proposed at the subject site. This report is based upon proposed development as described above and may not be applicable if modified.

2.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

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

According to the *Geologic Map of the Camas Quadrangle, Clark County, Washington, and Multnomah County, Oregon* (Russell C. Evarts, USGS Geological Survey, 2008),

near-surface soils at 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], 2021 Website) identifies surface soils as Powell silt loam. Although soil conditions may vary from the broad USDA descriptions, Powell soils are generally fine-textured silts and clays with low permeability, moderate water capacity, moderate shrink-swell potential, and moderate shear strength. They exhibit a slight erosion hazard based primarily on 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. Liquefaction is discussed later in Section 5.3.1, *Soil Liquefaction and Dynamic Settlement*.

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

Portland Hills Fault Zone

The Portland Hills Fault Zone consists of several northwest-trending faults located along the northeastern margin of the Tualatin Mountains, also known as the Portland Hills, and the southwest margin of the Portland Basin. The fault zone is approximately 25 to 30 miles in length and is located approximately 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 2 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, six test pits (TP-1 through TP-6) and five infiltration tests (IT-1.1 to IT-5.2) occurred on August 5, 2021. The 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 9.82-acre subject site consists of three tax parcels located at 3010 NE 18th Avenue in Camas, Washington. The site is bounded by NW 18th Avenue to the north, NW Hood Street to the west, residential acreage to the south, and newly developed Haven Heights subdivision to the east. The site currently has a multi-story residence, two multi-story garage structures, and an agricultural structure located on the east half of the site. The area on the west is undeveloped grassy field. Site vegetation primarily consisted of grass in open areas with trees and shrubs concentrated around existing structures.

Field reconnaissance and review of site topographic mapping indicate rolling to gently sloped terrain with grades of 10 to 15 percent characterizing the site. Site elevations range from approximately 692 feet above mean sea level (amsl) in the southwest corner of the parcel to approximately 754 feet amsl in the northeast corner. 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, *Geologic Hazards*.

4.2 Subsurface Exploration and Investigation

The test pits were explored to a maximum depth of 13.5 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 8 to 14 inches of sod and topsoil in the observed locations. Underlying the topsoil layer subsurface soils resembling native USDA Powell soil series description were encountered. Subsurface lithology may generally be described by soil types identified in the following text. Field logs and observed stratigraphy for the encountered materials are presented in Appendix B, *Subsurface Exploration Logs*.

Soil Type 1 – Lean CLAY / CLAY with Sand

Soil Type 1 was observed to primarily consist of light brown to brown, brown, gray to light brown, dry, moist, lean CLAY and CLAY with sand. Occasional sub-rounded cobbles were observed. Soil Type 1 was observed below topsoil layer in all test pits and extended to the maximum depth of exploration in all test pits.

4.2.2 Groundwater

Groundwater was not encountered within the test pit explorations to maximum explored depth of 13.5 feet bgs. Groundwater levels are often subject to seasonal variance and may rise during extended periods of increased precipitation or flooding. 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.

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

5.0 GEOLOGIC HAZARDS

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 south central portion 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-6 were explored in sloped areas. Subsurface native soils at the locations tested generally consisted of lean CLAY and CLAY with sand with trace to some gravels and cobbles. 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 62 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, in Columbia West’s opinion slopes on the subject site do not 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 6.9, *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 fine-textured soils. Design recommendations are presented in the following text sections.

6.1 Site Preparation and Grading

Vegetation, organic material, unsuitable fill, and deleterious material that may be encountered should be cleared from areas identified for structures and site grading. Vegetation, other organic material, and debris should be removed from the site. Stripped topsoil should also be removed or used only as landscape fill in nonstructural areas with slopes less than 25 percent. The stripping depth for sod and highly organic topsoil is anticipated to vary between approximately 8 and 14 inches. The required stripping depth may increase in areas of existing fill, heavy organics, or previously existing structures. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed. The post-construction maximum depth of landscape fill placed or spread at any location onsite should not exceed one foot.

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

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

6.2 Engineered Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in the preceding text. Surface soils should be scarified and compacted prior to additional fill placement. Engineered structural fill should be placed in loose lifts not exceeding 12 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within two percentage points of optimum conditions. A field density at least equal to 95 percent of the maximum dry density, obtained from the modified Proctor moisture-density relationship test (*ASTM 1557*), is recommended for structural fill placement and scarified and recompacted subgrade.

Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with *ASTM D6938*. Field compaction testing should be performed for each vertical foot of engineered fill placed. Engineered fill placement should be observed by Columbia West.

Engineered structural fill placement activities should be performed during dry summer months if possible. Most clean native soils may be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native soils may require addition of moisture during periods of dry weather. Compacted fill soils should be covered shortly after placement.

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

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

6.3 Cut and Fill Slopes

Fill placed on existing grades steeper than 5H:1V should be horizontally benched at least 10 feet into the slope. Fill slopes greater than six feet in height should be vertically keyed into existing subsurface soil. A typical fill slope cross-section is shown in Figure 3. Drainage implementations, including subdrains or perforated drainpipe trenches, may also be necessary in proximity to cut and fill slopes if seeps or springs are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by Columbia West during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion.

Final cut or fill slopes at the site should not exceed 2H:1V or 10 feet in height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 4.

Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Fill slopes should be constructed by placing fill material in maximum 12-inch level lifts, compacting as described in Section 6.2, *Engineered Structural Fill* and horizontally benching where appropriate. Fill slopes should be overbuilt, compacted, and trimmed at least two feet horizontally to provide adequate compaction of the outer slope face. Proper cut and fill slope construction is critical to overall project stability and should be observed and documented by Columbia West.

6.4 Foundations

Foundations for proposed structures are anticipated to consist of shallow continuous perimeter or column spread footings. Footings should be designed by a licensed structural

engineer and conform to the recommendations below. Typical building loads are not expected to exceed approximately 3 kips per foot for perimeter footings or 10 kips per column. If actual loading exceeds anticipated loading, additional analysis should be conducted for the specific load conditions and proposed footing dimensions.

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

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

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

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

6.5 Slabs on Grade

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

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

For lightly loaded slabs not exceeding 200 psf, the modulus of subgrade reaction is estimated to be 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 13.5 feet using a track-mounted excavator. Bedrock was not encountered and blasting or specialized rock-excavation techniques are not anticipated. Groundwater was not encountered within the test pit explorations. However, perched groundwater layers may exist at shallower depths depending on seasonal fluctuations in the water table.

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

Site-specific shoring design may be required if open-cut excavations are infeasible or if excavations are proposed adjacent to existing infrastructure. Typical methods for stabilizing excavations consist of soldier piles and timber lagging, sheet pile walls, tiebacks and shotcrete, or pre-fabricated hydraulic shoring. Because lateral earth pressure distributions acting on below-grade structures are dependent upon the type of shoring system used, Columbia West should be contacted to conduct additional analysis when shoring type, excavation depths, and locations are known.

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

6.8 Lateral Earth Pressure

Lateral earth pressures should be considered during design of retaining walls and below grade structures. Hydrostatic pressure and additional surcharge loading should also be considered. Wall foundation construction and bearing capacity should adhere to specifications provided previously in Section 6.4, *Foundations*. Retained material may include engineered structural backfill or undisturbed native soil. Structural wall backfill should consist of imported granular material meeting *Section 9-03.12(2)* of *WSDOT*

Standard Specifications. Backfill should be prepared and compacted to at least 95 percent of maximum dry density as determined by the modified Proctor test (*ASTM D1557*). Recommended parameters for lateral earth pressures for retained soils and engineered structural backfill consisting of imported granular fill meeting WSDOT specifications for *Gravel Backfill for Walls 9-03.12(2)* are presented in Table 1.

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

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

Table 1. Recommended Lateral Earth Pressure Parameters for Level Backfill

Retained Soil	Equivalent Fluid Pressure for Level Backfill			Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive		
Undisturbed native Lean CLAY and CLAY with Sand (Soil Type 1)	61 pcf	42 pcf	319 pcf	115 pcf	28°
Approved Structural Backfill Material	52 pcf	32 pcf	568 pcf	135 pcf	38°
WSDOT 9-03.12(2) compacted aggregate backfill					

**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.*

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

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

6.9 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	0.375 g
0.2 sec Spectral Acceleration	0.832 g
1.0 sec Spectral Acceleration	0.358 g

The listed probabilistic ground motion values are based upon “firm rock” sites with an assumed shear wave velocity of 2,500 ft/s in the upper 100 feet of soil profile. These values should be adjusted for site class effects by applying site coefficients F_a , F_v , and F_{PGA} as defined in *ASCE 7-16* and associated *ASCE 7-16 Supplement 1* dated December 12, 2018, *Tables 11.4-1, 11.4-2, and 11.8.1*. The site coefficients are intended to more accurately characterize estimated peak ground and respective earthquake spectral response accelerations by considering site-specific soil characteristics and index properties. As mentioned previously in Section 5.3.2 *Ground Shaking Amplification*, the Site Class Map of Clark County indicates site soils may be represented by Site Class C.

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

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

6.10 Infiltration Testing Results

To investigate the feasibility of subsurface disposal of stormwater, Columbia West conducted in situ infiltration testing at various locations within the project area on August 5, 2021. Results of in situ infiltration testing are presented in Table 3. The soil classification presented in Table 3 is based upon laboratory analysis and visual classification. The infiltration rate is presented as a recommended coefficient of permeability (k) and has been reported without application of a factor of safety.

As indicated in Table 3, the test was conducted in test pit TP-2, TP-4, and TP-5 at depths of 2 to 5 feet bgs. Soils in the tested location were observed and sampled to adequately characterize the subsurface profile. Tested native soils are classified as lean CLAY (CL) and CLAY with sand (CL) according to USCS specifications. Soil laboratory analytical test reports are provided in Appendix A.

Single-ring, falling head infiltration testing was performed by inserting a three-inch diameter pipe into the soil at the noted depth. The test was conducted by filling the apparatus with water and measuring time relative to changes in hydraulic head at regular intervals. Using

Darcy's Law for saturated flow in homogenous media, the coefficient of permeability (k) was then calculated.

Table 3. Infiltration Test Results

Test Number	Location	Test Depth (feet bgs)	Groundwater Depth on 08/05/21 (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-2.1	TP-2	2.0	Not Observed to 13 bgs	CL, Lean CLAY	4	88.1	< 0.06
IT-2.2		5.0		CL, Lean CLAY*	4	-	< 0.06
IT-4.1	TP-4	4.0	Not Observed to 12.5 bgs	CL, Lean CLAY*	4	-	< 0.06
IT-5.1	TP-5	2.0	Not Observed to 13 bgs	CL, CLAY with Sand*	4	-	< 0.06
IT-5.2		5.0		CL, CLAY with Sand	4	82.9	< 0.06

* WWHM Classifications are Based Upon Subsurface Investigation and Infiltration Testing Conducted at the Locations Shown.

Columbia West also classified tested near-surface soils into a representative soil group 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 *2021 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 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 via infiltration is likely infeasible and is not recommended without further study.

6.11 Drainage

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. Drainage design in general should conform to City of 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 ft³ of clean, washed drain rock per linear foot of pipe and wrapped with geotextile filter fabric. Open-graded drain rock with a maximum particle size of 3 inches and less than 2 percent passing the No. 200 sieve is recommended. Geotextile filter fabric should consist of Mirafi 140N or approved equivalent, with AOS between No. 70 and No. 100 sieve. The water permittivity should be greater than 1.5/sec. Figure 5 presents a typical foundation

drain. Perimeter drains may limit increased hydrostatic pressure beneath footings and assist in reducing potential perched moisture areas.

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

Drains should be closely monitored after construction to assess their effectiveness. If additional surface or shallow subsurface seeps become evident, the drainage provisions may require modification or additional drains. Columbia West should be consulted to provide appropriate recommendations.

6.12 Bituminous Asphalt and Portland Cement Concrete

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

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.

6.13 Wet Weather Construction Methods and Techniques

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

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

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

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

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

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

6.14 Erosion Control Measures

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.

6.15 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

Geotechnical Site Investigation
NW 18th Avenue Subdivision, Camas, Washington

Page 18

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



9-15-2021

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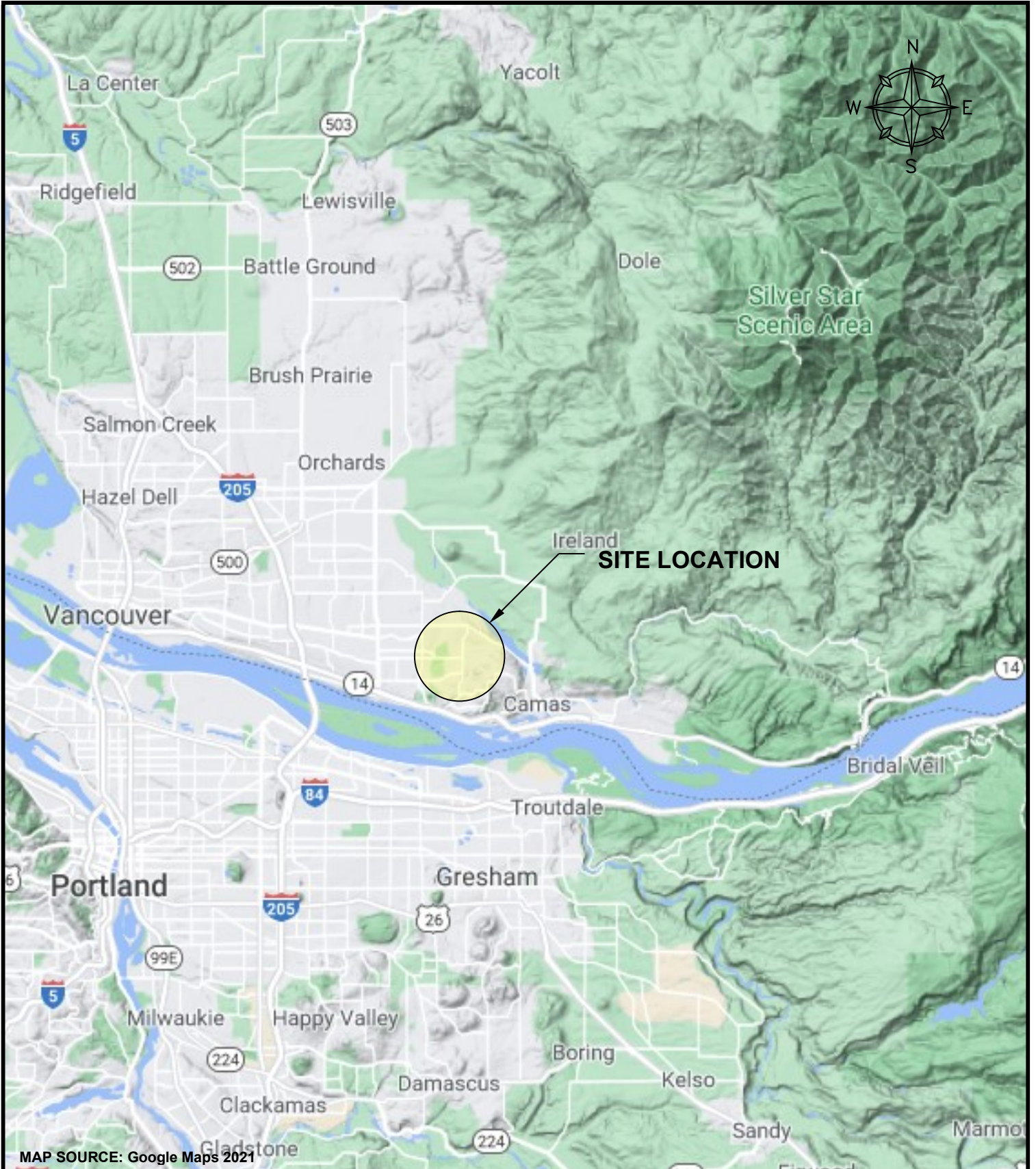
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FIGURES



MAP SOURCE: Google Maps 2021



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Design	Drawn: EMU		
Checked: ASR	Date: 08/11/2021		
Client: Mod Dwellings	Rev	By	Date
Job No.: 21184			
CAD File: FIGURE 1			
Scale: NONE			

SITE LOCATION MAP

NW 18TH AVENUE SUBDIVISION
 CAMAS, WASHINGTON

FIGURE
 1

NW 18TH AVENUE

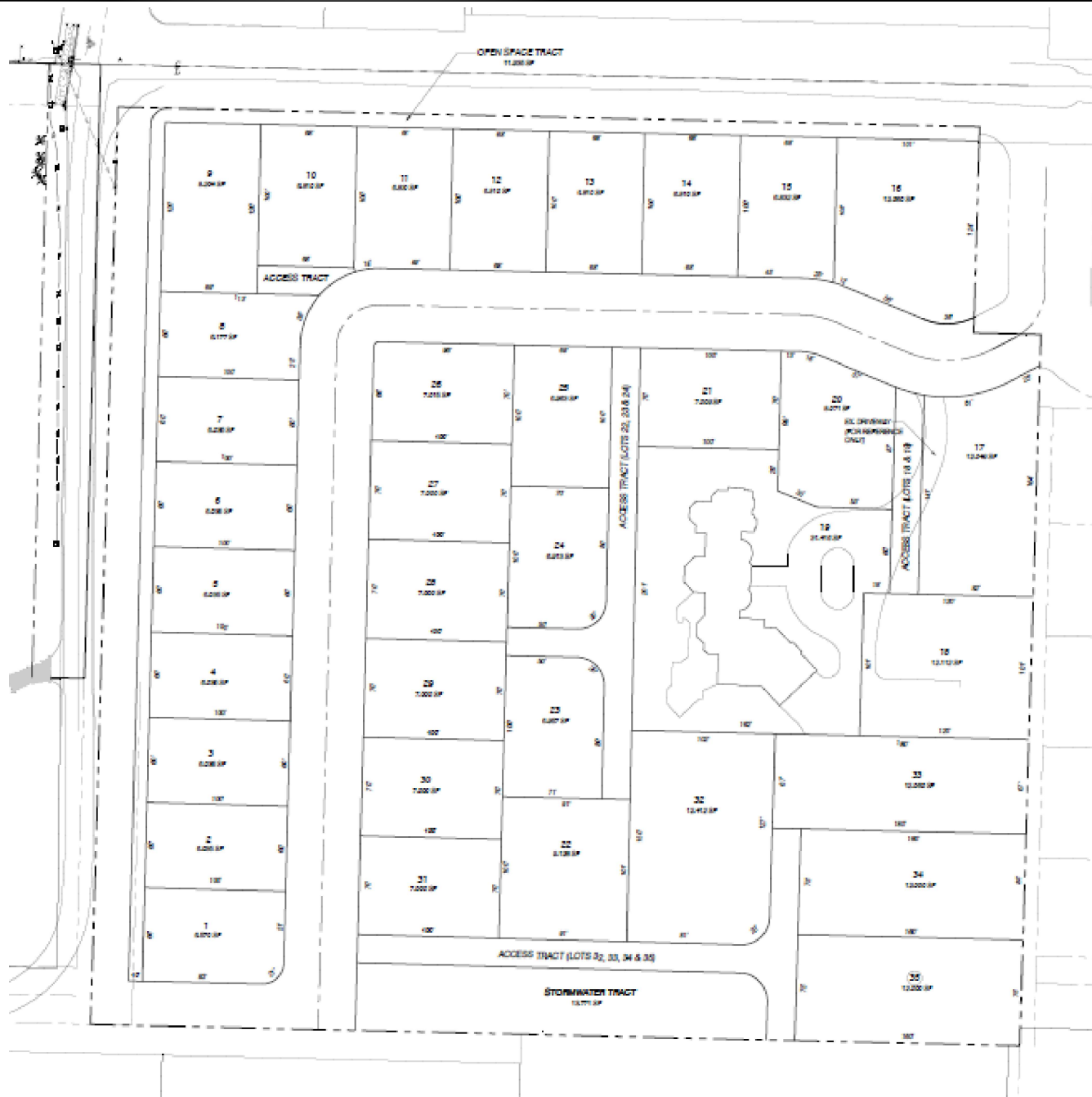
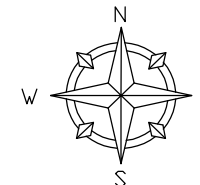
NW HOOD STREET

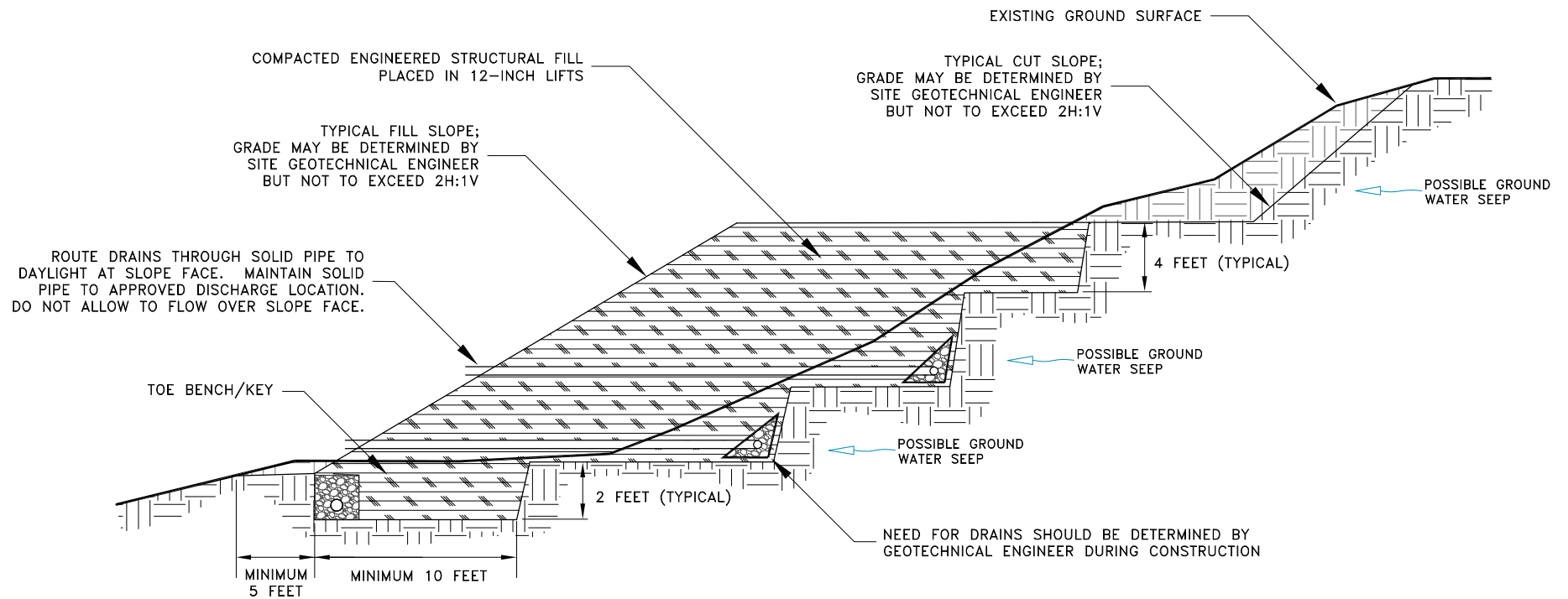


Infiltration Test Results						
Test Number	Location	Approximate Test Depth (feet bgs)	Approximate Depth to Groundwater on 08/05/21 (feet bgs)	USCS Soil Type (* Indicates Visual Classification)	Passing No. 200 Sieve (%)	Infiltration Rate (Coefficient of Permeability, k) (inches/hour)
IT-2.1	TP-2	2.0	Not observed to 13 feet bgs	CL, Lean CLAY	88.1	< 0.06
IT-2.2		5.0		CL, Lean CLAY*	-	< 0.06
IT-4	TP-4	4.0	Not observed to 12.5 feet bgs	CL, Lean CLAY*	-	< 0.06
IT-5.1	TP-5	2.0	Not observed to 13 feet bgs	CL, CLAY with Sand*	-	< 0.06
IT-5.2		5.0		CL, CLAY with Sand	82.9	< 0.06



--- APPROXIMATE SITE BOUNDARY APPROXIMATE LOCATION OF INFILTRATION TEST APPROXIMATE LOCATION OF TEST PIT



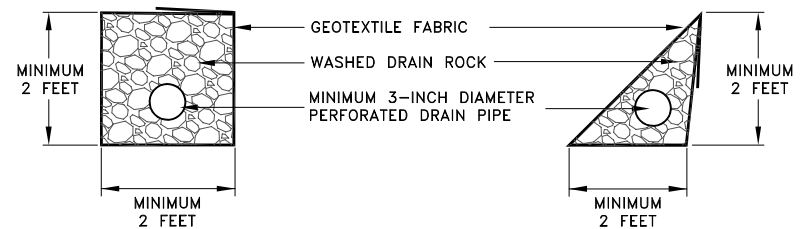


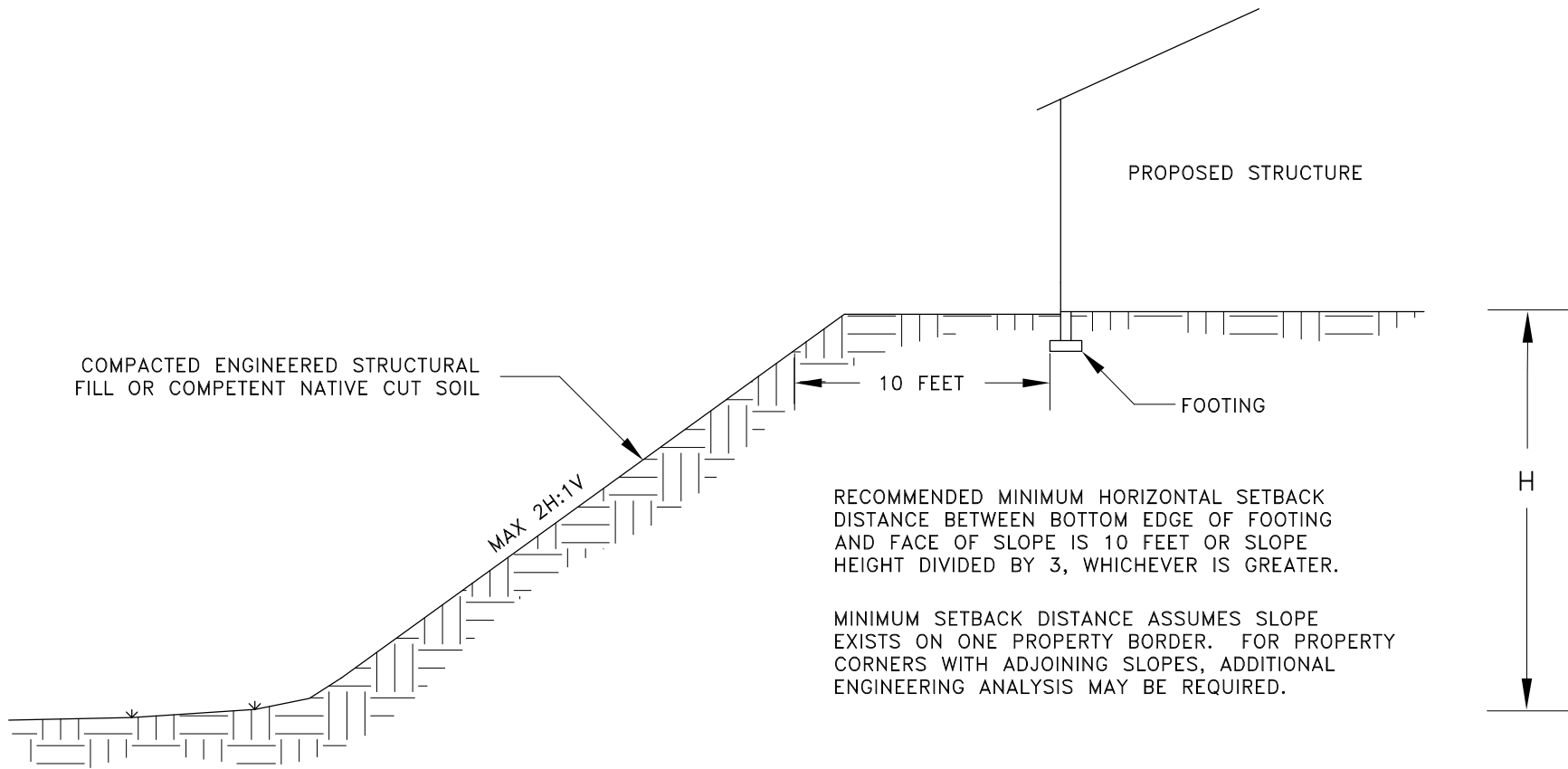
DRAIN SPECIFICATIONS

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





COMPACTED ENGINEERED STRUCTURAL FILL OR COMPETENT NATIVE CUT SOIL

MAX 2H:1V

PROPOSED STRUCTURE

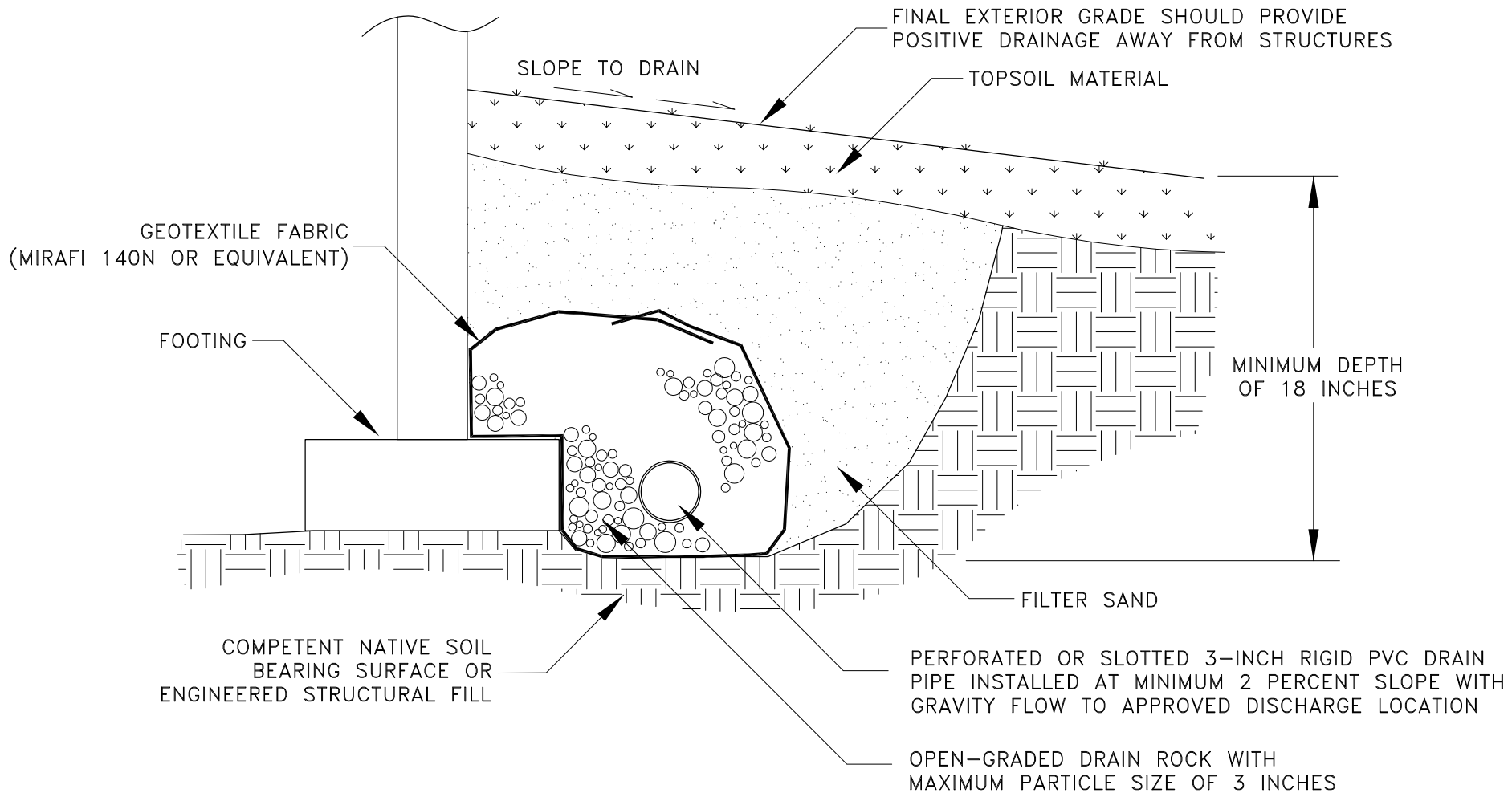
10 FEET

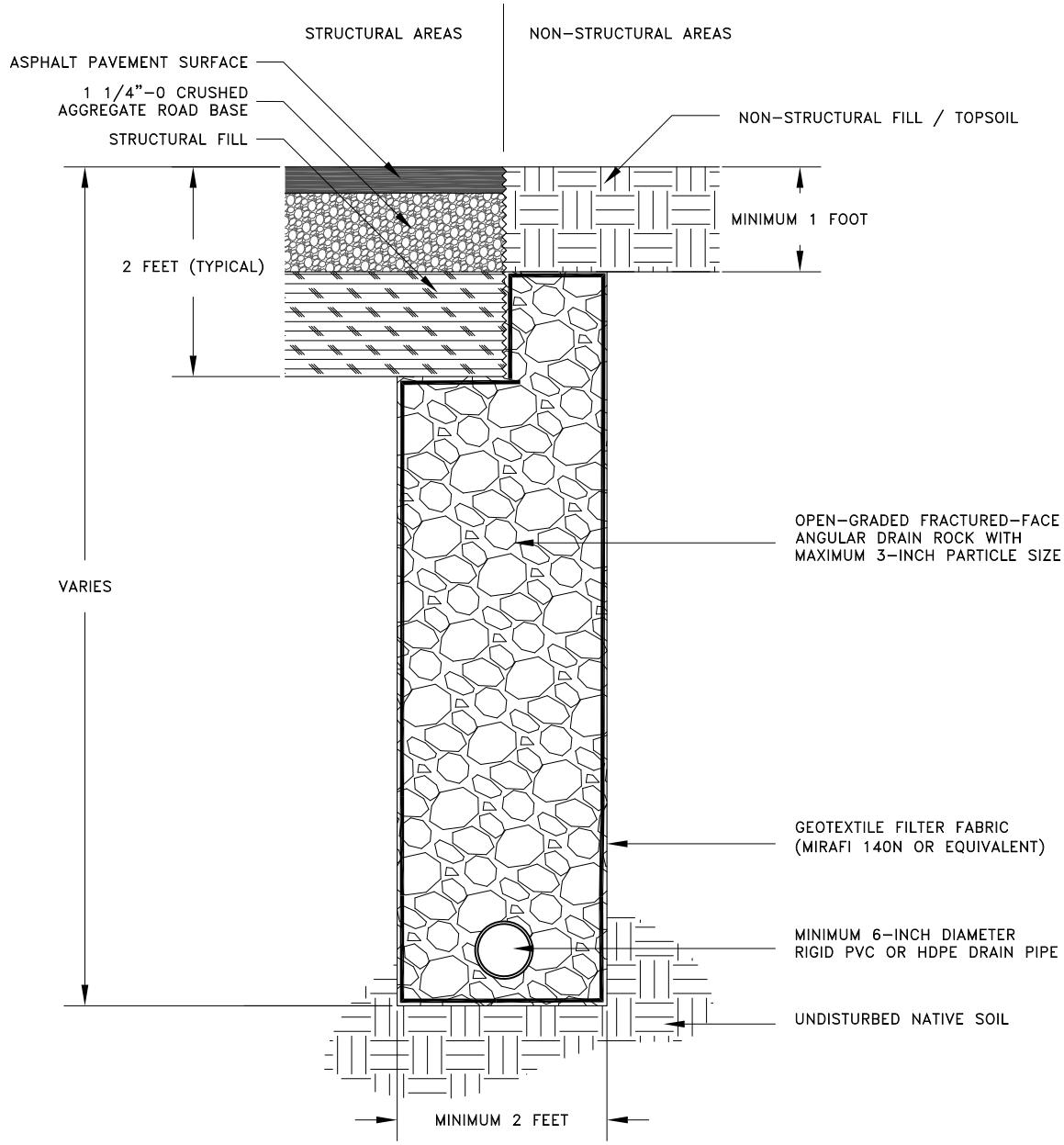
FOOTING

H

RECOMMENDED MINIMUM HORIZONTAL SETBACK DISTANCE BETWEEN BOTTOM EDGE OF FOOTING AND FACE OF SLOPE IS 10 FEET OR SLOPE HEIGHT DIVIDED BY 3, WHICHEVER IS GREATER.

MINIMUM SETBACK DISTANCE ASSUMES SLOPE EXISTS ON ONE PROPERTY BORDER. FOR PROPERTY CORNERS WITH ADJOINING SLOPES, ADDITIONAL ENGINEERING ANALYSIS MAY BE REQUIRED.





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.

APPENDIX A
LABORATORY TEST RESULTS

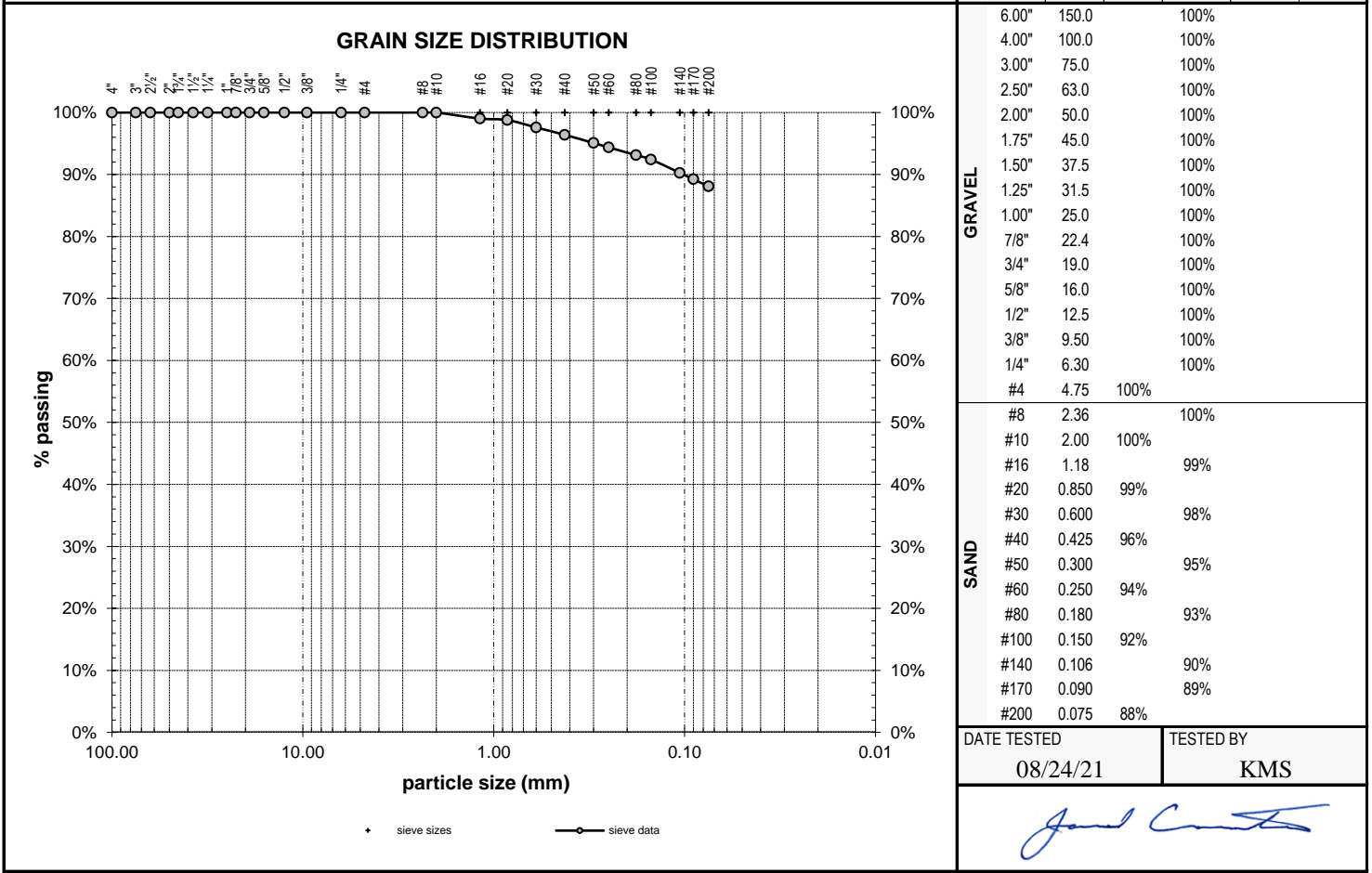
PARTICLE-SIZE ANALYSIS REPORT

PROJECT NW 18th Avenue Subdivision Camas, Washington	CLIENT Modern Dwellings, LLC 8101 NE Glisan Street Portland, Oregon 97213	PROJECT NO. 21184	LAB ID S21-0708
		REPORT DATE 08/27/21	FIELD ID TP2.1
		DATE SAMPLED 08/05/21	SAMPLED BY EMU

MATERIAL DATA	
MATERIAL SAMPLED Lean CLAY	MATERIAL SOURCE Test Pit, TP-02 depth = 2 feet
SPECIFICATIONS none	USCS SOIL TYPE CL, Lean Clay
AASHTO CLASSIFICATION A-6(16)	

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

ADDITIONAL DATA	SIEVE DATA
initial dry mass (g) = 140.36 as-received moisture content = 22.7% liquid limit = 37 plastic limit = 18 plasticity index = 19 fineness modulus = n/a	coefficient of curvature, C_c = n/a coefficient of uniformity, C_u = n/a effective size, $D_{(10)}$ = n/a $D_{(30)}$ = n/a $D_{(60)}$ = n/a
	% gravel = 0.0% % sand = 11.9% % silt and clay = 88.1%



DATE TESTED 08/24/21	TESTED BY KMS

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 www.columbiawestengineering.com



ATTERBERG LIMITS REPORT

PROJECT NW 18th Avenue Subdivision Camas, Washington	CLIENT Modern Dwellings, LLC 8101 NE Glisan Street Portland, Oregon 97213	PROJECT NO. 21184	LAB ID S21-0708
		REPORT DATE 08/27/21	FIELD ID TP2.1
		DATE SAMPLED 08/05/21	SAMPLED BY EMU

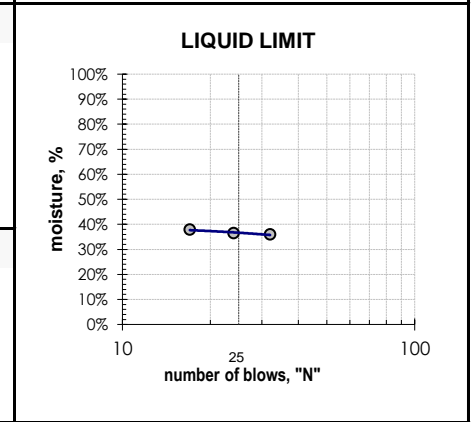
MATERIAL DATA

MATERIAL SAMPLED Lean CLAY	MATERIAL SOURCE Test Pit, TP-02 depth = 2 feet	USCS SOIL TYPE CL, Lean Clay
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LABORATORY TEST DATA

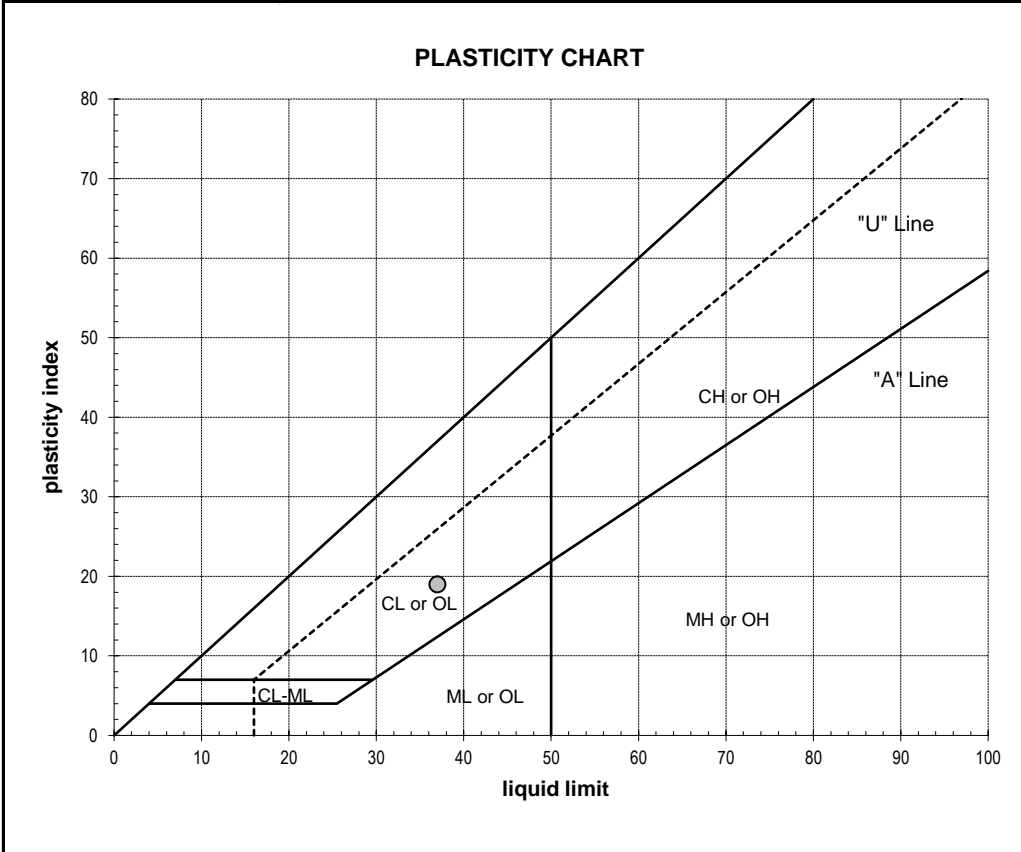
LABORATORY EQUIPMENT Liquid Limit Machine, Hand Rolled	TEST PROCEDURE ASTM D4318
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ATTERBERG LIMITS liquid limit = 37 plastic limit = 18 plasticity index = 19	LIQUID LIMIT DETERMINATION <table style="width: 100%; text-align: center;"> <tr> <td></td> <td>1</td> <td>2</td> <td>3</td> <td>4</td> </tr> <tr> <td>wet soil + pan weight, g =</td> <td>34.72</td> <td>35.69</td> <td>34.85</td> <td></td> </tr> <tr> <td>dry soil + pan weight, g =</td> <td>31.02</td> <td>31.71</td> <td>30.90</td> <td></td> </tr> <tr> <td>pan weight, g =</td> <td>20.71</td> <td>20.79</td> <td>20.47</td> <td></td> </tr> <tr> <td>N (blows) =</td> <td>32</td> <td>24</td> <td>17</td> <td></td> </tr> <tr> <td>moisture, % =</td> <td>35.9 %</td> <td>36.5 %</td> <td>37.9 %</td> <td></td> </tr> </table>		1	2	3	4	wet soil + pan weight, g =	34.72	35.69	34.85		dry soil + pan weight, g =	31.02	31.71	30.90		pan weight, g =	20.71	20.79	20.47		N (blows) =	32	24	17		moisture, % =	35.9 %	36.5 %	37.9 %	
	1	2	3	4																											
wet soil + pan weight, g =	34.72	35.69	34.85																												
dry soil + pan weight, g =	31.02	31.71	30.90																												
pan weight, g =	20.71	20.79	20.47																												
N (blows) =	32	24	17																												
moisture, % =	35.9 %	36.5 %	37.9 %																												



SHRINKAGE shrinkage limit = n/a shrinkage ratio = n/a	PLASTIC LIMIT DETERMINATION <table style="width: 100%; text-align: center;"> <tr> <td></td> <td>1</td> <td>2</td> <td>3</td> <td>4</td> </tr> <tr> <td>wet soil + pan weight, g =</td> <td>27.78</td> <td>28.22</td> <td></td> <td></td> </tr> <tr> <td>dry soil + pan weight, g =</td> <td>26.71</td> <td>27.04</td> <td></td> <td></td> </tr> <tr> <td>pan weight, g =</td> <td>20.80</td> <td>20.60</td> <td></td> <td></td> </tr> <tr> <td>moisture, % =</td> <td>18.1 %</td> <td>18.3 %</td> <td></td> <td></td> </tr> </table>		1	2	3	4	wet soil + pan weight, g =	27.78	28.22			dry soil + pan weight, g =	26.71	27.04			pan weight, g =	20.80	20.60			moisture, % =	18.1 %	18.3 %		
	1	2	3	4																						
wet soil + pan weight, g =	27.78	28.22																								
dry soil + pan weight, g =	26.71	27.04																								
pan weight, g =	20.80	20.60																								
moisture, % =	18.1 %	18.3 %																								

ADDITIONAL DATA	
% gravel =	0.0%
% sand =	11.9%
% silt and clay =	88.1%
% silt =	n/a
% clay =	n/a
moisture content =	22.7%



DATE TESTED 08/26/21	TESTED BY KMS
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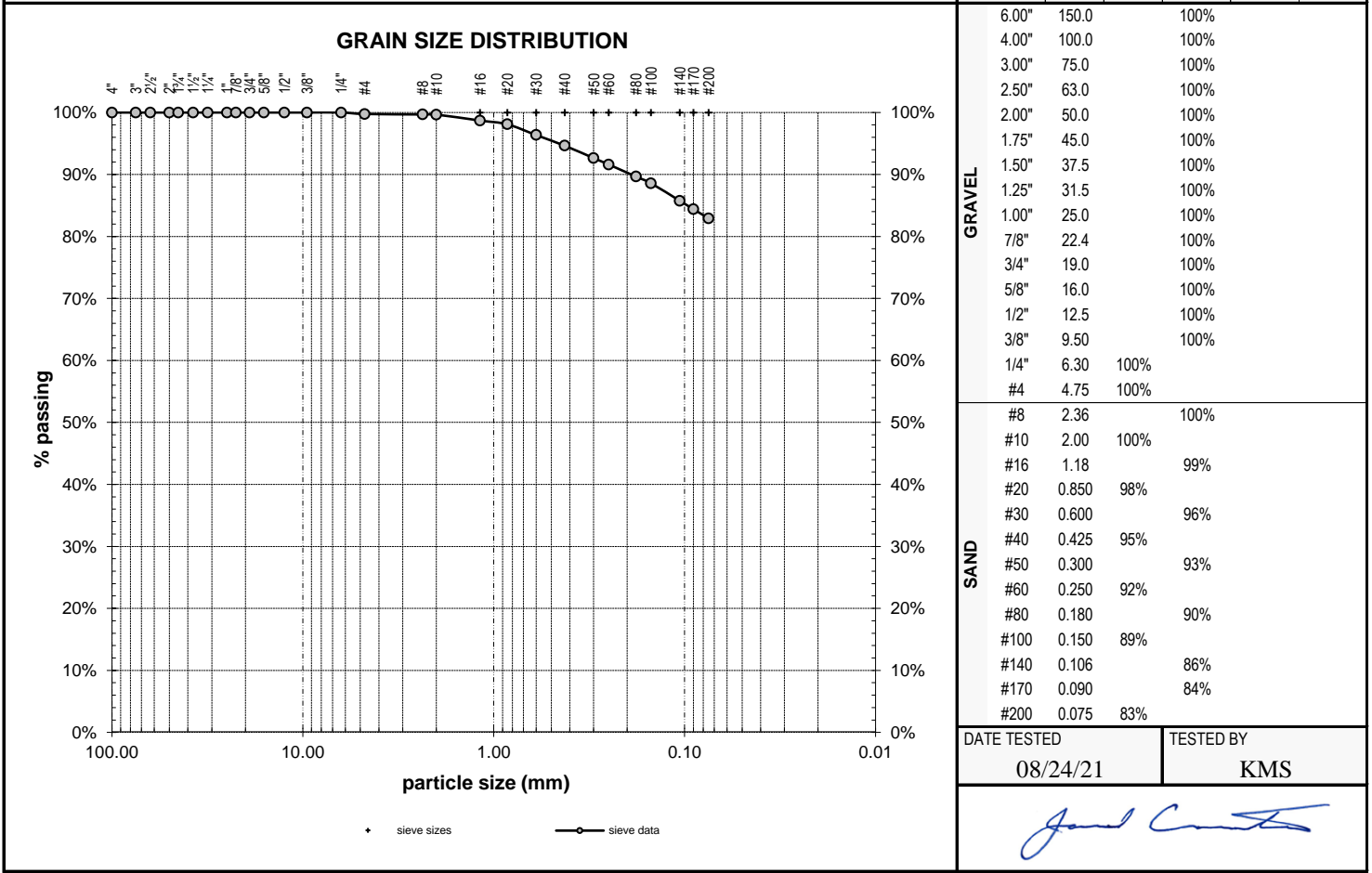
PARTICLE-SIZE ANALYSIS REPORT

PROJECT NW 18th Avenue Subdivision Camas, Washington	CLIENT Modern Dwellings, LLC 8101 NE Glisan Street Portland, Oregon 97213	PROJECT NO. 21184	LAB ID S21-0709
		REPORT DATE 08/27/21	FIELD ID TP5.2
		DATE SAMPLED 08/05/21	SAMPLED BY EMU

MATERIAL DATA		
MATERIAL SAMPLED Lean CLAY with Sand	MATERIAL SOURCE Test Pit, TP-05 depth = 5 feet	USCS SOIL TYPE CL, Lean Clay with Sand
SPECIFICATIONS none		AASHTO CLASSIFICATION A-6(15)

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

ADDITIONAL DATA initial dry mass (g) = 143.71 as-received moisture content = 22.8% liquid limit = 38 plastic limit = 19 plasticity index = 19 fineness modulus = n/a coefficient of curvature, C_c = n/a coefficient of uniformity, C_u = n/a effective size, $D_{(10)}$ = n/a $D_{(30)}$ = n/a $D_{(60)}$ = n/a	SIEVE DATA % gravel = 0.2% % sand = 16.8% % silt and clay = 82.9%
--	---



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ATTERBERG LIMITS REPORT

PROJECT NW 18th Avenue Subdivision Camas, Washington	CLIENT Modern Dwellings, LLC 8101 NE Glisan Street Portland, Oregon 97213	PROJECT NO. 21184	LAB ID S21-0709
		REPORT DATE 08/27/21	FIELD ID TP5.2
		DATE SAMPLED 08/05/21	SAMPLED BY EMU

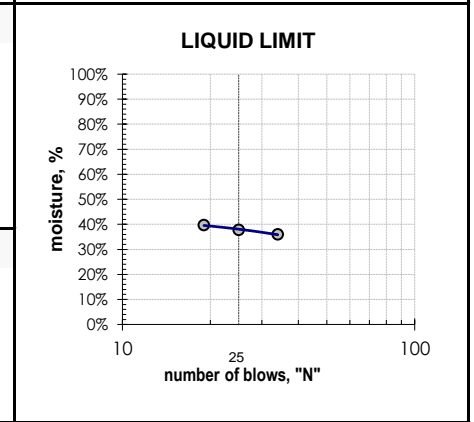
MATERIAL DATA

MATERIAL SAMPLED Lean CLAY with Sand	MATERIAL SOURCE Test Pit, TP-05 depth = 5 feet	USCS SOIL TYPE CL, Lean Clay with Sand
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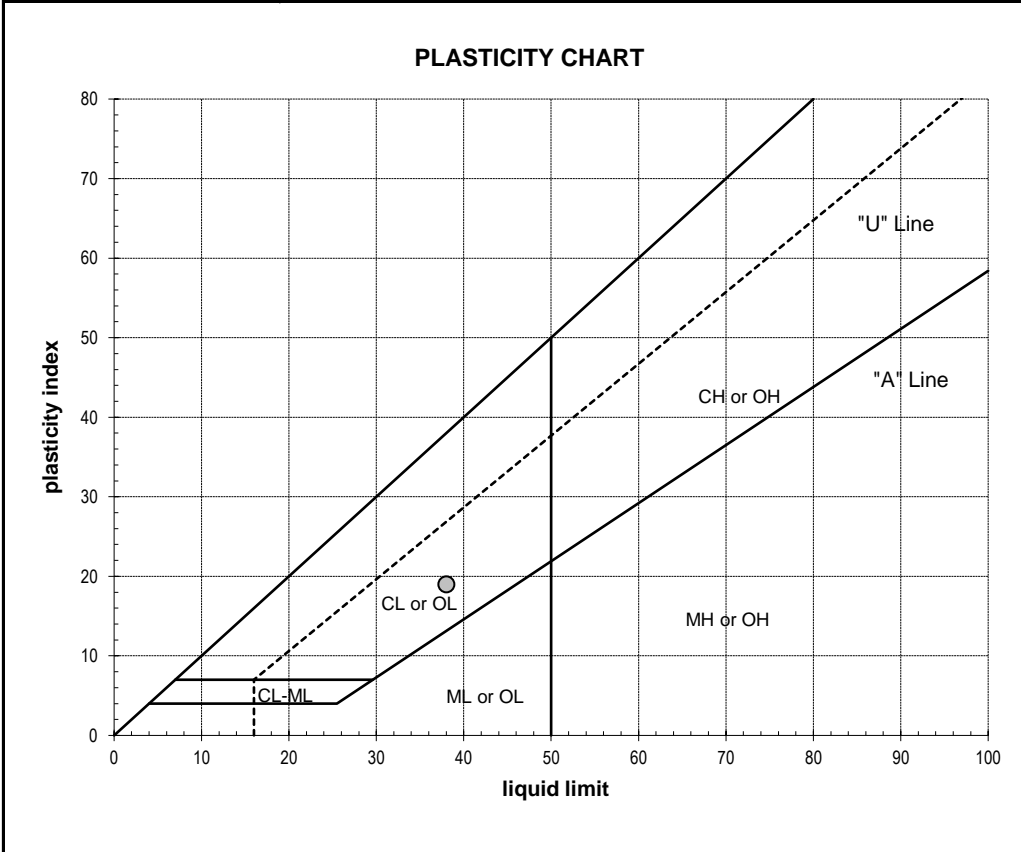
LABORATORY TEST DATA

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

ATTERBERG LIMITS liquid limit = 38 plastic limit = 19 plasticity index = 19	LIQUID LIMIT DETERMINATION <table style="width: 100%; text-align: center;"> <tr> <td></td> <td>1</td> <td>2</td> <td>3</td> <td>4</td> </tr> <tr> <td>wet soil + pan weight, g =</td> <td>33.66</td> <td>33.95</td> <td>33.56</td> <td></td> </tr> <tr> <td>dry soil + pan weight, g =</td> <td>30.27</td> <td>30.38</td> <td>29.87</td> <td></td> </tr> <tr> <td>pan weight, g =</td> <td>20.84</td> <td>20.93</td> <td>20.58</td> <td></td> </tr> <tr> <td>N (blows) =</td> <td>34</td> <td>25</td> <td>19</td> <td></td> </tr> <tr> <td>moisture, % =</td> <td>36.0 %</td> <td>37.8 %</td> <td>39.7 %</td> <td></td> </tr> </table>		1	2	3	4	wet soil + pan weight, g =	33.66	33.95	33.56		dry soil + pan weight, g =	30.27	30.38	29.87		pan weight, g =	20.84	20.93	20.58		N (blows) =	34	25	19		moisture, % =	36.0 %	37.8 %	39.7 %	
	1	2	3	4																											
wet soil + pan weight, g =	33.66	33.95	33.56																												
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SHRINKAGE shrinkage limit = n/a shrinkage ratio = n/a	PLASTIC LIMIT DETERMINATION <table style="width: 100%; text-align: center;"> <tr> <td></td> <td>1</td> <td>2</td> <td>3</td> <td>4</td> </tr> <tr> <td>wet soil + pan weight, g =</td> <td>27.47</td> <td>28.62</td> <td></td> <td></td> </tr> <tr> <td>dry soil + pan weight, g =</td> <td>26.37</td> <td>27.36</td> <td></td> <td></td> </tr> <tr> <td>pan weight, g =</td> <td>20.52</td> <td>20.77</td> <td></td> <td></td> </tr> <tr> <td>moisture, % =</td> <td>18.8 %</td> <td>19.1 %</td> <td></td> <td></td> </tr> </table>		1	2	3	4	wet soil + pan weight, g =	27.47	28.62			dry soil + pan weight, g =	26.37	27.36			pan weight, g =	20.52	20.77			moisture, % =	18.8 %	19.1 %		
	1	2	3	4																						
wet soil + pan weight, g =	27.47	28.62																								
dry soil + pan weight, g =	26.37	27.36																								
pan weight, g =	20.52	20.77																								
moisture, % =	18.8 %	19.1 %																								



ADDITIONAL DATA

% gravel =	0.2%
% sand =	16.8%
% silt and clay =	82.9%
% silt =	n/a
% clay =	n/a
moisture content =	22.8%

DATE TESTED 08/26/21	TESTED BY KMS

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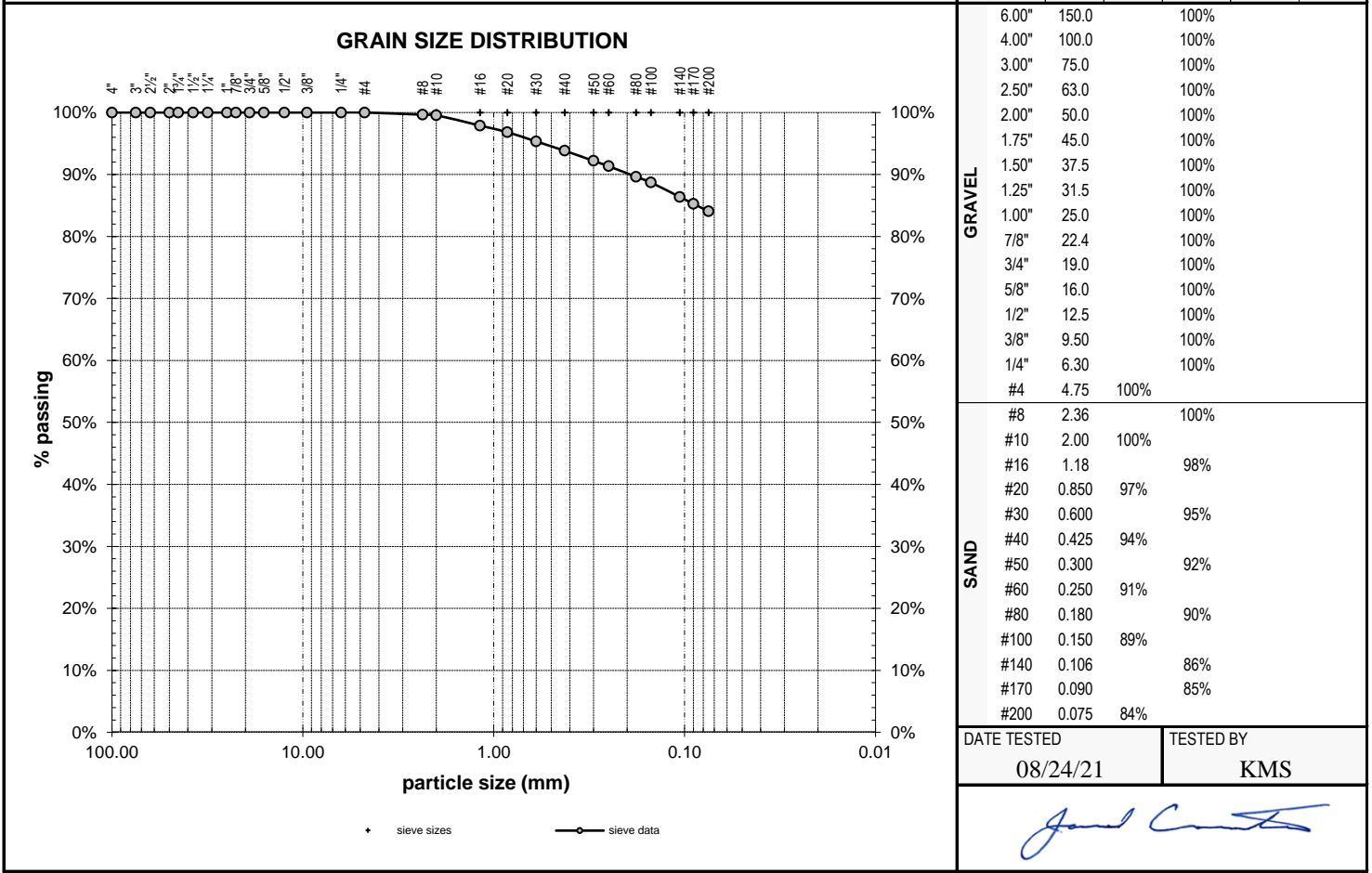
PARTICLE-SIZE ANALYSIS REPORT

PROJECT NW 18th Avenue Subdivision Camas, Washington	CLIENT Modern Dwellings, LLC 8101 NE Glisan Street Portland, Oregon 97213	PROJECT NO. 21184	LAB ID S21-0710
		REPORT DATE 08/27/21	FIELD ID TP6.1
		DATE SAMPLED 08/05/21	SAMPLED BY EMU

MATERIAL DATA		
MATERIAL SAMPLED Lean CLAY with Sand	MATERIAL SOURCE Test Pit, TP-06 depth = 11 feet	USCS SOIL TYPE CL, Lean Clay with Sand
SPECIFICATIONS none		AASHTO CLASSIFICATION A-6(17)

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

ADDITIONAL DATA initial dry mass (g) = 153.53 as-received moisture content = 20.8% liquid limit = 40 plastic limit = 19 plasticity index = 21 fineness modulus = n/a coefficient of curvature, C _C = n/a coefficient of uniformity, C _U = n/a effective size, D ₍₁₀₎ = n/a D ₍₃₀₎ = n/a D ₍₆₀₎ = n/a	SIEVE DATA % gravel = 0.0% % sand = 15.9% % silt and clay = 84.1%
---	---



DATE TESTED 08/24/21	TESTED BY KMS

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ATTERBERG LIMITS REPORT

PROJECT NW 18th Avenue Subdivision Camas, Washington	CLIENT Modern Dwellings, LLC 8101 NE Glisan Street Portland, Oregon 97213	PROJECT NO. 21184	LAB ID S21-0710
		REPORT DATE 08/27/21	FIELD ID TP6.1
		DATE SAMPLED 08/05/21	SAMPLED BY EMU

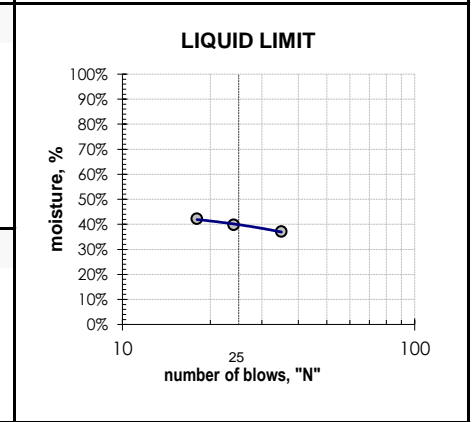
MATERIAL DATA

MATERIAL SAMPLED Lean CLAY with Sand	MATERIAL SOURCE Test Pit, TP-06 depth = 11 feet	USCS SOIL TYPE CL, Lean Clay with Sand
---	---	---

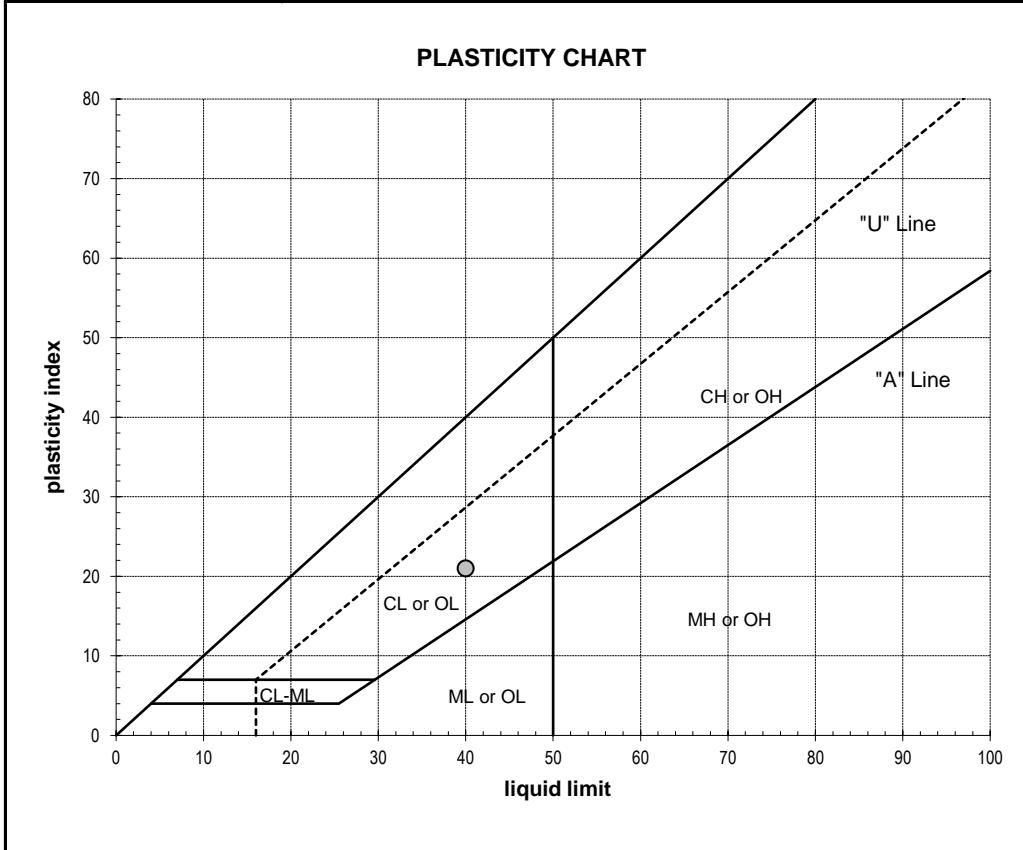
LABORATORY TEST DATA

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

ATTERBERG LIMITS liquid limit = 40 plastic limit = 19 plasticity index = 21	LIQUID LIMIT DETERMINATION <table border="1" style="width: 100%; text-align: center; border-collapse: collapse;"> <thead> <tr> <th></th> <th>1</th> <th>2</th> <th>3</th> <th>4</th> </tr> </thead> <tbody> <tr> <td>wet soil + pan weight, g =</td> <td>33.84</td> <td>33.42</td> <td>33.81</td> <td></td> </tr> <tr> <td>dry soil + pan weight, g =</td> <td>30.35</td> <td>29.87</td> <td>29.99</td> <td></td> </tr> <tr> <td>pan weight, g =</td> <td>20.93</td> <td>20.95</td> <td>20.93</td> <td></td> </tr> <tr> <td>N (blows) =</td> <td>35</td> <td>24</td> <td>18</td> <td></td> </tr> <tr> <td>moisture, % =</td> <td>37.1 %</td> <td>39.8 %</td> <td>42.2 %</td> <td></td> </tr> </tbody> </table>		1	2	3	4	wet soil + pan weight, g =	33.84	33.42	33.81		dry soil + pan weight, g =	30.35	29.87	29.99		pan weight, g =	20.93	20.95	20.93		N (blows) =	35	24	18		moisture, % =	37.1 %	39.8 %	42.2 %	
	1	2	3	4																											
wet soil + pan weight, g =	33.84	33.42	33.81																												
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SHRINKAGE shrinkage limit = n/a shrinkage ratio = n/a	PLASTIC LIMIT DETERMINATION <table border="1" style="width: 100%; text-align: center; border-collapse: collapse;"> <thead> <tr> <th></th> <th>1</th> <th>2</th> <th>3</th> <th>4</th> </tr> </thead> <tbody> <tr> <td>wet soil + pan weight, g =</td> <td>27.71</td> <td>27.12</td> <td></td> <td></td> </tr> <tr> <td>dry soil + pan weight, g =</td> <td>26.61</td> <td>26.08</td> <td></td> <td></td> </tr> <tr> <td>pan weight, g =</td> <td>20.77</td> <td>20.61</td> <td></td> <td></td> </tr> <tr> <td>moisture, % =</td> <td>18.8 %</td> <td>19.0 %</td> <td></td> <td></td> </tr> </tbody> </table>		1	2	3	4	wet soil + pan weight, g =	27.71	27.12			dry soil + pan weight, g =	26.61	26.08			pan weight, g =	20.77	20.61			moisture, % =	18.8 %	19.0 %		
	1	2	3	4																						
wet soil + pan weight, g =	27.71	27.12																								
dry soil + pan weight, g =	26.61	26.08																								
pan weight, g =	20.77	20.61																								
moisture, % =	18.8 %	19.0 %																								



ADDITIONAL DATA	
% gravel =	0.0%
% sand =	15.9%
% silt and clay =	84.1%
% silt =	n/a
% clay =	n/a
moisture content =	20.8%

DATE TESTED 08/26/21	TESTED BY KMS
-------------------------	------------------

Paul Curtis

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

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TEST PIT LOG

PROJECT NAME NW 18th Avenue Subdivision		CLIENT Modern Dwellings, LLC		PROJECT NO. 21184	TEST PIT NO. TP-1
PROJECT LOCATION Camas, Washington		CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN EMU / BJB	DATE 08/05/21
TEST PIT LOCATION See Figure 2		APPROX. SURFACE ELEVATION 744 ft amsl	GROUNDWATER DEPTH Not Encountered	START TIME 0807	FINISH TIME 0833

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 8 to 10 inches of grass and topsoil.					
		Powell Silt Loam		CL		Brown, damp to moist, lean CLAY [Soil Type 1].					
5											
10						Sub-rounded cobbles.					
15						Bottom of test pit at 13.5 feet bgs. Groundwater not observed on 08/05/21.					

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TEST PIT LOG

PROJECT NAME NW 18th Avenue Subdivision		CLIENT Modern Dwellings, LLC		PROJECT NO. 21184	TEST PIT NO. TP-2
PROJECT LOCATION Camas, Washington		CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN EMU / BJB	DATE 08/05/21
TEST PIT LOCATION See Figure 2		APPROX. SURFACE ELEVATION 728 ft amsl	GROUNDWATER DEPTH Not Encountered	START TIME 0845	FINISH TIME 1135

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	TP2.1	Powell Silt Loam	A-6(16)	CL		Approximately 10 to 14 inches of grass and topsoil.	22.7	88.1	37	19	IT2.1 D = 2.0-ft k < 0.06 in/hr
5						Light brown to brown, dry to damp, lean CLAY [Soil Type 1]. Slow digging at 2 feet.					
5						Slow digging at 5 feet.					
15						Bottom of test pit at 13 feet bgs. Groundwater not observed on 08/05/21.					IT2.2 D = 5.0-ft k < 0.06 in/hr

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TEST PIT LOG

PROJECT NAME NW 18th Avenue Subdivision		CLIENT Modern Dwellings, LLC		PROJECT NO. 21184	TEST PIT NO. TP-3
PROJECT LOCATION Camas, Washington		CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN EMU / BJB	DATE 08/05/21
TEST PIT LOCATION See Figure 2		APPROX. SURFACE ELEVATION 732 ft amsl	GROUNDWATER DEPTH Not Encountered	START TIME 0915	FINISH TIME 0940

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 8 to 12 inches of grass and topsoil.					
		Powell Silt Loam		CL		Brown, damp to moist, lean CLAY [Soil Type 1].					
5											
10						Fractured cobbles.					
15						Bottom of test pit at 12 feet bgs. Groundwater not observed on 08/05/21.					

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TEST PIT LOG

PROJECT NAME NW 18th Avenue Subdivision		CLIENT Modern Dwellings, LLC		PROJECT NO. 21184	TEST PIT NO. TP-4
PROJECT LOCATION Camas, Washington		CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN EMU / BJB	DATE 08/05/21
TEST PIT LOCATION See Figure 2		APPROX. SURFACE ELEVATION 706 ft amsl	GROUNDWATER DEPTH Not Encountered	START TIME 0942	FINISH TIME 1123

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 10 to 14 inches of grass and topsoil.					
		Powell Silt Loam		CL		Light brown to brown, damp to moist, lean CLAY [Soil Type 1].					
5						Slow digging at 4 feet.					
10						Sub-rounded cobbles, round 4-12" cobbles					
15						Bottom of test pit at 12.5 feet bgs. Groundwater not observed on 08/05/21.					

IT4.1

 D = 4.0-ft
 k < 0.06 in/hr

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TEST PIT LOG

PROJECT NAME NW 18th Avenue Subdivision		CLIENT Modern Dwellings, LLC		PROJECT NO. 21184	TEST PIT NO. TP-5
PROJECT LOCATION Camas, Washington		CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN EMU / BJB	DATE 08/05/21
TEST PIT LOCATION See Figure 2		APPROX. SURFACE ELEVATION 694 ft amsl	GROUNDWATER DEPTH Not Encountered	START TIME 1015	FINISH TIME 1215

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 10 to 14 inches of grass and topsoil.					
		Powell Silt Loam		CL		Gray to light brown, dry to damp, CLAY with sand [Soil Type 1]. Slow digging at 2 feet.					IT5.1 D = 2.0-ft k < 0.06 in/hr
5	TP5.2		A-6(15)			Slow digging at 5 feet.	22.8	82.9	38	19	IT5.2 D = 5.0-ft k < 0.06 in/hr
10						Sub-rounded cobbles.					
15						Bottom of test pit at 13 feet bgs. Groundwater not observed on 08/05/21.					

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TEST PIT LOG

PROJECT NAME NW 18th Avenue Subdivision		CLIENT Modern Dwellings, LLC		PROJECT NO. 21184	TEST PIT NO. TP-6
PROJECT LOCATION Camas, Washington		CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN EMU / BJB	DATE 08/05/21
TEST PIT LOCATION See Figure 2		APPROX. SURFACE ELEVATION 724 ft amsl	GROUNDWATER DEPTH Not Encountered	START TIME 1045	FINISH TIME 1112

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 8 to 14 inches of grass and topsoil.					
5		Powell Silt Loam		CL		Light brown to brown, dry to damp, lean CLAY with sand [Soil Type 1].					
10	TP6.1		A-6(17)			Becomes orange, brown and damp to moist Sub-rounded cobbles.	20.8	84.1	40	21	
15						Bottom of test pit at 12 feet bgs. Groundwater not observed on 08/05/21.					

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)		
	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
<u>Characteristics of fraction passing 0.425 mm (No. 40)</u>							
Liquid limit				40 max	41 min	40 max	41 min
Plasticity index	6 max	N.P.		10 max	10 max	11 min	11 min
General rating as subgrade	Excellent to good				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.

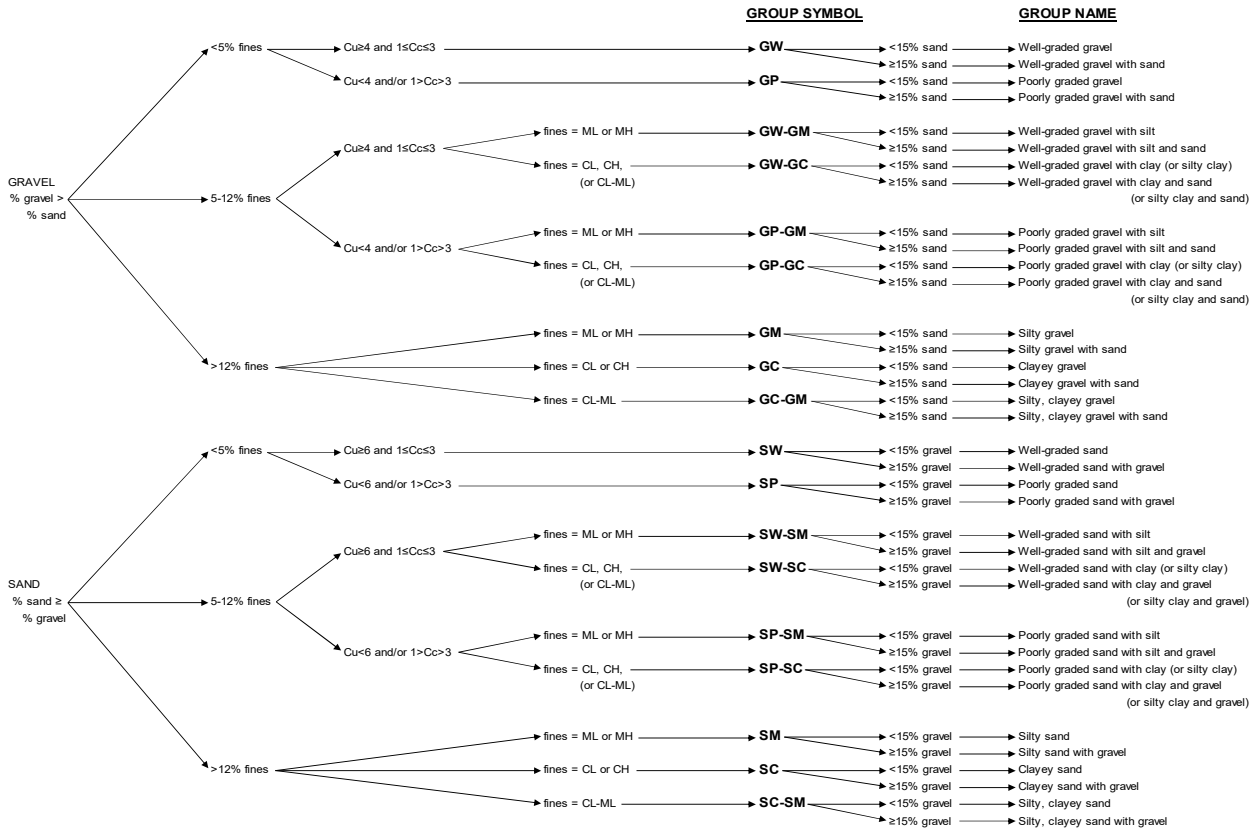
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

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
<u>Characteristics of fraction passing 0.425 mm (No. 40)</u>											
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	11 min
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			

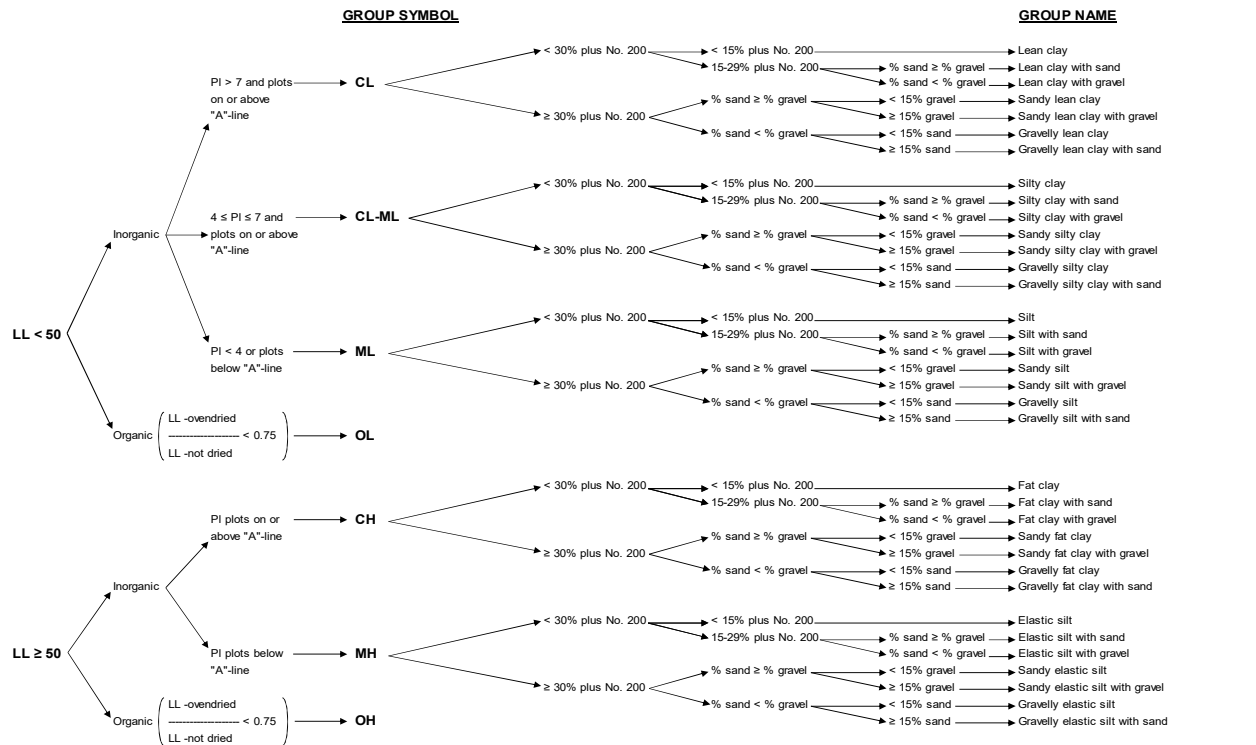
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

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**

NW 18th Avenue Subdivision

August, 2021

Camas, Washington



Test Pit Profile, TP-1

NW 18th Avenue Subdivision

August, 2021

Camas, Washington



Test Pit Profile, TP-3

NW 18th Avenue Subdivision

August, 2021

Camas, Washington



Test Pit Profile, TP-5

NW 18th Avenue Subdivision

August, 2021

Camas, Washington



TP-2 Site Area, Facing Southeast

NW 18th Avenue Subdivision

August, 2021

Camas, Washington



Central Site Area, Facing West

NW 18th Avenue Subdivision

August, 2021

Camas, Washington



Southwest Site Area, Facing Northeast

NW 18th Avenue Subdivision

August, 2021

Camas, Washington



Eastern Site Area, Facing South

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION



Date: September 15, 2021
Project: NW 18th Avenue Subdivision
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.

Report Ownership

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

Consultant Responsibility

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