Project 2018-254 Clark College, Advanced Manufacturing Center

Attachment 6b

GeoTech Report

Note: The exploratory test pits dug for this investigation are at least 300 feet remote from the proposed Advanced Manufacturing Center site.

Geotechnical Site Investigation

Clark College
North County Campus

Ridgefield, Washington

January 10, 2019

Geotechnical ▪ Environmental ▪ Special Inspections

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GEOTECHNICAL SITE INVESTIGATION
CLARK COLLEGE NORTH COUNTY CAMPUS
RIDGEFIELD, WASHINGTON

Prepared For:	**Mr. Jim Watkins** **Clark College** **1933 Fort Vancouver Way** **Vancouver, Washington 98663**
Site Location:	**266 N 65th Avenue** **Parcel Nos. 214197000, 214196000, and 214247000** **Ridgefield, Washington**
Prepared By:	**Columbia West Engineering, Inc.** **11917 NE 95th Street** **Vancouver, Washington 98682** **Phone: 360-823-2900** **Fax: 360-823-2901**
Date Prepared:	**January 10, 2019**

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

GEOTECHNICAL SITE INVESTIGATION
CLARK COLLEGE NORTH COUNTY CAMPUS
RIDGEFIELD, WASHINGTON

1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Clark College to conduct a geotechnical site investigation for the proposed Clark College North County Campus project located in Ridgefield, 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 executed November 6, 2018. 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 6.0, *Conclusion and Limitations*, and Appendix E.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site consists of an approximate 5.6-acre study area located at 266 N 65th Avenue in Ridgefield, Washington and is comprised of portions of tax parcel numbers 214197000, 214196000, and 214247000. The approximate latitude and longitude are N 45° 49’ 00” and W 122° 40’ 41”, and the legal description is a portion of the NW and SW ¼ of Section 22, T4N, R1E, Willamette Meridian. The regulatory jurisdictional agency is the City of Ridgefield.

1.2 Proposed Development

Correspondence with the design team indicates proposed development will consist of a new college facility building. Proposed development is indicated on Figure 2A. Development will also include asphalt concrete drive aisles and parking, underground utilities, and stormwater management facilities. Columbia West has not reviewed preliminary grading plans but understands that cut and fill will likely be proposed at the subject site. This report is based upon proposed development as described above and may not be applicable if modified.

2.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

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

According to the *Geologic Map of the Ridgefield Quadrangle, Clark and Cowlitz Counties, Washington* (Russell C. Evarts, USGS Geological Survey, 2004), near-surface soils are expected to consist of upper-Pleistocene, rhythmically bedded, fine-grained periglacial deposits derived from catastrophic outburst floods of Glacial Lake Missoula (Qfs).

The *Web Soil Survey* (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2018 Website) identifies surface soils Odne silt loam and Gee silt loam. Although soil conditions may vary from the broad USDA descriptions, Gee and Odne series soils are generally fine-textured clays and silts with very low permeability, moderate to high water capacity, and low shear strength. Gee and Odne soils are generally moisture sensitive, somewhat compressible, and described as having low to moderate shrink-swell potential. The erosion hazard is slight primarily based upon slope grade.

3.0 REGIONAL SEISMOLOGY

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

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

<u>Portland Hills Fault Zone</u>

The Portland Hills Fault Zone consists of several northwest-trending faults located along the northeastern margin of the Tualatin Mountains, also known as the Portland Hills, and the southwest margin of the Portland Basin. The fault zone is approximately 25 to 30 miles in length and is located approximately 14 miles southwest of the site. According to *Seismic Design Mapping, State of Oregon* (Geomatrix Consultants, 1995)*,* there is no definitive consensus among geologists as to the zone fault type. Several alternate interpretations have been suggested.

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

However, evidence suggests that fault movement has impacted shallow Holocene deposits and deeper Pleistocene sediments. Seismologists recorded a M3.2 earthquake thought to be associated with the fault zone near Kelly Point Park in November 2012, a M3.9

earthquake thought to be associated with the fault zone near Kelly Point Park in April 2003, and a M3.5 earthquake possibly associated with the fault zone occurred 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.

<u>Gales Creek-Newberg-Mt. Angel Fault Zone</u>

Located approximately 38 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, but 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.

<u>Lacamas Lake-Sandy River Fault Zone</u>

The northwest-trending Lacamas Lake Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 20 miles southeast of the site, and form part of the northeastern margin of the Portland basin. According to *Geology and Groundwater Conditions of Clark County Washington* (USGS Water Supply Paper 1600, Mundorff, 1964) and the *Geologic Map of the Lake Oswego Quadrangle* (Oregon DOGAMI Series GMS-59, 1989), the Lacamas Lake fault zone consists of shear contact between the Troutdale Formation and underlying Oligocene andesite-basalt bedrock. Secondary shear contact associated with the fault zone may have produced a series of prominent northwest-southeast geomorphic lineaments in proximity to the site.

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

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

Cascadia Subduction Zone

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

4.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance, five test pits (TP-1 through TP-5), two soil borings (SB-1 and SB-2), and two cone penetrometer test (CPT) soundings (CPT-1 and CPT-2) was conducted at the site on November 20, 27, and 30, 2018. Test pits were explored with a track-mounted excavator. Soil borings were drilled with track-mounted mud-rotary apparatus. CPT soundings were advanced with track- and truck-mounted CPT apparatus. 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 subject site consists of an approximate 5.6-acre study area in portions of three parcels located at 266 N 65th Avenue in Ridgefield, Washington. Site observations during exploration indicate the proposed development area is currently utilized for agriculture. The site is bounded by agricultural terrain to the north and east and grassy open areas sparsely vegetated with coniferous and deciduous trees to the south and west. As indicated in Appendix D, drainage ditches with apparent surface water were observed at the south and west borders of parcel numbers 214247000 and 214196000. Field reconnaissance and review of site topographic mapping indicate relatively flat to gently rolling terrain with site grades ranging from 0 to 5 percent. Site elevations in the proposed development area range from 270 feet amsl at the northeast boundary of parcel number 214247000 to 284 feet amsl in at the west boundary of parcel 214197000.

4.2 Subsurface Exploration and Investigation

Soil borings were drilled to a maximum depth of 51 ½ feet below ground surface (bgs). CPT soundings were advanced to a maximum depth of 91 feet bgs. Test pits were explored to a maximum depth of 15 feet 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 16 to 18 inches of sod and topsoil in observed locations. Underlying topsoil layers, subsurface soils resembling native USDA Gee and Odne soil series descriptions were encountered. Subsurface lithology was reasonably consistent at explored locations and may generally be described by soil types identified in the following text.

Soil Type 1 – Lean CLAY / Lean CLAY with Sand / Sandy Lean CLAY / Fat CLAY

Soil Type 1 was observed to primarily consist of varicolored, lightly to heavily mottled, moist to very moist, medium stiff to very stiff lean CLAY, lean CLAY with sand, sandy lean CLAY, and fat CLAY. Soil Type 1 was observed below the topsoil layer in explored areas and extended to the maximum depth of exploration in test pits TP-1 through TP-5 and soil boring SB-2. Soil Type 1 extended to approximately 40 feet bgs in soil boring SB-1 where it was underlain by Soil Type 2.

Analytical laboratory testing conducted upon representative soil samples obtained explorations TP-4, SB-1, and SB-2 indicated 64.8 to 94.5 percent by weight passing the No. 200 sieve and in situ moistures ranging from 27.3 to 33.3 percent. Atterberg Limits analysis indicated liquid limits ranging from 35 to 55 percent and plasticity indices ranging from 13 to 33 percent. Laboratory tested samples of Soil Type 1 are classified CL and CH according to USCS specifications and A-6(13), A-6(10), A-7-6(29), A-7-6(28), A-7-6(27), A-7-6(33), and A-7-6(11) according to AASHTO specifications.

Soil Type 2 – Silty Clayey SAND

Soil Type 2 was observed to primarily consist of heavily mottled red to orange-brown, moist, medium dense to dense silty clayey SAND. Soil Type 2 was observed below Soil Type 1 in soil boring SB-1 and extended to the maximum depth of exploration.

Analytical laboratory testing conducted upon a representative soil sample obtained from soil boring SB-1 indicated 38.9 percent by weight passing the No. 200 sieve and an in situ moisture content of 19.8 percent. Atterberg Limits analysis indicated a liquid limit of 24 percent and a plasticity index of 6 percent. The laboratory tested sample of Soil Type 2 is classified SC-SM according to USCS specifications and A-4(0) according to AASHTO specifications.

4.2.2 Groundwater

Groundwater was not observed within subsurface explorations to a maximum explored depth of 51 ½ feet bgs. As previously indicated, surface water was observed in drainage ditches at the south and west boundaries of parcel numbers 214247000 and 214196000. Based on review of hydric soil mapping and wetland delineation on *Clark County Maps Online* and previously conducted geotechnical explorations at nearby parcels, Columbia West anticipates shallow seasonal groundwater conditions in the general vicinity of the site during wet weather months. Groundwater levels are often subject to seasonal variance and may rise during extended periods of increased precipitation. Perched groundwater may also be present in localized areas. Seeps and springs may become evident during site grading, primarily along slopes or in areas cut below existing grade. Structures, roads, and drainage design should be planned accordingly.

5.0 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 proposed development are shallow seasonal groundwater and fine-textured soil. Design recommendations are presented in the following text sections.

5.1 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 16 and 18 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. Demolition work prior to site improvements construction may generate unsuitable fill and disturbed soils in areas of old foundations, basement walls, utilities, and debris. These materials should also be thoroughly removed from structural areas and backfilled with engineered structural fill.

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

Site grading activities should be performed in accordance with requirements specified in the *2015 International Building Code* (IBC), Chapter 18 and Appendix J, with exceptions noted in the text herein. Site preparation, soil stripping, and grading activities should be observed and documented by Columbia West.

5.2 Engineered Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in the preceding text. Surface soils should then be scarified and compacted prior to additional fill placement. Engineered structural fill should be placed in loose lifts not exceeding 12 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within two percentage points of optimum conditions. A field density at least equal to 95 percent of the maximum dry density, obtained from the standard Proctor moisture-density relationship test (ASTM D698), is recommended for structural fill placement. Engineered structural fill placed on sloped grades should be benched to provide a horizontal surface for compaction.

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

Engineered structural fill placement activities should be performed during dry summer months if possible. Most clean fine-textured native soils may be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native soils with a plasticity index greater than 25 (portions of Soil Type 1) should be evaluated and approved by Columbia West prior to use as structural fill. 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.

5.3 Cut and Fill Slopes

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

Final cut or fill slopes at the site should not exceed 2H:1V or 20 feet in height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three

(H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 4.

Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Fill slopes should be constructed by placing fill material in maximum 12-inch level lifts, compacting as described in Section 5.2, *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.

5.4 Foundations

Building foundations 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 5 kips per foot for perimeter footings or 150 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 5.1, *Site Preparation and Grading*, and Section 5.2, *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 2,000 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 1.5H:1V angle projected down from the outside bottom footing edge without additional geotechnical analysis.

Foundations should not be permitted to bear upon existing fill or disturbed soil. Because soil is often heterogeneous and anisotropic, Columbia West should observe foundation excavations prior to placing forms or reinforcing bar to verify subgrade support conditions are as anticipated in this report.

5.4.1 Luminaire, Signal, and Sign Foundations

Foundations for luminaire, signal, and sign poles should be designed in accordance with the *International Building Code (IBC) Chapter 18* by a licensed structural engineer. Based upon review of *IBC* literature, and SPT blow count observations made during the field exploration, the allowable lateral bearing pressure for foundations installed in competent native soils or engineered structural fill is 100 psf/ft up to a maximum of 1,500 psf. Columbia West should be contacted to review foundation designs and evaluate compatibility with geotechnical design assumptions.

5.5 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. The modulus of subgrade reaction is estimated to be 100 psi/inch.

Preparation and compaction beneath slabs should be performed in accordance with the recommendations presented in Section 5.1, *Site Preparation and Grading* and Section 5.2, *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. 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.

5.6 Static Settlement

Based upon the anticipated structural loading and allowable soil bearing pressure described above, Columbia West analyzed estimated static settlement for the proposed structure. Settlement analysis was conducted using Schmertmann’s (1970, 1978) method to calculate vertical foundation displacement using CPT results.

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.

The resulting vertical displacement after loading may be due to elastic distortion, dissipation of excess pore pressure, or soil creep. Expansion of subgrade may also occur due to uplift rebound forces after unloading of native soils in deep cut areas. Settlement described above pertains to static loads and does not consider vertical displacements caused by cyclic loading. Settlement risk related to potential seismic events and dynamic loading is discussed in Section 5.11, *Soil Liquefaction and Dynamic Settlement*.

5.7 Excavation

Soil borings were drilled with track-mounted mud-rotary drill apparatus to a maximum depth of 51 ½ feet bgs. Blasting or specialized rock-excavation techniques are not anticipated. As indicated previously in Section 4.2.2, *Groundwater,* surface water was observed at the west and south boundaries of parcels 214247000 and 214196000. Groundwater levels are often subject to seasonal variance and may rise during extended periods of increased precipitation. Recommendations as described in Section 5.8, *Dewatering* should be considered in locations where subsurface construction activities intersect 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 in no case should excavation be conducted in excess of all applicable local, state, and federal laws.

5.8 Dewatering

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

Significant pumping and dewatering may be required to temporarily reduce the groundwater elevation to allow construction of proposed below-grade structures, installation of utilities, or placement of structural fills. Dewatering via a sump within excavation zones may be insufficient to control groundwater and provide excavation side slope stability. Dewatering may be more feasibly conducted by installing a system of temporary well points and pumps around proposed excavation areas or utility trenches. Depending on proposed utility depths, a site-specific dewatering plan may be necessary. Well pumps should remain functioning at all times during the excavation and construction period. Suitable back-up pumps and power supplies should be available to prevent unanticipated shut-down of dewatering equipment. Failure to operate pumps full-time may result in flooding of the excavation zones, resulting in damage to forms, slopes, or equipment.

5.9 Lateral Earth Pressure

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

The design parameters presented in Table 1 are valid for static loading cases only and are based upon in situ soils or compacted granular fill. The recommended earth pressures do not include surcharge loads, dynamic loading, hydrostatic pressure, or seismic design.

If seismic design is required for unrestrained walls, seismic forces may be calculated by superimposing a uniform lateral force of $10H^2$ pounds per lineal foot of wall, where H is the total wall height in feet. The resultant force should be applied at 0.6H from the base of the wall. If sloped backfill conditions are proposed for the site, Columbia West should be contacted for additional analysis and associated recommendations.

Table 1. Lateral Earth Pressure Parameters for Level Backfill

Retained Soil	Equivalent Fluid Pressure for Level Backfill			Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive		
Undisturbed native Lean CLAY / Lean CLAY with Sand / Fat CLAY / Sandy Lean CLAY (Soil Type 1)	62 pcf	43 pcf	282 pcf	110 pcf	26°
Approved Structural Backfill Material WSDOT 9-03.12(2) compacted aggregate backfill	52 pcf	32 pcf	568 pcf	135 pcf	38°

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

A continuous one-foot-thick zone of free-draining, washed, open-graded 1-inch by 2-inch drain rock and a 4-inch perforated gravity drain pipe is assumed behind retaining walls. Geotextile filter fabric should be placed between the drain rock and backfill soil. Specifications for drainpipe design are presented in Section 5.12, *Drainage*. If walls cannot be gravity drained, saturated base conditions and/or applicable hydrostatic pressures should be assumed.

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

5.10 Seismic Design Considerations

According to the *United States Geologic Survey (USGS) 2012 Seismic Design Maps Summary Report,* 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	0.39 g
0.2 sec Spectral Acceleration	0.90 g
1.0 sec Spectral Acceleration	0.40 g

The listed probabilistic ground motion values are based upon "firm rock" sites with an assumed shear wave velocity of 2,500 ft/s in the upper 100 feet of soil profile. These values should be adjusted for site class effects by applying site coefficients Fa and Fv as defined in *2015 IBC Tables 1613.3.3(1)* and *(2)*; the PGA should be adjusted by applying the site coefficient F_{PGA} as defined by *ASCE 7, Chapter 11, Table 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.

The *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), indicates site soils may be represented by Site Class C as defined in 2015 IBC Section 1613.3.5. However, subsurface exploration, in situ soil testing, and review of local well logs and geologic maps indicates that site soils exhibit characteristics of Site Class D. This site class designation indicates that some amplification of seismic energy may occur during a seismic event because of subsurface conditions.

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

Identification of specific seismic response spectra is beyond the scope of this investigation. If site structures are designed in accordance with recommendations specified in the *2015 IBC*, the potential for peak ground accelerations in excess of the adjusted and amplified values should be understood.

5.11 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 and lateral spreading.

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

Based upon results of laboratory analysis, site-specific testing, and groundwater table observations, most observed site soils do not meet the criteria described above for liquefiable soils. Therefore, the potential for liquefaction of site soils is considered to be low. If a liquefaction-inducing event occurs, the total estimated liquefaction-induced settlement is not anticipated to exceed approximately one inch. Differential liquefaction-induced settlement is not anticipated to exceed approximately ½ inch over a span of 50 feet.

5.12 Drainage

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. Drainage design in general should conform to City of Ridgefield 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 the stormwater system or approved discharge location.

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

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

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

Foundation drains and subdrains 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.

5.13 Bituminous Asphalt and Portland Cement Concrete

Based upon correspondence with the client, proposed development will include new private asphalt concrete parking and drive aisles. Columbia West recommends adherence to City of Ridgefield paving guidelines for right-of-way improvements unless a site-specific pavement design is conducted. General recommendations and associated specifications for proposed private flexible pavement structures are provided in Table 4.

Table 4. Flexible Pavement Section Recommendations

Pavement Section Layer	Minimum Layer Thickness		Specifications
	Passenger Car Parking and Access Drives*	Heavy Vehicle Parking and Access Drives**	
Asphalt concrete surface HMA Class ½" PG 64-22	3 inches	4 inches	91 percent of maximum Rice density (ASTM D2041)
Base course (WSDOT 9-03.9(3) 1¼"-0 crushed aggregate	6 inches	10 inches	Compacted to 95 percent of maximum modified Proctor density (ASTM D1557)
Scarified and compacted existing subgrade material	12 inches	12 inches	Compacted to 95 percent of maximum standard Proctor density (ASTM D1557)

* Design recommendations assume that passenger car parking and access drives are not subject to bus or other heavy traffic.

** General recommendations based up on maximum traffic loading of up to 30 heavy vehicles or busses per day. If actual truck traffic substantially exceeds 30 trucks per day, reduced pavement serviceability and design life should be expected.

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 5.14, *Wet Weather Construction Methods and Techniques*. Subgrade conditions should be evaluated and tested by Columbia West prior to placement of crushed aggregate base. Subgrade evaluation should include nuclear gauge density testing and wheel proof-roll observations conducted with a 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 Ridgefield specifications.

Portland cement concrete curbs and sidewalks should be installed in accordance with City of Ridgefield 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 specimens 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.

5.14 Wet Weather Construction Methods and Techniques

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

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

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

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

Crushed aggregate base should be compacted to at least 95 percent of maximum dry density according to the modified Proctor density test (ASTM D1557). Compaction should be verified by nuclear gauge density testing. Observation of a proof-roll with a loaded dump truck is also recommended as an indication of the compacted aggregate's performance.

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

5.15 Erosion Control Measures

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

5.16 Soil Shrink/Swell Potential

Based upon laboratory analysis, near-surface soils may contain as much as 94.5 percent by weight passing the No. 200 sieve and exhibit plasticity indices ranging from 13 to 33 percent. This indicates the potential for soil shrinking or swelling and underscores the importance of proper moisture conditioning during fill placement. Medium to high plasticity soils should be placed and compacted at a moisture content approximately two percent above optimum as determined by laboratory analysis.

5.17 Utility Installation

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

Utilities should be installed in general accordance with manufacturer's recommendations. Utility trench backfill should consist of *WSDOT 9-03.19 Bank Run Gravel for Trench Backfill* or *WSDOT 9-03.14(2) Select Borrow* with a maximum particle size of 2 ½-inches. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). The remaining backfill should be compacted to at least 95 percent of maximum dry density as determined by the standard Proctor moisture-density test (ASTM D698). Clean, free-draining, fine bedding sand is recommended for use in the pipe zone. With exception of the pipe zone, backfill should be placed in loose lifts not exceeding 12 inches in thickness.

Compaction of utility trench backfill material should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D6938. It is recommended that field compaction testing 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.

6.0 CONCLUSION AND LIMITATIONS

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

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

This report is not an environmental assessment and should not be construed as a representative warranty of site subsurface conditions. The discovery of adverse environmental conditions, or subsurface soils that deviate significantly 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.

Daniel E. Lehto, PE, GE
Principal

DANIEL E. LEHTO
STATE OF WASHINGTON
44702
REGISTERED
PROFESSIONAL ENGINEER

1-10-19

Exp. 6-5-19

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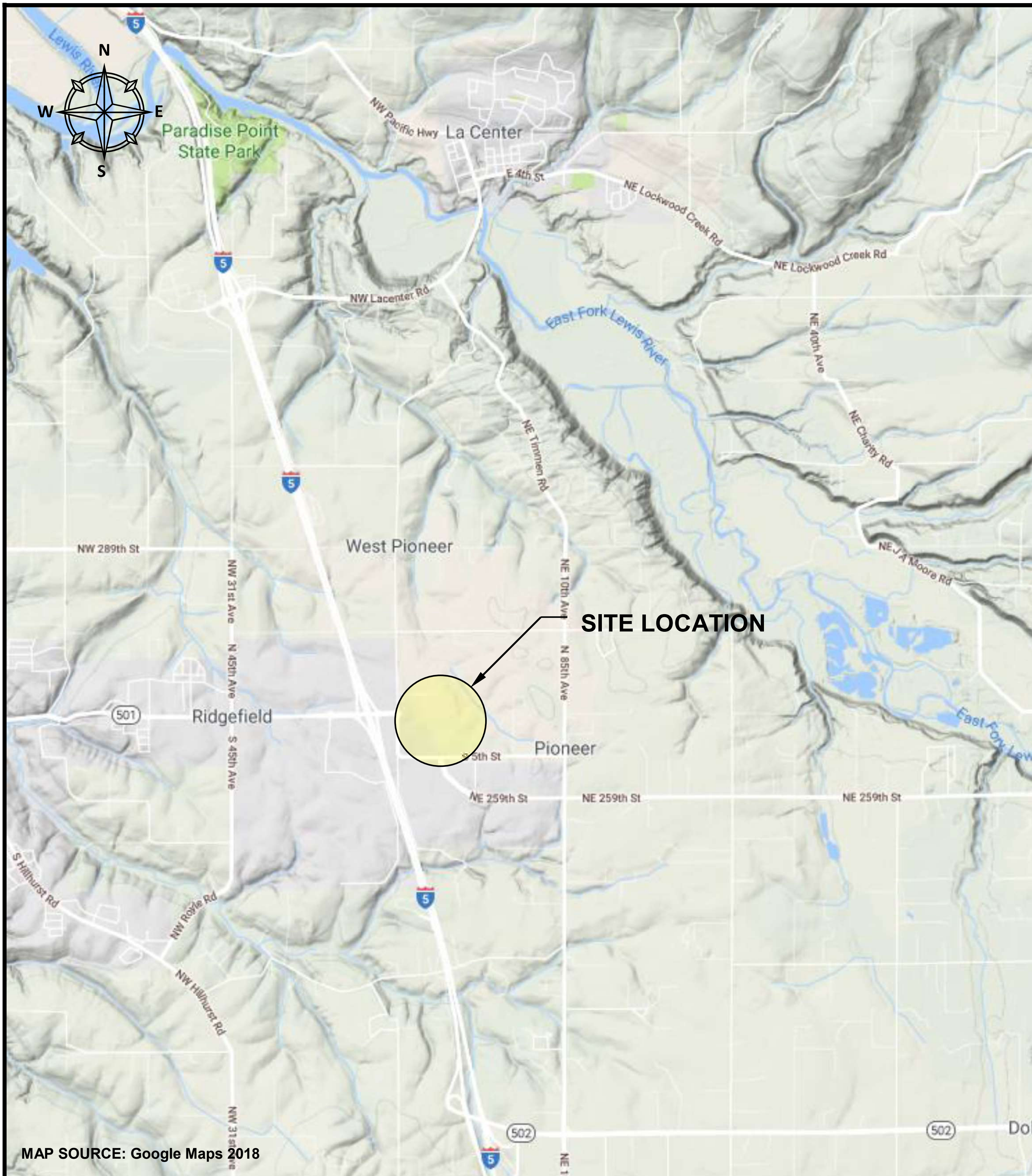
FIGURES



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Design	Drawn: HDG			
Checked: GLW	Date: 11/27/18			
Client: CLARK	Rev	By	Date	
Job No.: 18345				
CAD File: FIGURE 1				
Scale: NTS				

SITE LOCATION MAP

CLARK COLLEGE
NORTH COUNTY CAMPUS
RIDGEFIELD, WASHINGTON

FIGURE 1

NOTES:
1. SITE LOCATION: 266 N 65TH AVENUE, RIDGEFIELD, WASHINGTON.
2. SITE STUDY AREA CONSISTS OF PORTIONS OF PARCEL NOS. 214197000, 214196000, AND 214247000 TOTALING APPROXIMATELY 5.6 ACRES.
3. DRAWING IS NOT TO SCALE.
4. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
5. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
7. TEST PITS BACKFILLED LOOSELY WITH ONSITE SOILS ON NOVEMBER 27, 2018.
8. SOIL BORINGS BACKFILLED WITH BENTONITE ON NOVEMBER 30, 2018.

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Job No: 18345				
CAD File: FIGURE 2				
Scale: NONE				

EXPLORATION LOCATION MAP

CLARK COLLEGE
NORTH COUNTY CAMPUS
RIDGEFIELD, WASHINGTON

FIGURE 2

NOTES:
1. BASE MAP PROVIDED BY SCHREIBER STARLING WHITEHEAD ARCHITECTS.

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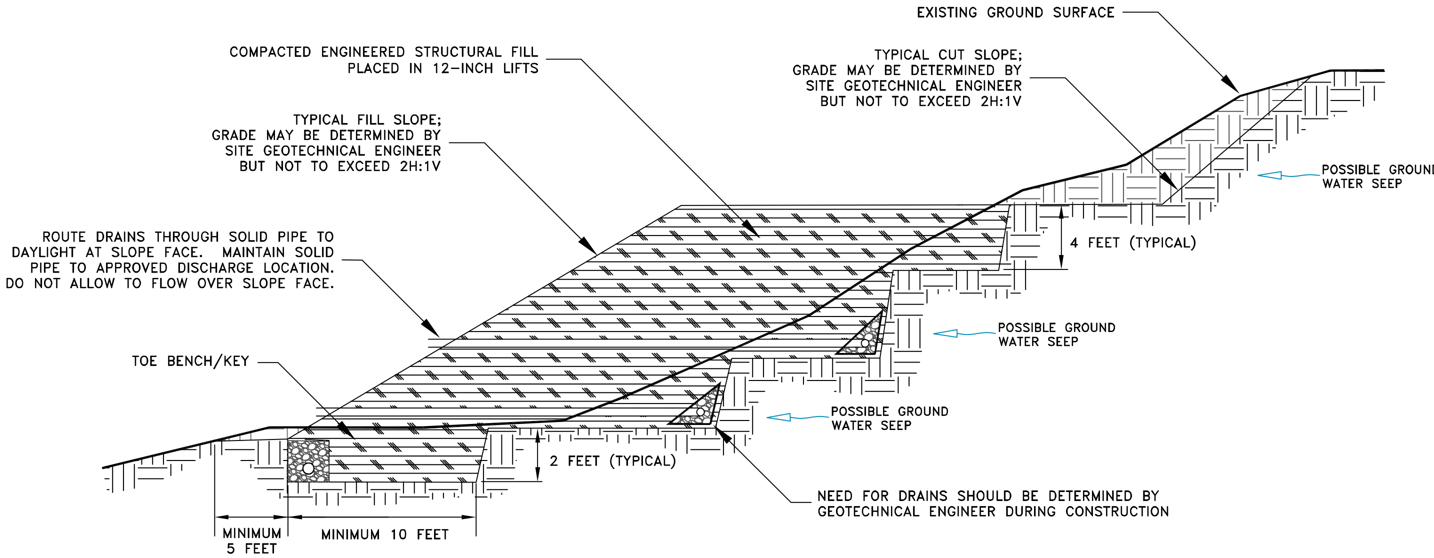
CAD File: FIGURE 2A			
Scale: NONE			

PROPOSED SITE PLAN

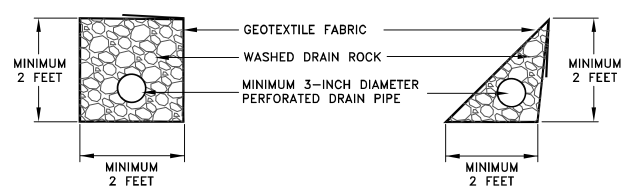
CLARK COLLEGE
NORTH COUNTY CAMPUS
RIDGEFIELD, WASHINGTON

FIGURE
2A

TYPICAL CUT AND FILL SLOPE CROSS-SECTION



TYPICAL DRAIN SECTION DETAIL



DRAIN SPECIFICATIONS

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

WASHED DRAIN ROCK SHALL BE OPEN-GRADED ANGULAR DRAIN ROCK WITH LESS THAN 2 PERCENT PASSING THE No. 200 SIEVE AND A MAXIMUM PARTICLE SIZE OF 3 INCHES.

NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FILL AND CUT SLOPE SECTION, AND MAY NOT BE SITE-SPECIFIC.

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Design:	Drawn: HDG			TYPICAL CUT AND FILL SLOPE CROSS-SECTION	FIGURE 3
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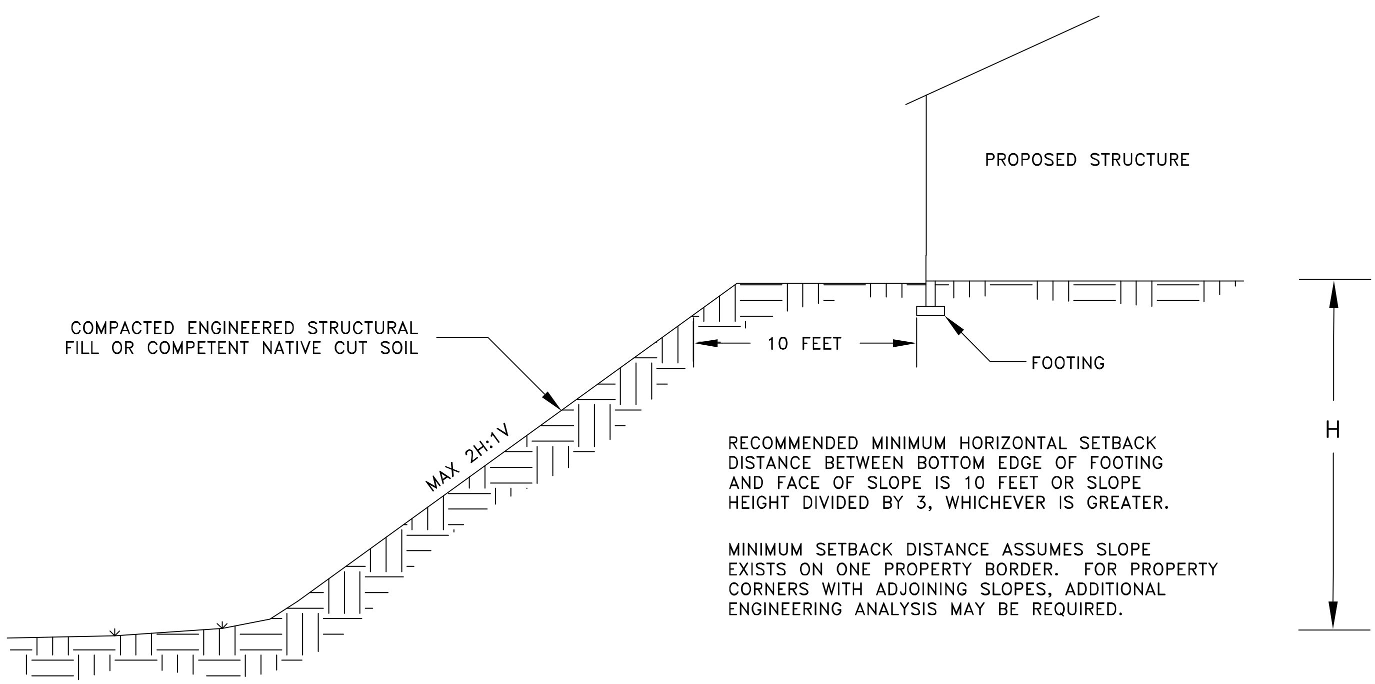
Client: CLARK	Rev	By	Date		
Job No: 18345				CLARK COLLEGE NORTH COUNTY CAMPUS RIDGEFIELD, WASHINGTON	
CAD File: FIGURE 3					
Scale: NONE					

MINIMUM FOUNDATION SLOPE SETBACK DETAIL



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

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Design:	Drawn: HDG	TYPICAL MINIMUM SLOPE SETBACK DETAIL	FIGURE 4
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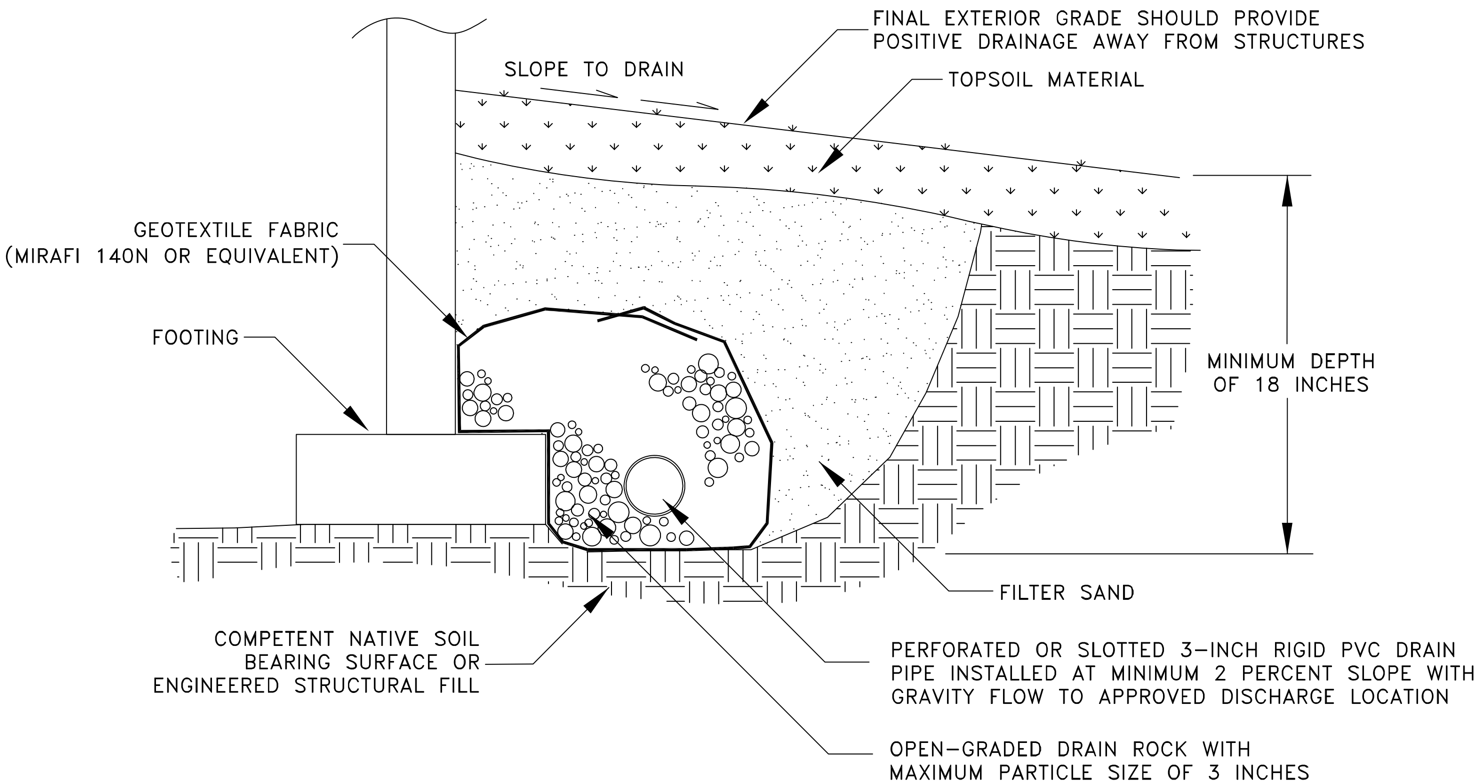
Job No: 18345		CLARK COLLEGE NORTH COUNTY CAMPUS	
CAD File: FIGURE 4		RIDGEFIELD, WASHINGTON	
Scale: NONE			

TYPICAL PERIMETER FOOTING DRAIN DETAIL



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

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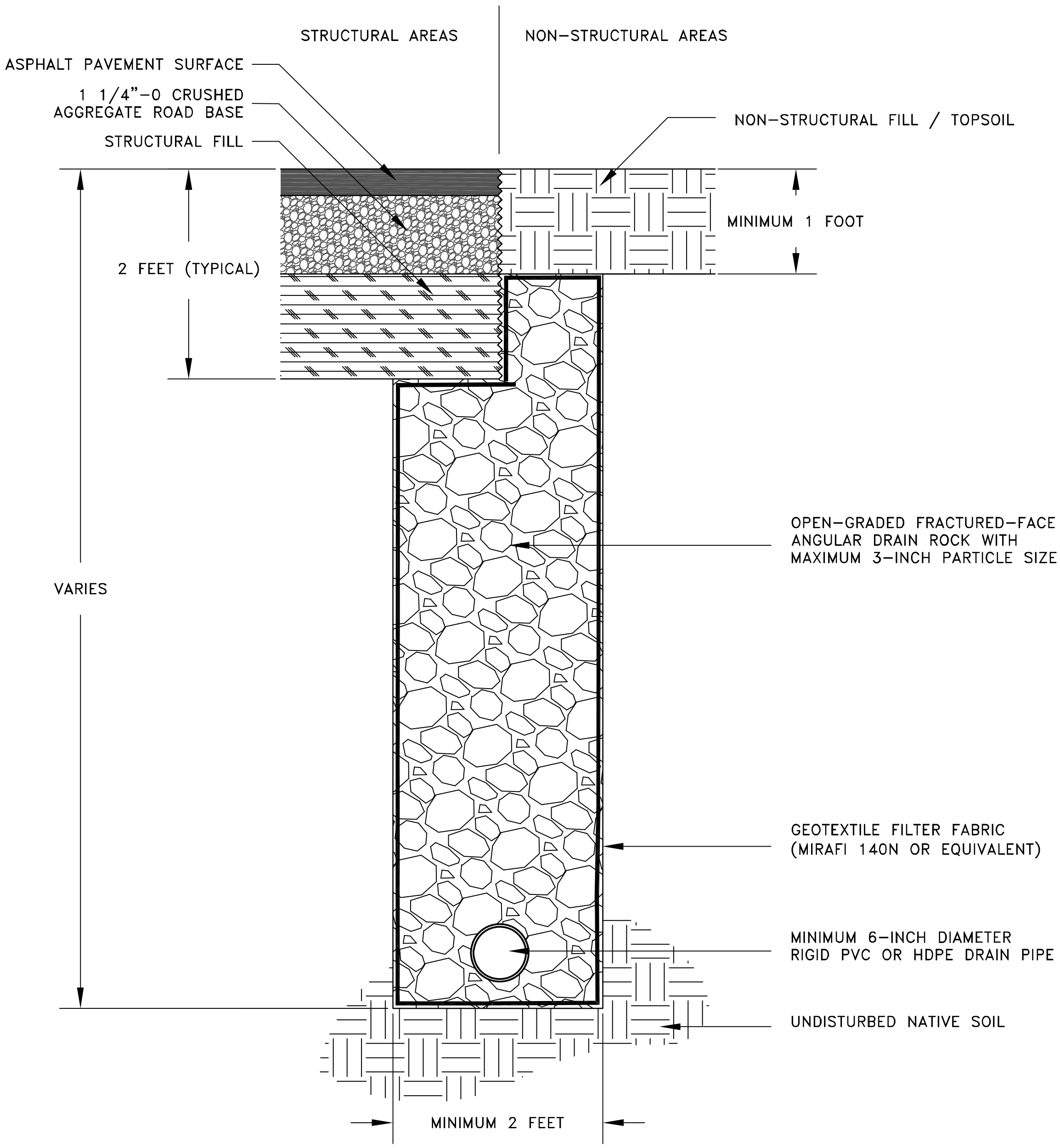
Client: CLARK	Rev	By	Date	
Job No: 18345				
CAD File: FIGURE 5				
Scale: NONE				

TYPICAL PERIMETER FOOTING DRAIN DETAIL

CLARK COLLEGE
NORTH COUNTY CAMPUS
RIDGEFIELD, WASHINGTON

FIGURE 5

TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL



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

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Design:	Drawn: HDG			TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL
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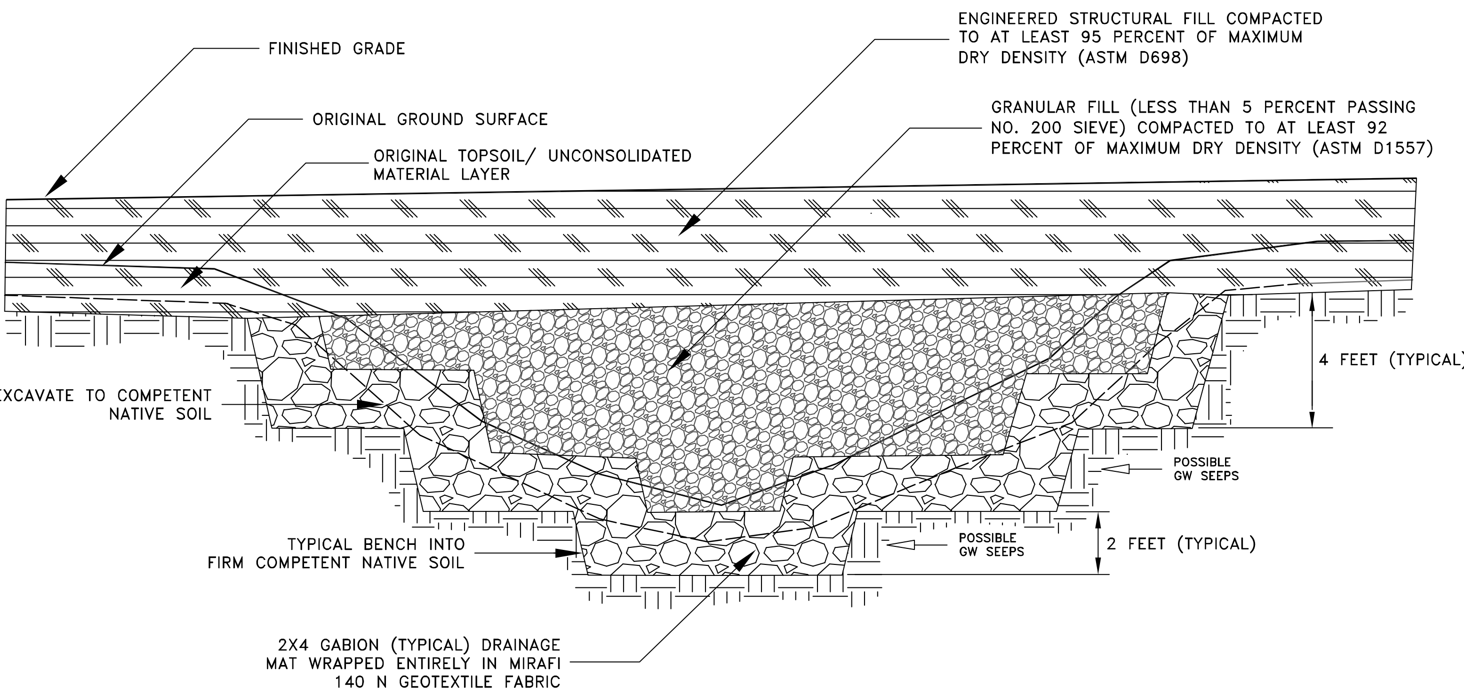
Job No: 18345				
CAD File: FIGURE 6				
Scale: NONE				

FIGURE 6

TYPICAL DRAINAGE MAT CROSS-SECTION



NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL DRAINAGE MAT SECTION AND MAY NOT BE SITE-SPECIFIC.
4. DEPTH, LOCATION, EXTENT, AND THICKNESS OF GABION MAT AND GRANULAR FILL LAYER SHOULD BE DETERMINED IN THE FIELD BY COLUMBIA WEST.
5. DRAIN PIPE MAY BE NEEDED AT LOWEST GRADIENT POINT OF DRAINAGE MAT TO CONTROL AND DIRECT FLOW.

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Design:	Drawn: HDG			TYPICAL DRAINAGE MAT SECTION	FIGURE 7
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Job No: 18345					
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Scale: NONE					

APPENDIX A
LABORATORY TEST RESULTS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1201
		REPORT DATE: 12/10/18	FIELD ID: TP4.1
		DATE SAMPLED: 11/27/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-04 depth = 2 feet	CL, Lean Clay with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-6(13)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

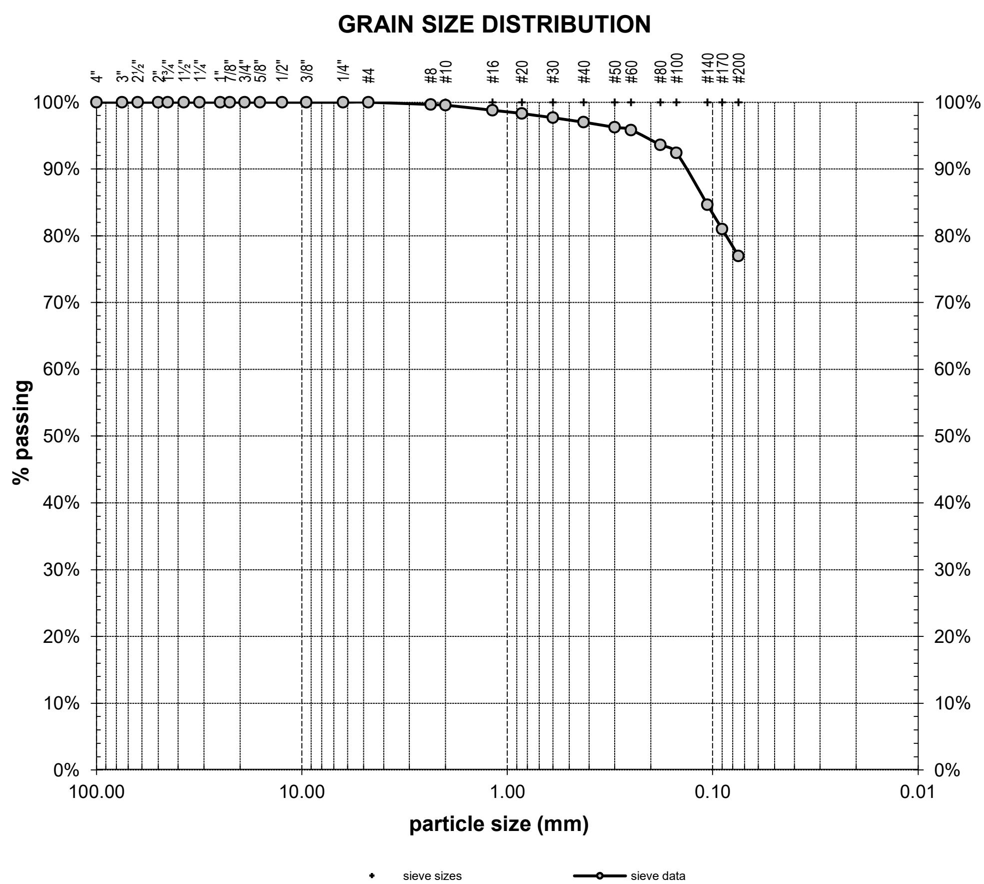
ADDITIONAL DATA

initial dry mass (g) =	200.60		
as-received moisture content =	30.0%	coefficient of curvature, C_C =	n/a
liquid limit =	40	coefficient of uniformity, C_U =	n/a
plastic limit =	23	effective size, $D_{(10)}$ =	n/a
plasticity index =	17	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 23.1%
% silt and clay = 76.9%

	SIEVE SIZE		PERCENT PASSING SIEVE		SPECS	
	US	mm	act.	interp.	max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		99%		
	#20	0.850	98%			
	#30	0.600		98%		
	#40	0.425	97%			
	#50	0.300		96%		
	#60	0.250	96%			
	#80	0.180		94%		
	#100	0.150	92%			
	#140	0.106		85%		
	#170	0.090		81%		
	#200	0.075	77%			



DATE TESTED	TESTED BY
12/04/18	KMS

COLUMBIA WEST ENGINEERING, INC. authorized signature

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ATTERBERG LIMITS REPORT

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Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1201
		REPORT DATE: 12/10/18	FIELD ID: TP4.1
		DATE SAMPLED: 11/27/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-04 depth = 2 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

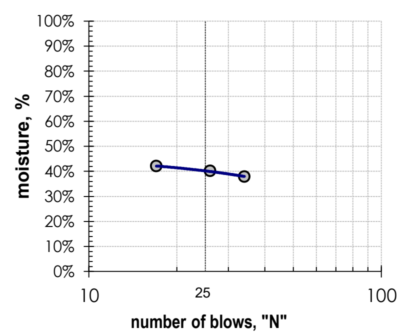
ATTERBERG LIMITS

liquid limit = 40
plastic limit = 23
plasticity index = 17

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.42	33.92	33.92	
dry soil + pan weight, g =	29.26	30.16	30.07	
pan weight, g =	20.91	20.80	20.91	
N (blows) =	34	26	17	
moisture, % =	37.8 %	40.2 %	42.0 %	

LIQUID LIMIT



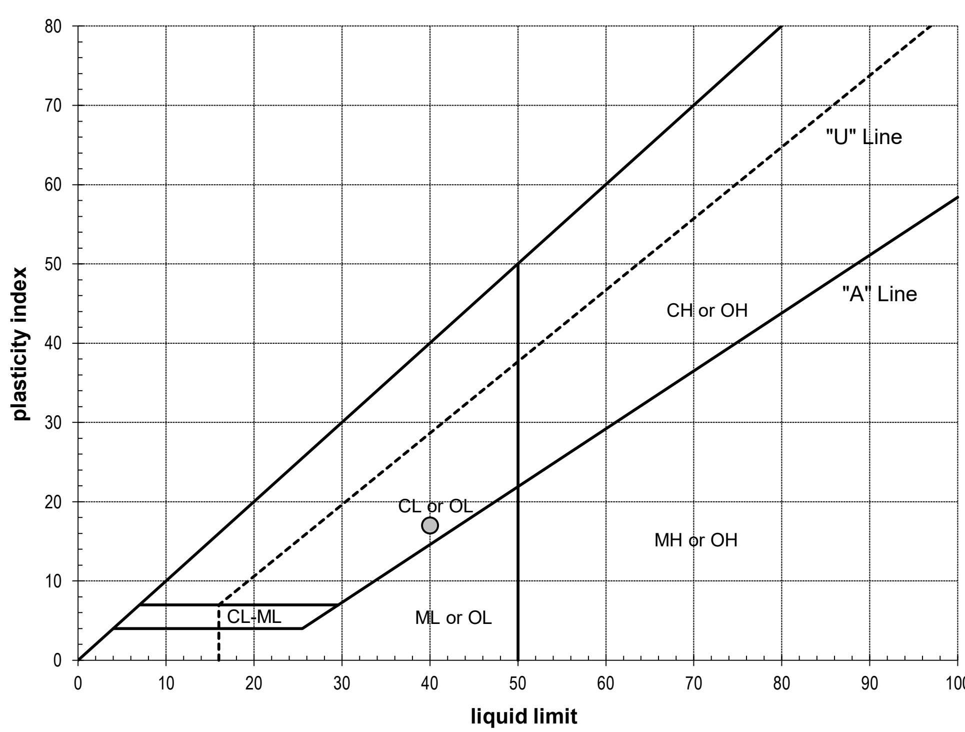
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.93	27.96		
dry soil + pan weight, g =	26.57	26.55		
pan weight, g =	20.74	20.59		
moisture, % =	23.3 %	23.7 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 23.1%
% silt and clay = 76.9%
% silt = n/a
% clay = n/a
moisture content = 30.0%

DATE TESTED	TESTED BY
12/05/18	RTT

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1202
		REPORT DATE: 12/10/18	FIELD ID: TP4.2
		DATE SAMPLED: 11/27/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-04 depth = 6 feet	CL, Lean Clay with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-6(10)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

ADDITIONAL DATA

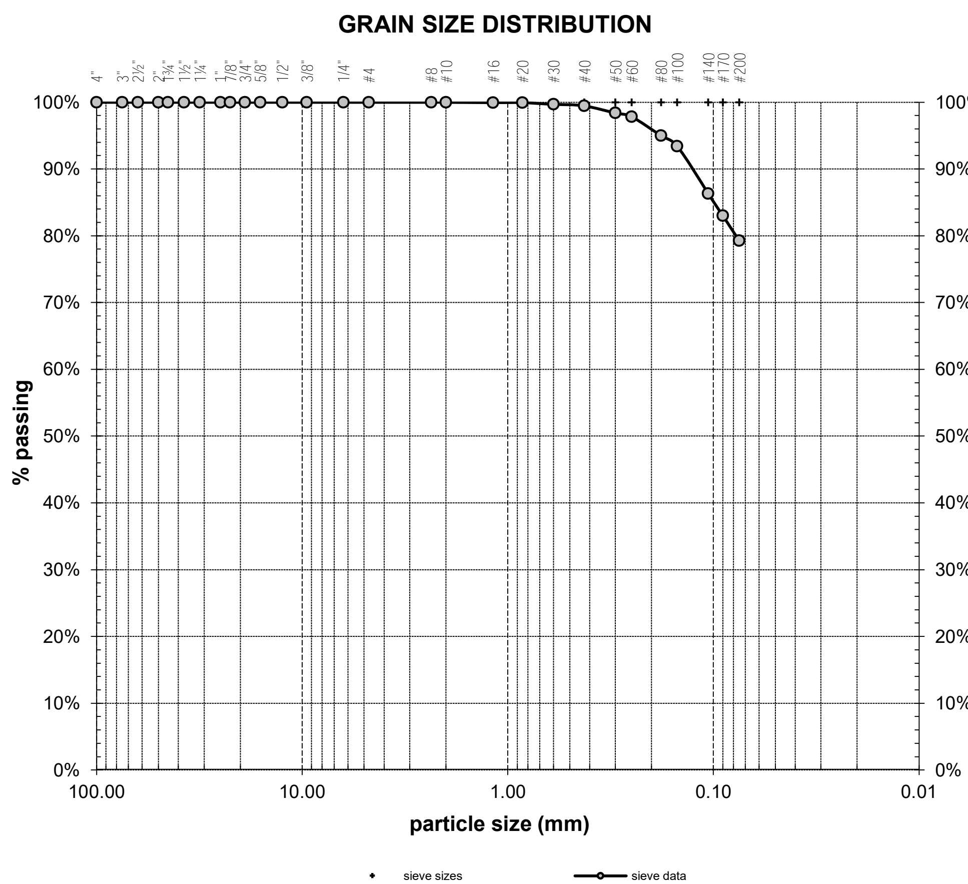
initial dry mass (g) =	187.02		
as-received moisture content =	31.2%	coefficient of curvature, C_C =	n/a
liquid limit =	35	coefficient of uniformity, C_U =	n/a
plastic limit =	22	effective size, $D_{(10)}$ =	n/a
plasticity index =	13	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 20.7%
% silt and clay = 79.3%

	SIEVE SIZE		PERCENT PASSING SIEVE		SPECS	
	US	mm	act.	interp.	max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		100%		
	#20	0.850	100%			
	#30	0.600		100%		
	#40	0.425	99%			
	#50	0.300		98%		
	#60	0.250	98%			
	#80	0.180		95%		
	#100	0.150	93%			
	#140	0.106		86%		
	#170	0.090		83%		
	#200	0.075	79%			

DATE TESTED	TESTED BY
12/04/18	KMS



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1202
		REPORT DATE: 12/10/18	FIELD ID: TP4.2
		DATE SAMPLED: 11/27/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-04 depth = 6 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

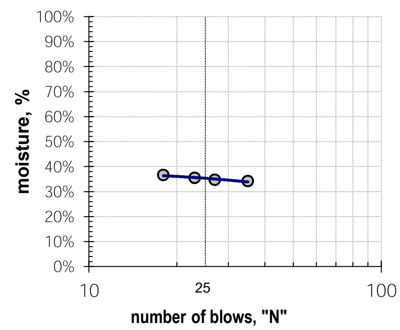
ATTERBERG LIMITS

liquid limit =	35
plastic limit =	22
plasticity index =	13

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.97	32.94	34.76	34.35
dry soil + pan weight, g =	29.14	29.81	31.08	30.70
pan weight, g =	20.84	20.77	20.71	20.74
N (blows) =	35	27	23	18
moisture, % =	34.1 %	34.6 %	35.5 %	36.7 %

LIQUID LIMIT



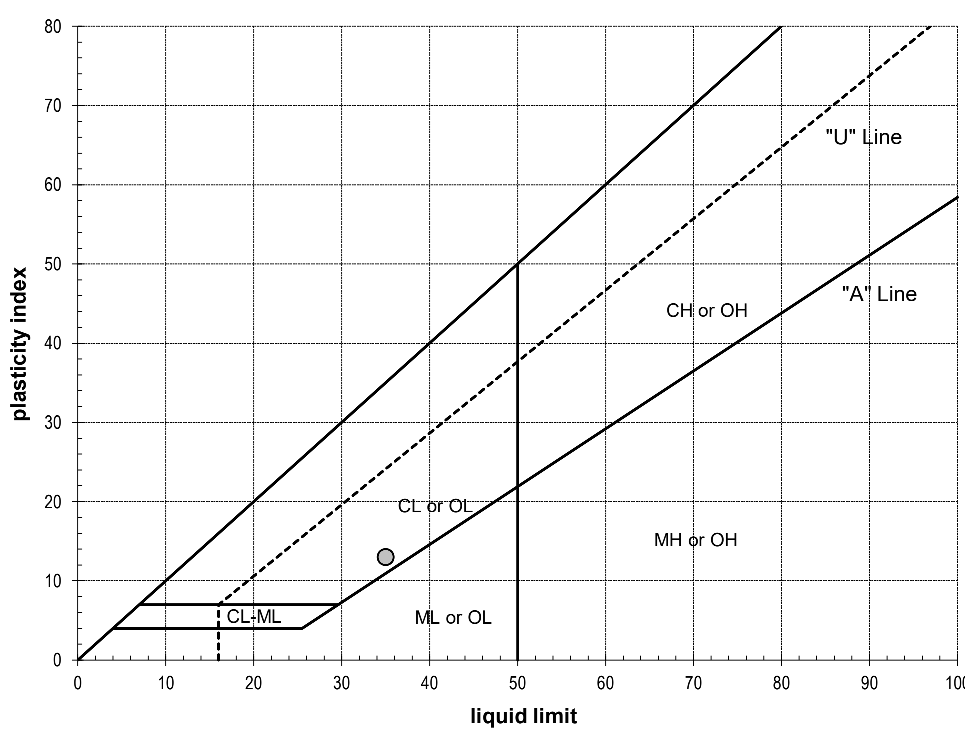
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.09	28.06		
dry soil + pan weight, g =	26.78	26.74		
pan weight, g =	20.80	20.60		
moisture, % =	21.9 %	21.5 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	20.7%
% silt and clay =	79.3%
% silt =	n/a
% clay =	n/a
moisture content =	31.2%

DATE TESTED	TESTED BY
12/05/18	RTT

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1228
		REPORT DATE: 12/14/18	FIELD ID: SB1.4
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Soil Boring SB-01 depth = 10.5 feet	CL, Lean Clay

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(29)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

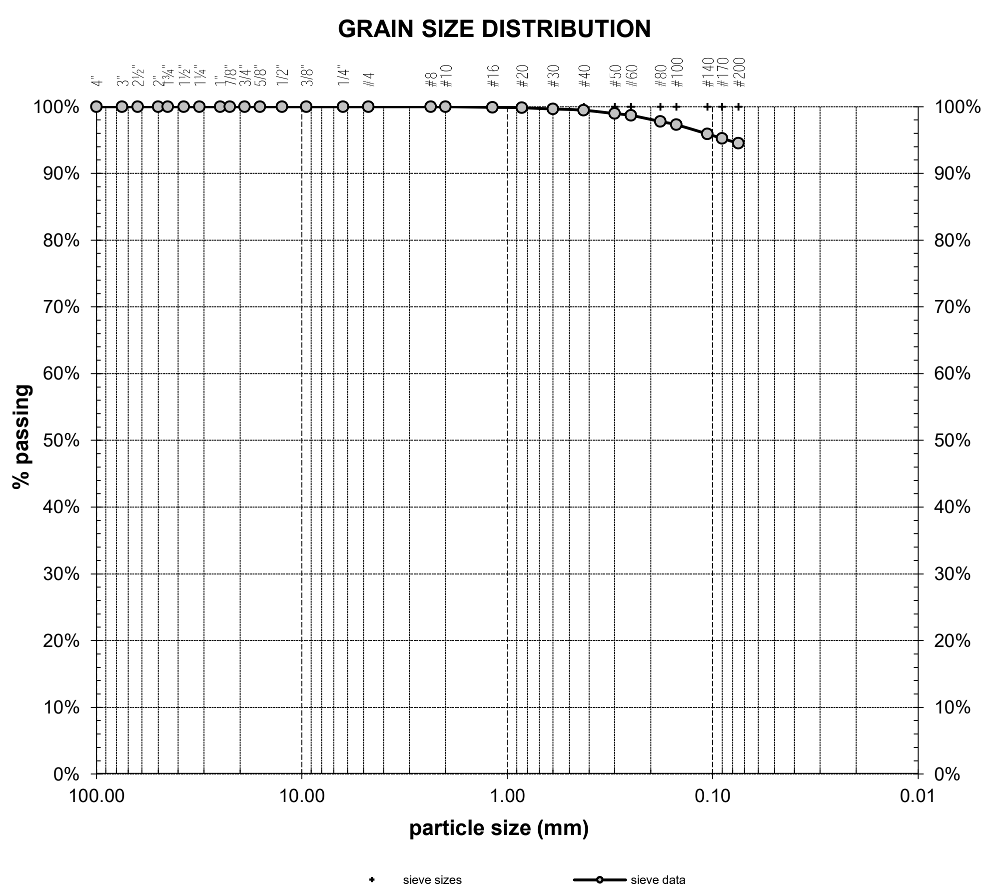
ADDITIONAL DATA

initial dry mass (g) =	156.63		
as-received moisture content =	33.3%	coefficient of curvature, C_C =	n/a
liquid limit =	47	coefficient of uniformity, C_U =	n/a
plastic limit =	18	effective size, $D_{(10)}$ =	n/a
plasticity index =	29	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 5.5%
% silt and clay = 94.5%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		100%		
	#20	0.850	100%			
	#30	0.600		100%		
	#40	0.425	99%			
	#50	0.300		99%		
	#60	0.250	99%			
	#80	0.180		98%		
	#100	0.150	97%			
	#140	0.106		96%		
	#170	0.090		95%		
	#200	0.075	94%			



DATE TESTED	TESTED BY
12/08/18	RTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1228
		REPORT DATE: 12/14/18	FIELD ID: SB1.4
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Soil Boring SB-01 depth = 10.5 feet	CL, Lean Clay

LABORATORY TEST DATA

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

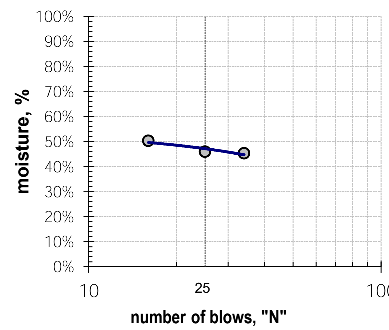
ATTERBERG LIMITS

liquid limit = 47
plastic limit = 18
plasticity index = 29

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	29.03	29.59	29.86	
dry soil + pan weight, g =	26.40	26.86	26.83	
pan weight, g =	20.60	20.91	20.80	
N (blows) =	34	25	16	
moisture, % =	45.3 %	45.9 %	50.3 %	

LIQUID LIMIT



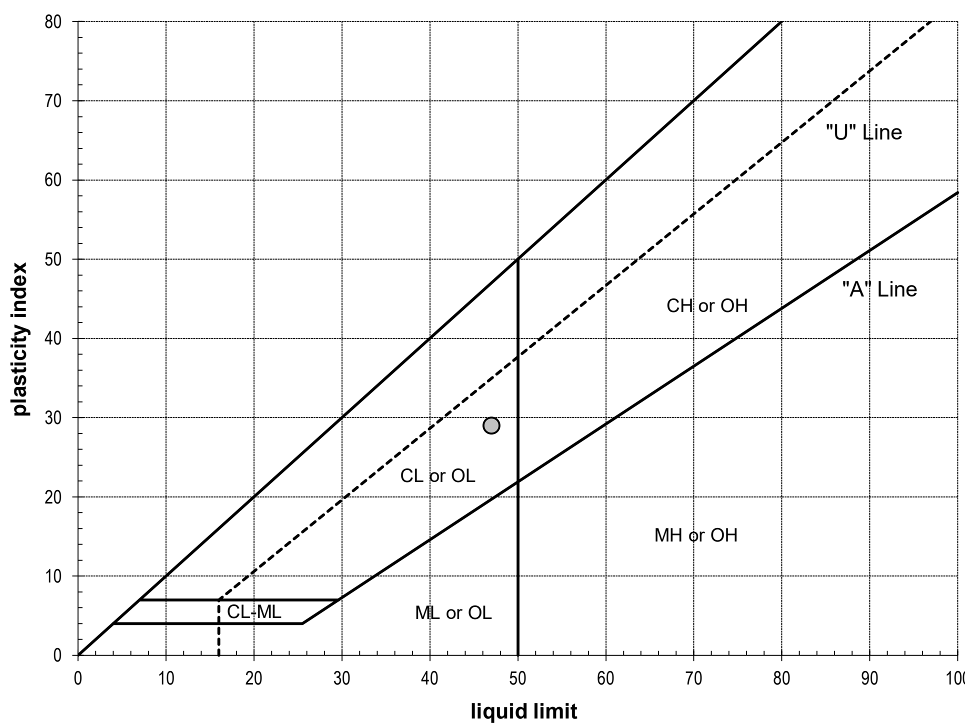
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	29.91	28.45		
dry soil + pan weight, g =	28.53	27.30		
pan weight, g =	20.84	20.65		
moisture, % =	18.0 %	17.3 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 5.5%
% silt and clay = 94.5%
% silt = n/a
% clay = n/a
moisture content = 33.3%

DATE TESTED	TESTED BY
12/07/18	KMS

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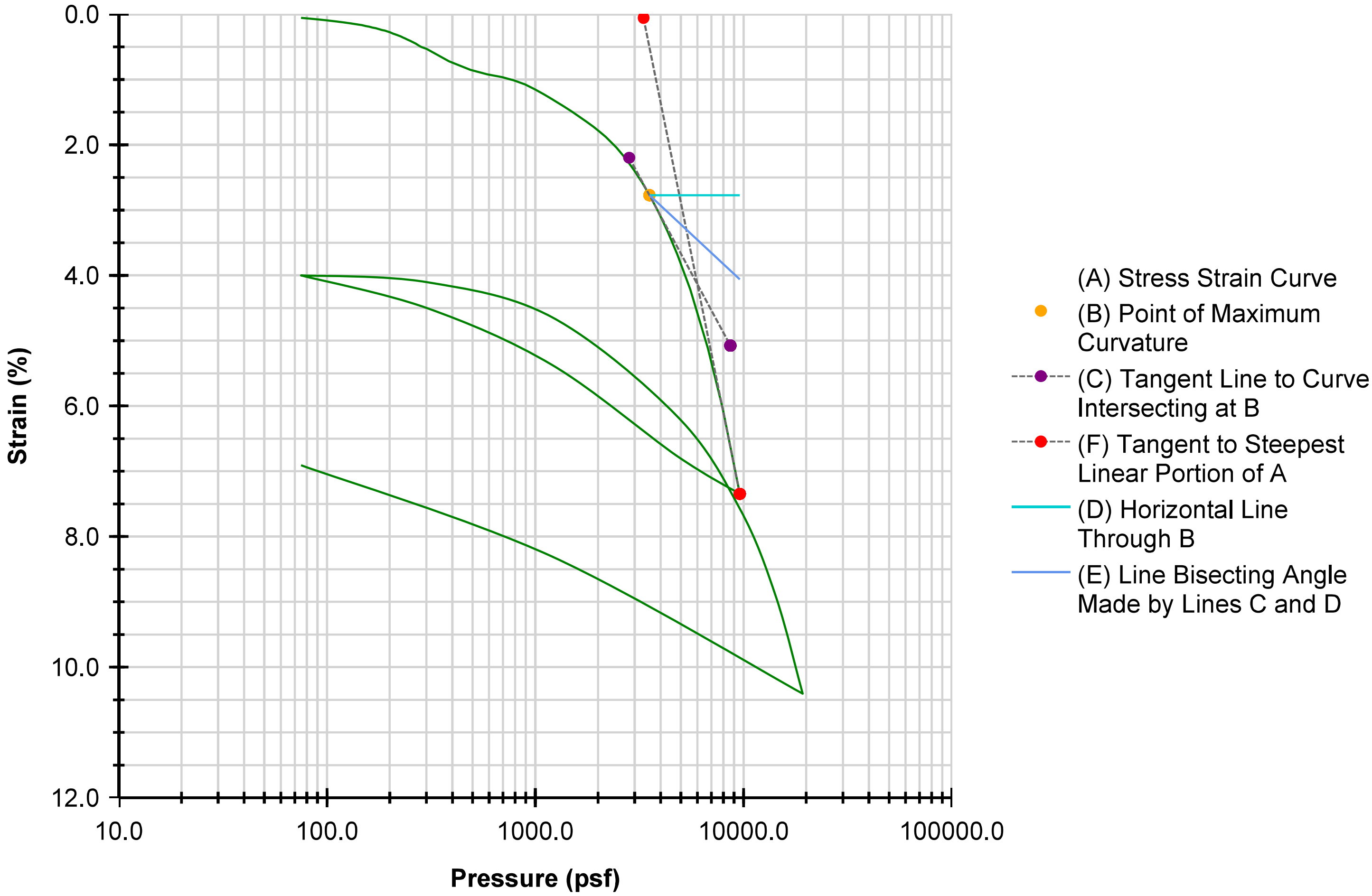
Percent Strain [Log]

ASTM D2435



Preconsolidation Stress (psf)	5315.87	Cc	0.169	Cr	0.020

	BEFORE	AFTER			
			Liquid Limits	47	Test Date 12/11/2018
Moisture (%)	31.0	29.4	Plastic Limits	18	
Dry Density (pcf)	89.1	95.0			
Saturation (%)	93.0	101.6			
Void Ratio	0.91	0.79	Specific Gravity	2.72	ASSUMED

Sample Description	CL, Gray and Brown Heavily Mottled Lean CLAY			
Project Number	18345	Depth (ft)	11.25	Remarks P200=94.5%
Sample Number	S18-1228	Boring Number	SB1.4	
Project	Clark College North County Campus			
Client	Clark College			
Location	Vancouver, Washington			

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1229
		REPORT DATE: 12/14/18	FIELD ID: SB1.7
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Soil Boring SB-01 depth = 20 feet	CL, Lean Clay

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(28)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

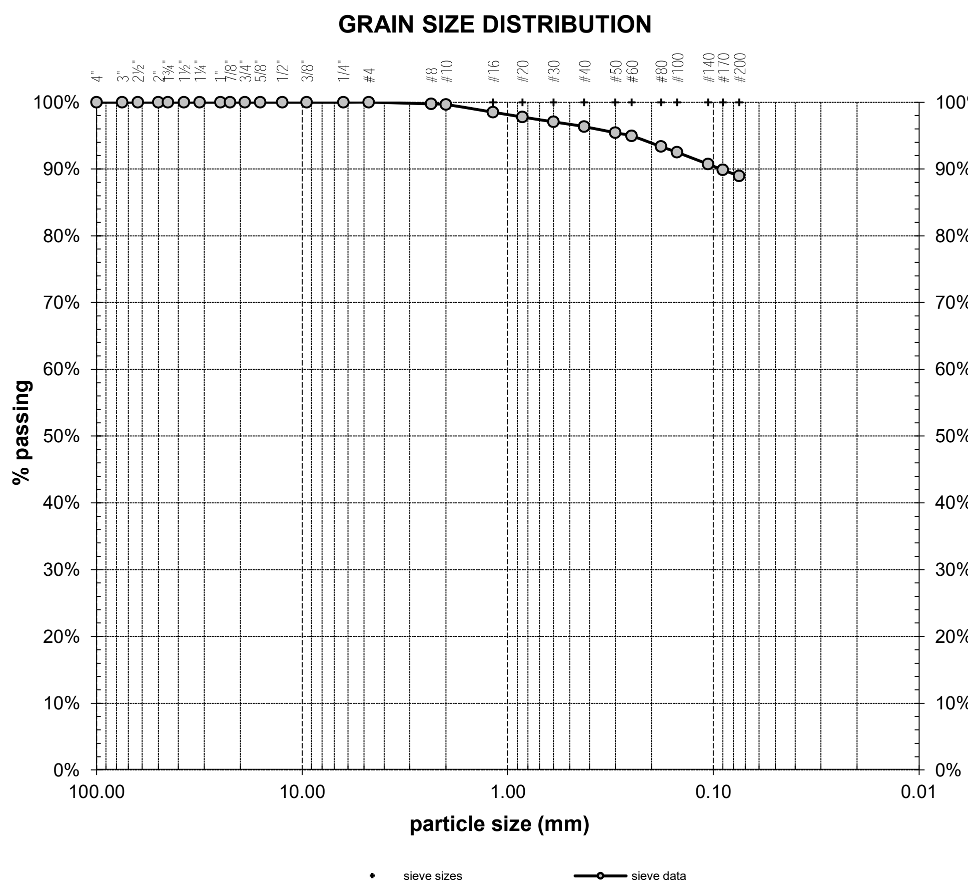
ADDITIONAL DATA

initial dry mass (g) =	162.55		
as-received moisture content =	27.3%	coefficient of curvature, C_C =	n/a
liquid limit =	48	coefficient of uniformity, C_U =	n/a
plastic limit =	18	effective size, $D_{(10)}$ =	n/a
plasticity index =	30	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 11.1%
% silt and clay = 88.9%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		98%		
	#20	0.850	98%			
	#30	0.600		97%		
	#40	0.425	96%			
	#50	0.300		95%		
	#60	0.250	95%			
	#80	0.180		93%		
	#100	0.150	92%			
	#140	0.106		91%		
	#170	0.090		90%		
	#200	0.075	89%			



DATE TESTED	TESTED BY
12/07/18	RTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1229
		REPORT DATE: 12/14/18	FIELD ID: SB1.7
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Soil Boring SB-01 depth = 20 feet	CL, Lean Clay

LABORATORY TEST DATA

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

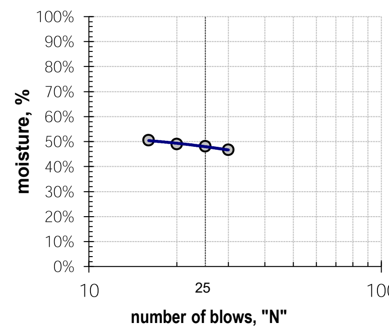
ATTERBERG LIMITS

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

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	30.38	32.23	33.15	31.26
dry soil + pan weight, g =	27.32	28.52	29.08	27.77
pan weight, g =	20.76	20.80	20.78	20.87
N (blows) =	30	25	20	16
moisture, % =	46.7 %	48.1 %	49.0 %	50.6 %

LIQUID LIMIT



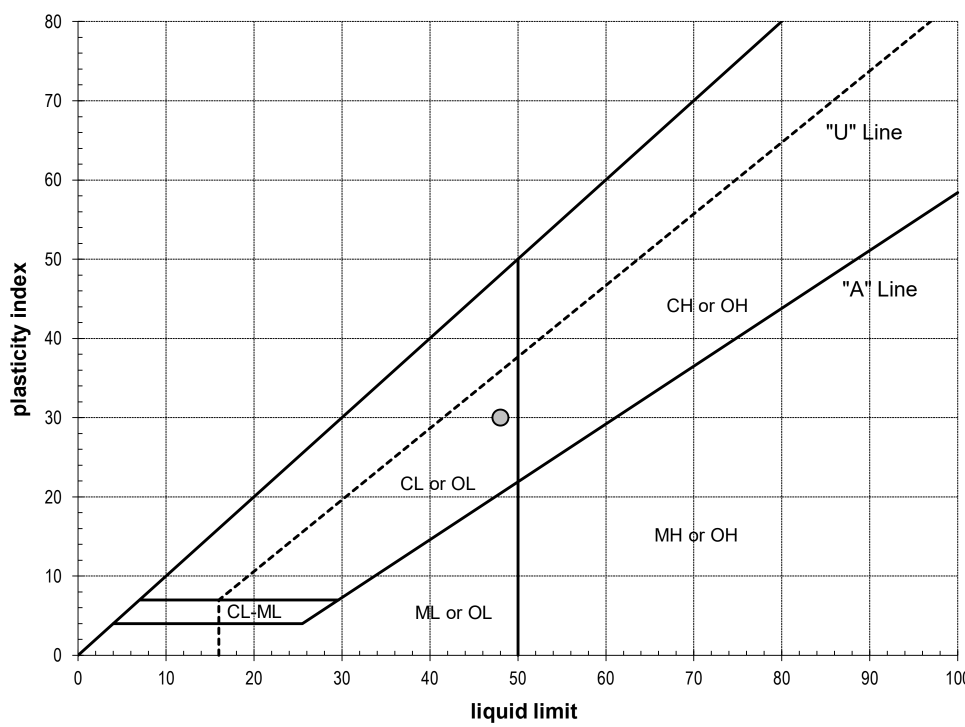
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.42	28.33		
dry soil + pan weight, g =	26.42	27.19		
pan weight, g =	20.73	20.86		
moisture, % =	17.6 %	18.0 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	11.1%
% silt and clay =	88.9%
% silt =	n/a
% clay =	n/a
moisture content =	27.3%

DATE TESTED	TESTED BY
12/07/18	RTT

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1230
		REPORT DATE: 12/14/18	FIELD ID: SB1.12
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty, Clayey SAND	Soil Boring SB-01 depth = 40 feet	SC-SM, Silty, Clayey Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-4(0)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

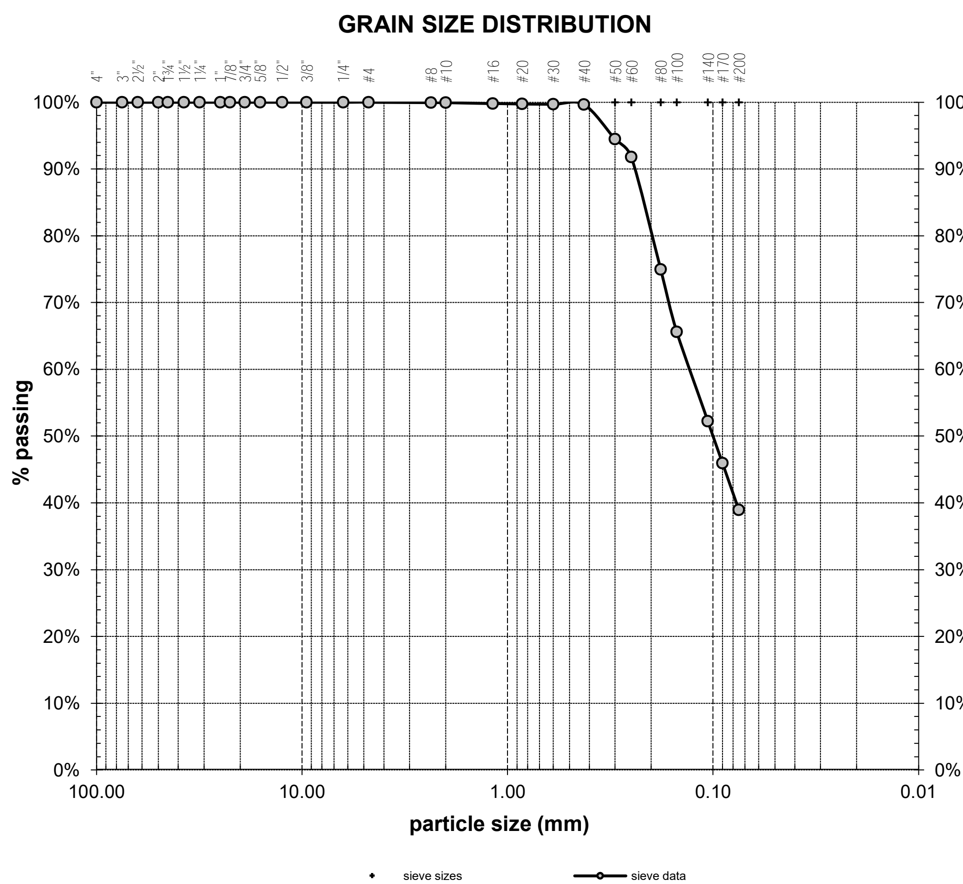
ADDITIONAL DATA

initial dry mass (g) =	132.90		
as-received moisture content =	19.8%	coefficient of curvature, C_C =	n/a
liquid limit =	24	coefficient of uniformity, C_U =	n/a
plastic limit =	18	effective size, $D_{(10)}$ =	n/a
plasticity index =	6	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	0.130 mm

SIEVE DATA

% gravel = 0.0%
% sand = 61.1%
% silt and clay = 38.9%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		100%		
	#20	0.850	100%			
	#30	0.600		100%		
	#40	0.425	100%			
	#50	0.300		94%		
	#60	0.250	92%			
	#80	0.180		75%		
	#100	0.150	66%			
	#140	0.106		52%		
	#170	0.090		46%		
	#200	0.075	39%			



DATE TESTED	TESTED BY
12/07/18	RTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1230
		REPORT DATE: 12/14/18	FIELD ID: SB1.12
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty, Clayey SAND	Soil Boring SB-01 depth = 40 feet	SC-SM, Silty, Clayey Sand

LABORATORY TEST DATA

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

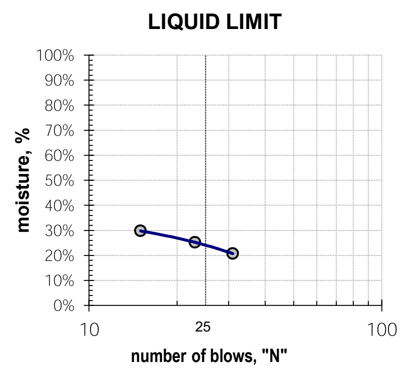
ATTERBERG LIMITS

liquid limit = 24
plastic limit = 18
plasticity index = 6

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.73	35.48	32.93	
dry soil + pan weight, g =	32.35	32.51	30.17	
pan weight, g =	20.87	20.71	20.90	
N (blows) =	31	23	15	
moisture, % =	20.7 %	25.2 %	29.8 %	

LIQUID LIMIT



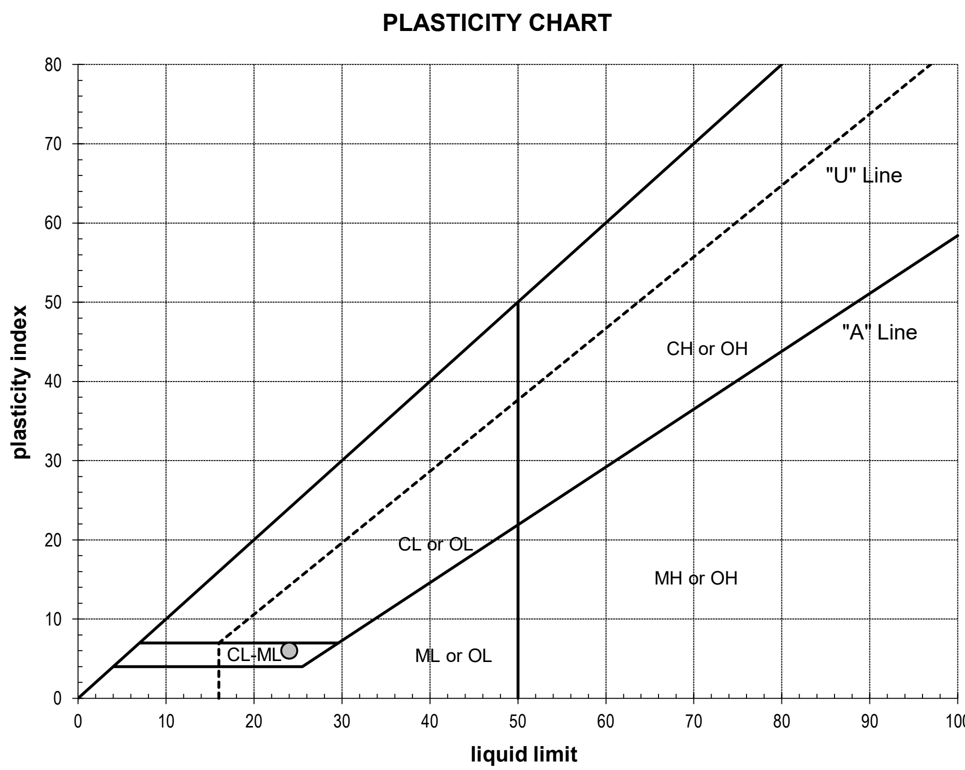
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.89	28.06		
dry soil + pan weight, g =	26.80	26.95		
pan weight, g =	20.74	20.80		
moisture, % =	18.0 %	18.1 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	61.1%
% silt and clay =	38.9%
% silt =	n/a
% clay =	n/a
moisture content =	19.8%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1231
		REPORT DATE: 12/14/18	FIELD ID: SB2.4
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Soil Boring SB-02 depth = 10 feet	CL, Lean Clay

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(27)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

ADDITIONAL DATA

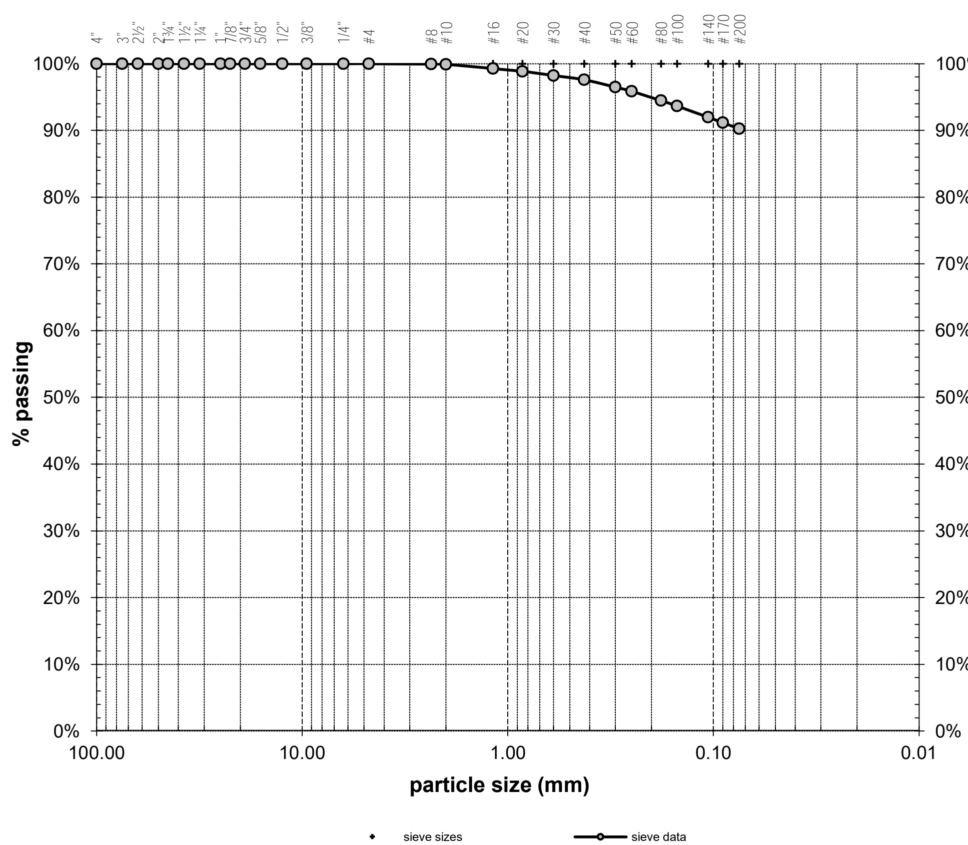
initial dry mass (g) =	121.72		
as-received moisture content =	33.2%	coefficient of curvature, C_C =	n/a
liquid limit =	47	coefficient of uniformity, C_U =	n/a
plastic limit =	19	effective size, $D_{(10)}$ =	n/a
plasticity index =	28	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 9.7%
% silt and clay = 90.3%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		99%		
	#20	0.850	99%			
	#30	0.600		98%		
	#40	0.425	98%			
	#50	0.300		96%		
	#60	0.250	96%			
	#80	0.180		94%		
	#100	0.150	94%			
	#140	0.106		92%		
	#170	0.090		91%		
	#200	0.075	90%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1231
		REPORT DATE: 12/14/18	FIELD ID: SB2.4
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Soil Boring SB-02 depth = 10 feet	CL, Lean Clay

LABORATORY TEST DATA

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

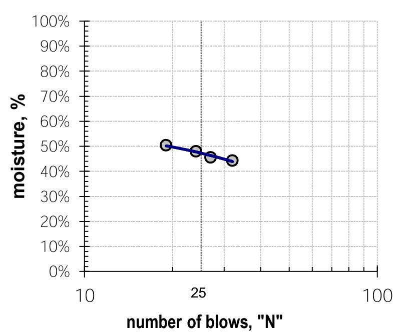
ATTERBERG LIMITS

liquid limit =	47
plastic limit =	19
plasticity index =	28

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	33.69	34.87	32.80	31.33
dry soil + pan weight, g =	29.71	30.47	28.93	27.79
pan weight, g =	20.73	20.80	20.87	20.76
N (blows) =	32	27	24	19
moisture, % =	44.3 %	45.5 %	48.0 %	50.4 %

LIQUID LIMIT



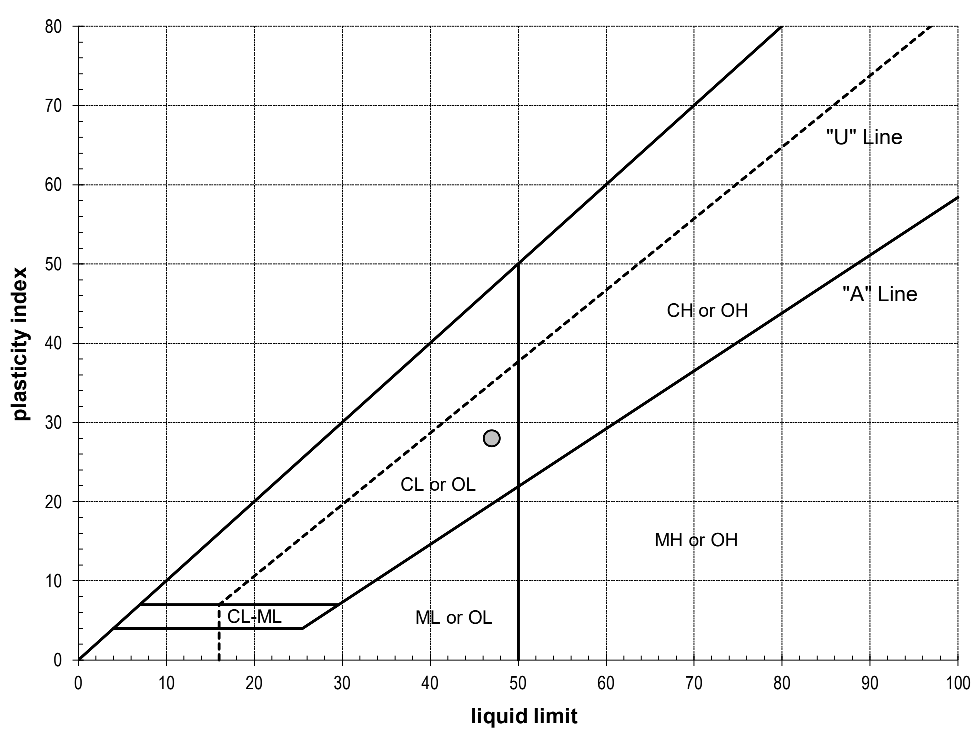
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.18	27.61		
dry soil + pan weight, g =	26.11	26.50		
pan weight, g =	20.51	20.78		
moisture, % =	19.1 %	19.4 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	9.7%
% silt and clay =	90.3%
% silt =	n/a
% clay =	n/a
moisture content =	33.2%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1232
		REPORT DATE: 12/14/18	FIELD ID: SB2.6
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY	Soil Boring SB-02 depth = 20 feet	CH, Fat Clay

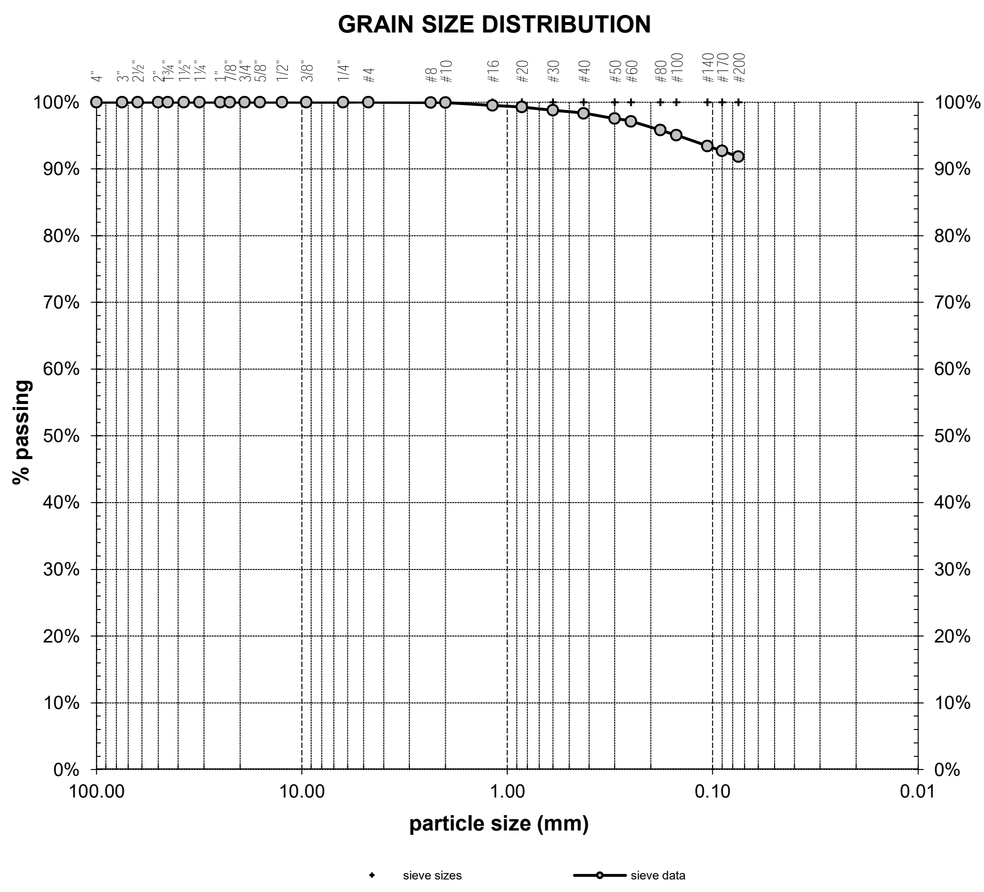
SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(33)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

ADDITIONAL DATA

initial dry mass (g) =	159.35		
as-received moisture content =	29.7%	coefficient of curvature, C_C =	n/a
liquid limit =	55	coefficient of uniformity, C_U =	n/a
plastic limit =	22	effective size, $D_{(10)}$ =	n/a
plasticity index =	33	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a



SIEVE DATA

% gravel = 0.0%
% sand = 8.2%
% silt and clay = 91.8%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		100%		
	#20	0.850	99%			
	#30	0.600		99%		
	#40	0.425	98%			
	#50	0.300		98%		
	#60	0.250	97%			
	#80	0.180		96%		
	#100	0.150	95%			
	#140	0.106		93%		
	#170	0.090		93%		
	#200	0.075	92%			

DATE TESTED	TESTED BY
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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1232
		REPORT DATE: 12/14/18	FIELD ID: SB2.6
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY	Soil Boring SB-02 depth = 20 feet	CH, Fat Clay

LABORATORY TEST DATA

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

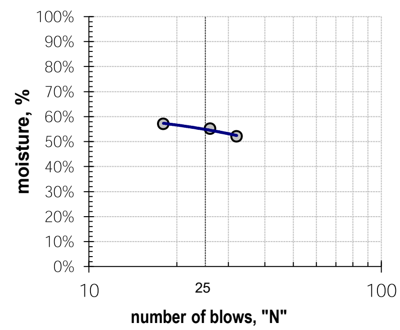
ATTERBERG LIMITS

liquid limit = 55
plastic limit = 22
plasticity index = 33

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	29.56	30.17	29.14	
dry soil + pan weight, g =	26.49	26.88	26.11	
pan weight, g =	20.60	20.91	20.80	
N (blows) =	32	26	18	
moisture, % =	52.1 %	55.1 %	57.1 %	

LIQUID LIMIT



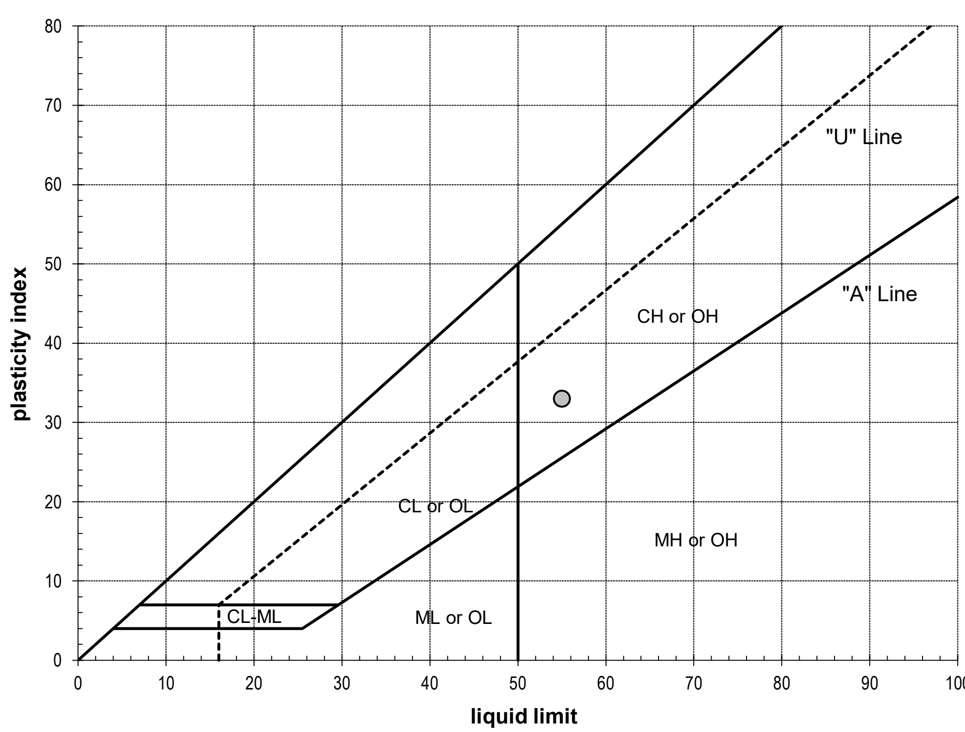
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.44	27.56		
dry soil + pan weight, g =	26.23	26.36		
pan weight, g =	20.74	20.86		
moisture, % =	22.0 %	21.8 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 8.2%
% silt and clay = 91.8%
% silt = n/a
% clay = n/a
moisture content = 29.7%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1233
		REPORT DATE: 12/14/18	FIELD ID: SB2.8
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Lean CLAY	Soil Boring SB-02 depth = 30 feet	CL, Sandy Lean Clay

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(11)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

ADDITIONAL DATA

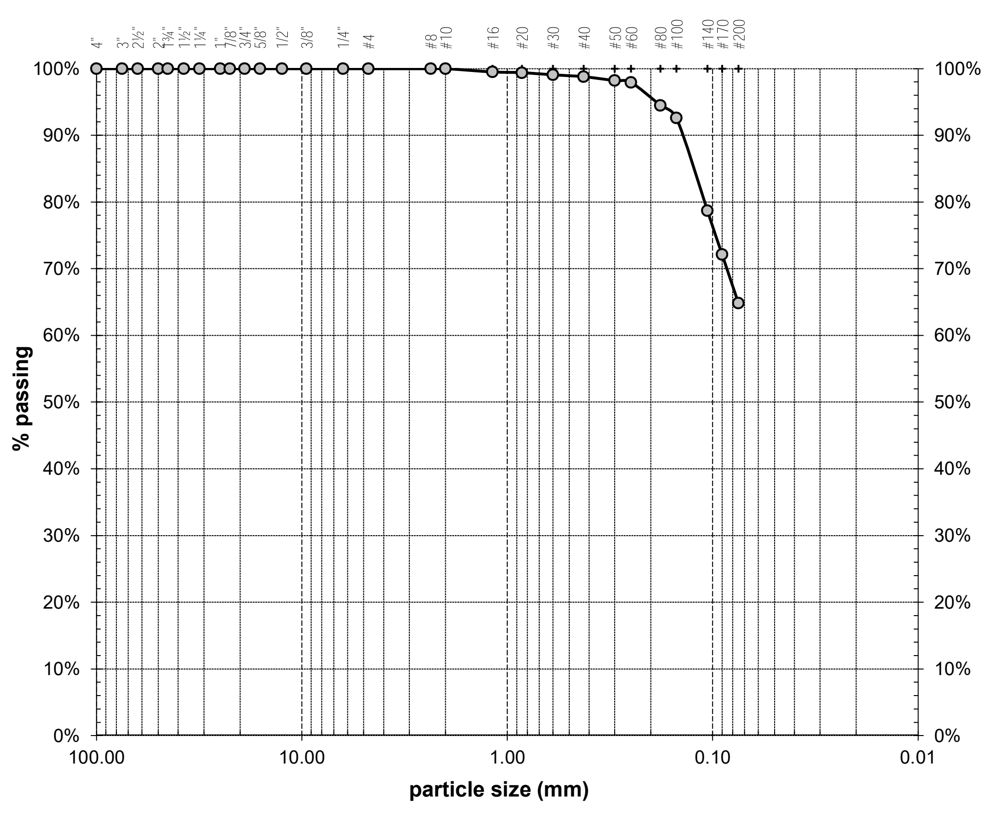
initial dry mass (g) =	127.47		
as-received moisture content =	30.2%	coefficient of curvature, C_C =	n/a
liquid limit =	42	coefficient of uniformity, C_U =	n/a
plastic limit =	22	effective size, $D_{(10)}$ =	n/a
plasticity index =	20	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 35.2%
% silt and clay = 64.8%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		99%		
	#20	0.850	99%			
	#30	0.600		99%		
	#40	0.425	99%			
	#50	0.300		98%		
	#60	0.250	98%			
	#80	0.180		94%		
	#100	0.150	93%			
	#140	0.106		79%		
	#170	0.090		72%		
	#200	0.075	65%			

GRAIN SIZE DISTRIBUTION



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Clark College North County Campus 264 N 65th Avenue Ridgefield, Washington	Mr. Jim Watkins Clark College 1933 Fort Vancouver Way Vancouver, Washington 98663	18345	S18-1233
		REPORT DATE: 12/14/18	FIELD ID: SB2.8
		DATE SAMPLED: 11/30/18	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Lean CLAY	Soil Boring SB-02 depth = 30 feet	CL, Sandy Lean Clay

LABORATORY TEST DATA

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

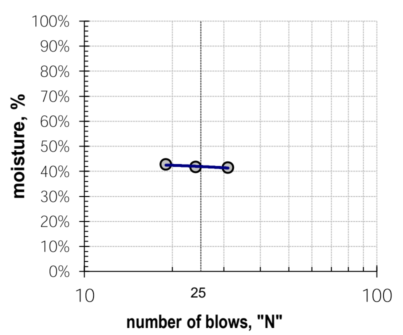
ATTERBERG LIMITS

liquid limit =	42
plastic limit =	22
plasticity index =	20

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	30.54	32.38	31.74	
dry soil + pan weight, g =	27.67	28.97	28.46	
pan weight, g =	20.75	20.79	20.78	
N (blows) =	31	24	19	
moisture, % =	41.5 %	41.7 %	42.7 %	

LIQUID LIMIT



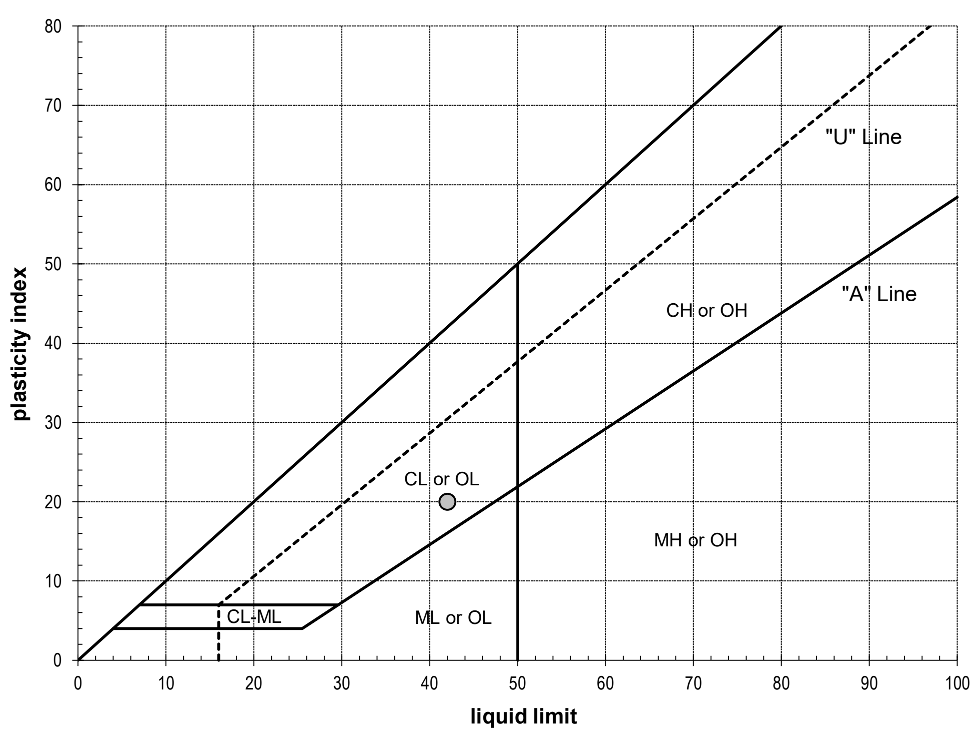
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.55	27.48		
dry soil + pan weight, g =	26.37	26.26		
pan weight, g =	20.85	20.65		
moisture, % =	21.4 %	21.8 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	35.2%
% silt and clay =	64.8%
% silt =	n/a
% clay =	n/a
moisture content =	30.2%

DATE TESTED	TESTED BY
12/10/18	KMS

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

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SOIL BORING LOG

PROJECT NAME	CLIENT		PROJECT NO.	BORING NO.
Clark College North County Campus	Clark College		18345	SB-1
PROJECT LOCATION	DRILLING CONTRACTOR	DRILL RIG	ENGINEER	PAGE NO.
Ridgefield, Washington	Western States	CME 55 Track Rig	HDG	1 of 2
BORING LOCATION	DRILLING METHOD	SAMPLING METHOD	START DATE	START TIME
See Figure 2	Mud-rotary	SPT/Shelby	11/30/18	0930
REMARKS	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	FINISH DATE	FINISH TIME
None	282 ft amsl	Not encountered	11/30/18	1225

Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected) 0 25 50 75	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Wet Density (PCF)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0							Approximately 16 to 18 inches of topsoil and grass.					
2	280			CL			Heavily to lightly mottled brown to gray lean CLAY, moist to very moist, medium stiff to very stiff. [Soil Type 1]					
		SPT SB-1.1	13									
4												
		SPT SB-1.2	10									
6	276											
8		SPT SB-1.3	8									
10	272	SHELBY										
		SB-1.4							33.3	94.5	47	29
12												
		SPT SB-1.5	7									
14	268											
		SPT SB-1.6	13									
16												
18	264											
20												
		SPT SB-1.7	20						27.3	88.9	48	30
22	260						Black nodules observed in soil samples.					
24												
		SPT SB-1.8	21									
26	256											
28												
30	252											

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SOIL BORING LOG

PROJECT NAME	CLIENT		PROJECT NO.	BORING NO.
Clark College North County Campus	Clark College		18345	SB-1
PROJECT LOCATION	DRILLING CONTRACTOR	DRILL RIG	ENGINEER	PAGE NO.
Ridgefield, Washington	Western States	CME 55 Track Rig	HDG	2 of 2
BORING LOCATION	DRILLING METHOD	SAMPLING METHOD	START DATE	START TIME
See Figure 2	Mud-rotary	SPT/Shelby	11/30/18	0930
REMARKS	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	FINISH DATE	FINISH TIME
None	282 ft amsl	Not encountered	11/30/18	1225

Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Wet Density (PCF)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
30	252	SPT SB-1.9	13				Heavily to lightly mottled brown to gray lean CLAY, moist to very moist, medium stiff to very stiff. [Soil Type 1]					
32												
34	248											
36		SHELBY SB-1.10										
38	244	SPT SB-1.11	13									
40		SPT SB-1.12	18	SC-SM			Heavily mottled red to orange-brown silty clayey SAND, moist, medium dense to dense. [Soil Type 2]		19.8	38.9	24	6
42	240											
44												
46	236	SPT SB-1.13	31									
48												
50	232	SPT SB-1.14	26									
52							Soil boring terminated at 51.5 ft bgs. Groundwater not encountered.					
54	228											
56												
58	224											
60												

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SOIL BORING LOG

PROJECT NAME	CLIENT		PROJECT NO.	BORING NO.
Clark College North County Campus	Clark College		18345	SB-2
PROJECT LOCATION	DRILLING CONTRACTOR	DRILL RIG	ENGINEER	PAGE NO.
Ridgefield, Washington	Western States	CME 55 Track Rig	HDG	1 of 2
BORING LOCATION	DRILLING METHOD	SAMPLING METHOD	START DATE	START TIME
See Figure 2	Mud-rotary	SPT/Shelby	11/30/18	1245
REMARKS	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	FINISH DATE	FINISH TIME
None	282 ft amsl	Not encountered	11/30/18	1515

Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected) 0 25 50 75	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Wet Density (PCF)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0							Approximately 16 to 18 inches of topsoil and grass.					
2	280			CL			Lightly mottled brown to gray lean CLAY, moist, medium stiff to very stiff. [Soil Type 1]					
		SPT SB-2.1	16									
4												
6	276	SPT SB-2.2	11									
8		SPT SB-2.3	5									
10	272			CH			Heavily mottled dark brown to gray fat CLAY, very moist, medium stiff to very stiff. [Soil Type 1]					
		SPT SB-2.4	7						33.2	90.3	47	18
12												
14	268											
16		SPT SB-2.5	15									
18	264						Black nodules observed in soil samples.					
20		SPT SB-2.6	23						29.7	91.8	55	33
22	260											
24												
				CL			Heavily mottled red to orange-brown sandy lean CLAY, moist to saturated, very stiff. [Soil Type 1]					
26	256	SPT SB-2.7	23									
28												
30	252											

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SOIL BORING LOG

PROJECT NAME	CLIENT		PROJECT NO.	BORING NO.
Clark College North County Campus	Clark College		18345	SB-2
PROJECT LOCATION	DRILLING CONTRACTOR	DRILL RIG	ENGINEER	PAGE NO.
Ridgefield, Washington	Western States	CME 55 Track Rig	HDG	2 of 2
BORING LOCATION	DRILLING METHOD	SAMPLING METHOD	START DATE	START TIME
See Figure 2	Mud-rotary	SPT/Shelby	11/30/18	1245
REMARKS	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	FINISH DATE	FINISH TIME
None	282 ft amsl	Not encountered	11/30/18	1515

Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected) 0 25 50 75	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Wet Density (PCF)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
30	252	SPT SB-2.8	17				Heavily mottled red to orange-brown sandy lean CLAY, moist to saturated, very stiff. [Soil Type 1]		30.2	64.8	42	20
32							Soil boring terminated at 31.5 ft bgs. Groundwater not encountered.					
34	248											
36												
38	244											
40												
42	240											
44												
46	236											
48												
50	232											
52												
54	228											
56												
58	224											
60												

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Clark College North County Campus	Clark College		18345	TP-1
PROJECT LOCATION: Ridgefield, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	ENGINEER: HDG	DATE: 11/27/18
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 284 ft amsl	GROUNDWATER DEPTH: Not encountered	START TIME: 0800	FINISH TIME: 0815

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 16 to 18 inches of topsoil and sod in till zone. Material soft to approximately 2 feet bgs.					
		Gee silt loam	A-6	CL		Brown lean CLAY with sand, moist, soft to medium stiff, heavy gray to light tan mottling observed. [Soil Type 1]					
5											
10											
						Bottom of test pit at 14.0 feet. Groundwater not encountered.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Clark College North County Campus	Clark College		18345	TP-2
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**ENGINEER**	**DATE**
Ridgefield, Washington	L&S Contractors	Excavator	HDG	11/27/18
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	285 ft amsl	Not encountered	0820	0845

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 16 to 18 inches of topsoil and sod in till zone. Material soft to approximately 2 feet bgs.					
		Gee silt loam	A-6	CL		Brown lean CLAY with sand, moist, soft to medium stiff, heavy gray to light tan mottling observed. [Soil Type 1]					
5											
10											
						Bottom of test pit at 14.0 feet. Groundwater not encountered.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Clark College North County Campus	Clark College		18345	TP-3
PROJECT LOCATION: Ridgefield, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	ENGINEER: HDG	DATE: 11/27/18
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 282 ft amsl	GROUNDWATER DEPTH: Not encountered	START TIME: 0900	FINISH TIME: 0920

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 16 to 18 inches of topsoil and sod in till zone. Material soft to approximately 2 feet bgs.					
		Gee silt loam	A-6	CL		Brown lean CLAY with sand, moist, soft to medium stiff, heavy gray to light tan mottling observed. [Soil Type 1]					
5											
10											
15						Bottom of test pit at 15.0 feet. Groundwater not encountered.					
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Clark College North County Campus	Clark College		18345	TP-4
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Ridgefield, Washington	L&S Contractors	Excavator	HDG	11/27/18
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	274 ft amsl	Not encountered	0930	0945

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 16 to 18 inches of topsoil and sod in till zone. Material soft to approximately 2 feet bgs.					
	TP-4.1	Odne silt loam	A-6(13)	CL		Brown lean CLAY with sand, moist, soft to medium stiff, heavy gray to light tan mottling observed. [Soil Type 1]	30.0	76.9	40	17	
5											
	TP-4.2		A-6(10)				31.2	79.3	35	13	
10											
15						Bottom of test pit at 14.0 feet. Groundwater not encountered.					
20											

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TEST PIT LOG

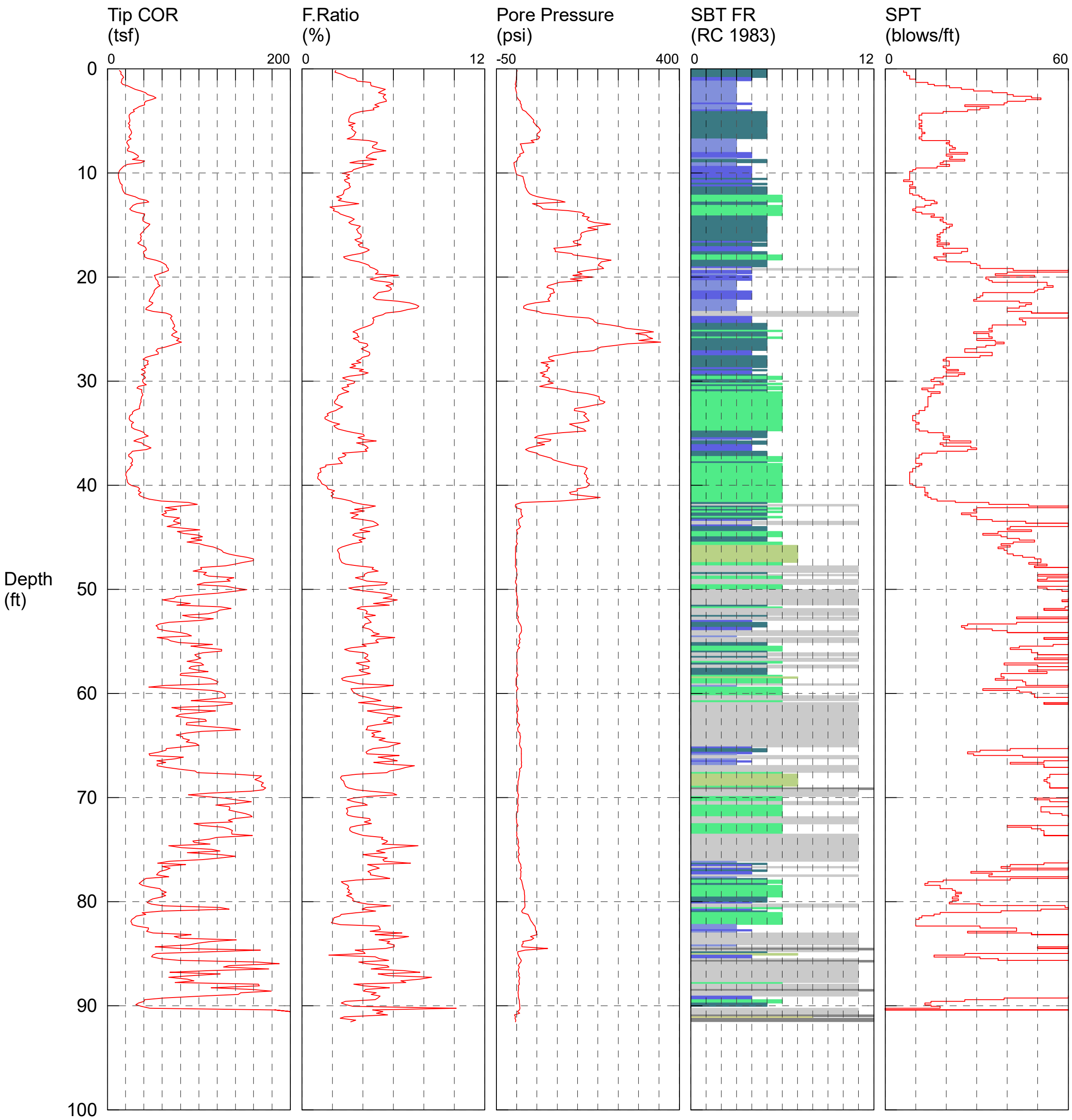
PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Clark College North County Campus	Clark College		18345	TP-5
PROJECT LOCATION: Ridgefield, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	ENGINEER: HDG	DATE: 11/27/18
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 275 ft amsl	GROUNDWATER DEPTH: Not encountered	START TIME: 0950	FINISH TIME: 1005

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 16 to 18 inches of topsoil and sod in till zone. Material soft to approximately 2 feet bgs.					
		Odne silt loam		CL		Brown lean CLAY with sand, moist, soft to medium stiff, heavy gray to light tan mottling observed. [Soil Type 1]					
5											
10											
						Bottom of test pit at 14.0 feet. Groundwater not encountered.					
15											
20											

CPT-01

CPT CONTRACTOR: In Situ Engineering
CUSTOMER: Columbia West Engineering
LOCATION: Ridgefield
JOB NUMBER: 1834.5

OPERATOR: Okbay/Mayfield
CONE ID: DDG1424
TEST DATE: 11/20/2018 9:19:48 AM
PREDRILL:
BACKFILL: 20% Bentonite Slurry
SURFACE PATCH:



COMMENT:

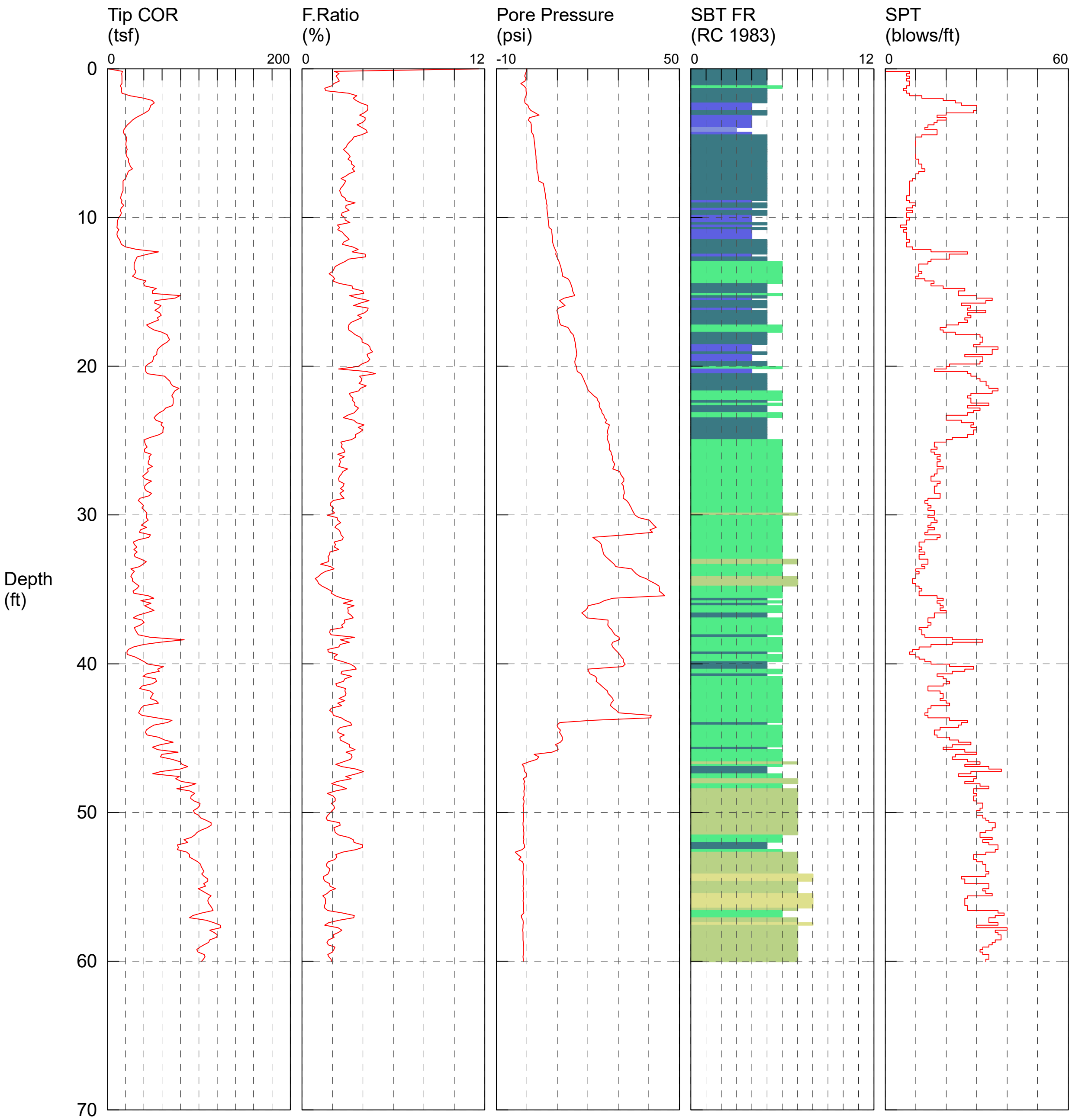
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

CPT-02

CPT CONTRACTOR: In Situ Engineering
CUSTOMER: Columbia West Engineering
LOCATION: Ridgefield
JOB NUMBER: 1834.5

OPERATOR: Okbay/Mayfield
CONE ID: DDG1263
TEST DATE: 11/20/2018 9:42:35 AM
PREDRILL:
BACKFILL: 20% Bentonite Slurry
SURFACE PATCH:



COMMENT:

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

APPENDIX C
SOIL CLASSIFICATION INFORMATION

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

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

Consistency for Cohesive Soil

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

Relative Density for Granular Soil

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

Moisture Designations

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

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

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

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

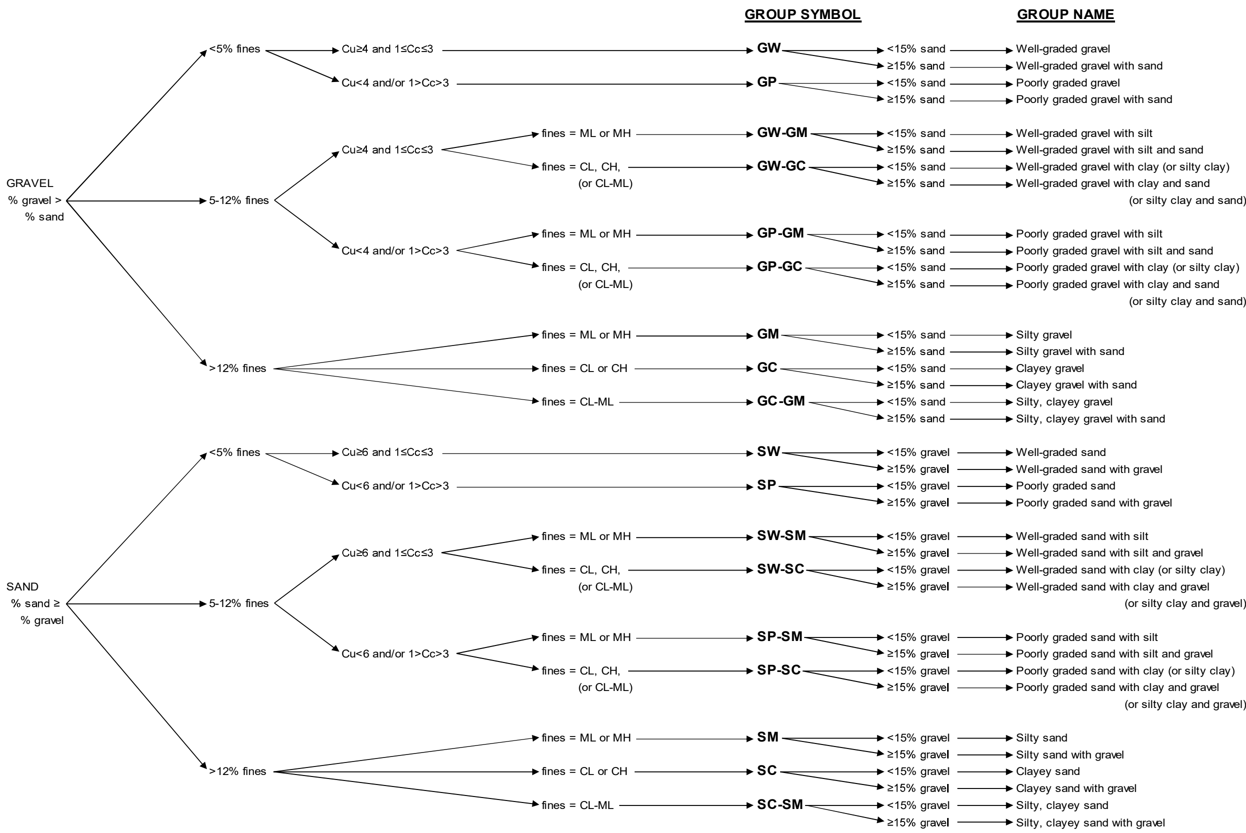
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

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

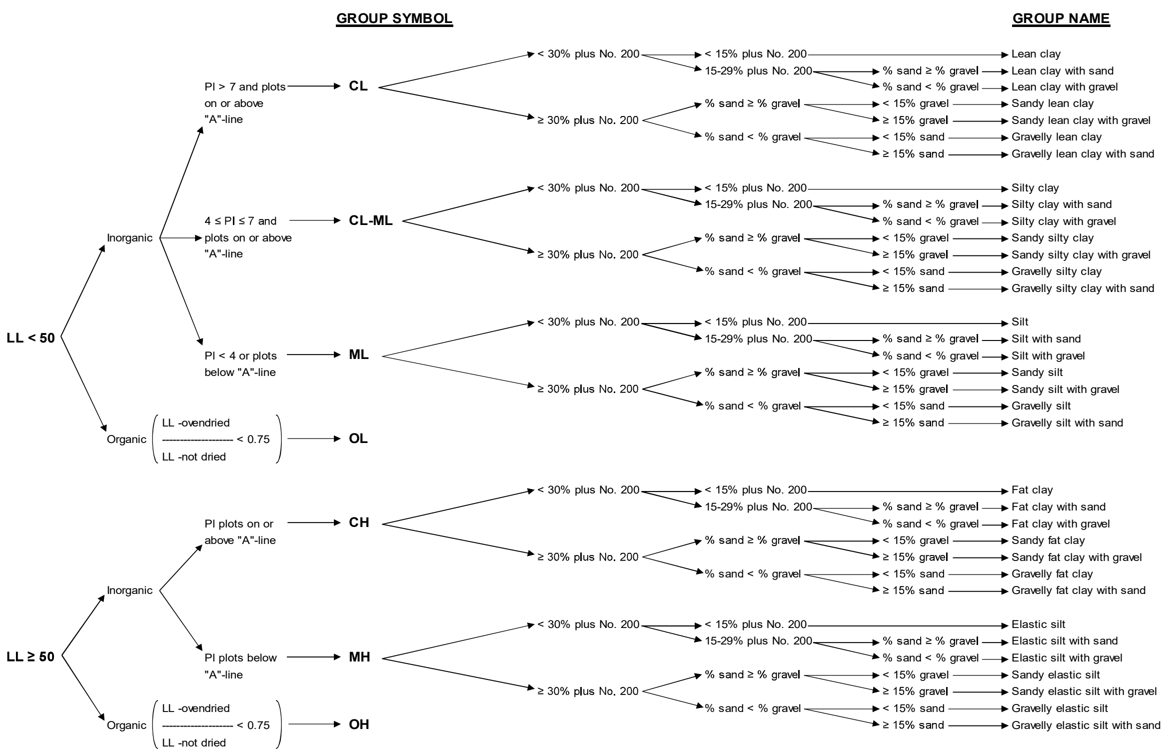
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

USCS SOIL CLASSIFICATION SYSTEM



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



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

APPENDIX D
PHOTO LOG

Clark College North County Campus

Site Overview – November 27, 2018

Ridgefield, Washington

Overview Photograph: Facing northeast from southwest corner of parcel no. 214247000.

Clark College North County Campus

Site Photography Log – November 20, 27 and 30, 2018

Ridgefield, Washington

Photograph 1: Conducting CPT-1 sounding with track-mounted cone apparatus.

Photograph 2: Inside of track-mounted CPT apparatus, running porewater dissipation testing for CPT-1.

Clark College North County Campus

Site Photography Log – November 20, 27 and 30, 2018

Ridgefield, Washington

Photograph 3: Existing residential and agricultural buildings at the approximate northeast site corner.

Photograph 4: Site exploration observations, test pit TP-1.

Clark College North County Campus

Site Photography Log – November 20, 27 and 30, 2018

Ridgefield, Washington

Photograph 5: Standing water observed in drainage ditch at the south parcel boundary of parcel no. 214247000, running east-west.

Photograph 6: Standing water observed in drainage ditch at the west parcel boundary of parcel no. 214247000, running north-south.

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Clark College North County Campus

Site Photography Log – November 20, 27 and 30, 2018

Ridgefield, Washington

Photograph 7: Surface organic soils in test pit TP-5. Observations indicate generally consistent topsoil layers in test pit explorations.

Photograph 8: Site exploration activity with mud-rotary drill apparatus, soil boring SB-1.

Clark College North County Campus

Site Photography Log – November 20, 27 and 30, 2018

Ridgefield, Washington

Photograph 9: Split spoon observed from soil boring SB-2 at 40 feet below ground surface.

Photograph 10: Gray banding observed in soil boring SB-2 at 40 feet below ground surface.

Clark College North County Campus

Test Pit Photography Log – November 27, 2018

Ridgefield, Washington

Test Pit TP-1

Clark College North County Campus

Test Pit Photography Log – November 27, 2018

Ridgefield, Washington

Test Pit TP-2

Clark College North County Campus

Test Pit Photography Log – November 27, 2018

Ridgefield, Washington

Test Pit TP-3

Clark College North County Campus

Test Pit Photography Log – November 27, 2018

Ridgefield, Washington

Test Pit TP-4

Clark College North County Campus

Test Pit Photography Log – November 27, 2018

Ridgefield, Washington

Test Pit TP-5

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

Date: January 10, 2019
Project: Clark College North County Campus
Ridgefield, Washington

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

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

Report Conclusions and Preliminary Nature

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

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

Additional Investigation and Construction QA/QC

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

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

Collected Samples

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

Report Contents

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

Report Limitations for Contractors

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

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Consultant Responsibility

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