Geotechnical Site Investigation

Minit Management
Commercial Development

Ridgefield, Washington

September 4, 2019

Geotechnical ■ Environmental ■ Special Inspections

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GEOTECHNICAL SITE INVESTIGATION
MINIT MANAGEMENT COMMERCIAL DEVELOPMENT
RIDGEFIELD, WASHINGTON

Prepared For:	**Mr. Don Rhoads** **Minit Management, LLC** **P.O. Box 5889** **Vancouver, Washington 98668**
Site Location:	**2814 NW 319th Street** **Parcel No. 209738000** **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:	**September 4, 2019**

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1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Minit Management, LLC to conduct a geotechnical site investigation for the Minit Management Commercial Development 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 the proposed development. The scope of services was outlined in a proposal contract dated July 16, 2019. 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 F.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located at 2814 NW 319th Street in Ridgefield, Washington. The site is comprised of tax parcel number 209738000 and additional unregistered land totaling approximately 4.4 acres. The regulatory jurisdictional agency is the City of La Center, Washington. The approximate latitude and longitude are N 45° 51' 11" and W 122° 42' 04", and the legal description is a portion of the SW ¼ of Section 04, T4N, R1E, Willamette Meridian.

1.2 Proposed Development

As indicated on Figure 2A, Columbia West understands that planned improvements at the site consist of a one-story, 2,300 square-foot drive-through restaurant; a one-story, 5,000 square-foot convenience store and associated fueling island; a one-story 16,680 square-foot multi-tenant retail building; and a four-story, 38,800 square-foot, 93-unit hotel. Development will also include private paved parking and drive aisles, stormwater management facilities, and essential underground utilities. Columbia West has not reviewed a preliminary grading plan but understands that cut and fill areas may be proposed. This report is based upon the 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 *Geological Map of the Ridgefield Quadrangle, Clark County, Washington, and Multnomah County, Oregon,* (U.S. Geological Survey Scientific Investigations Map 2844), near-surface soils are expected to consist of Pleistocene-aged, unconsolidated, rhythmically bedded periglacial clay, silt, and fine- to medium-textured sand deposits derived from catastrophic outburst floods of Glacial Lake Missoula (Qfs).

The *Web Soil Survey* (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2019 Website) identifies surface soils as Gee silt loam and Odne silt loam.

Although soil conditions may vary from the broad USDA descriptions, Gee and Odne soils are generally fine- to medium textured sands, silts and clays with low permeability, moderate to high water capacity, and low shear strength. They are generally moisture sensitive, somewhat compressible, and described as having low to moderate shrink swell potential. They exhibit a slight erosion hazard based primarily on slope grade.

3.0 REGIONAL SEISMOLOGY

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

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

Portland Hills Fault Zone

The Portland Hills Fault Zone consists of several northwest-trending faults located along the northeastern margin of the Tualatin Mountains, also known as the Portland Hills, and the southwest margin of the Portland Basin. The fault zone is approximately 25 to 30 miles in length and is located approximately 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.

Gales Creek-Newberg-Mt. Angel Fault Zone

Located approximately 30 ½ 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 22 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.

<u>Cascadia Subduction Zone</u>

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), one infiltration test (IT-3.1), two cone penetration tests (CPT-1 and CPT-2), and two soil borings (SB-1 and SB-2) was conducted at the site on August 9 and 13, 2019. Test pit exploration was performed with a track-mounted excavator. The CPTs were performed with a truck-mounted CPT rig. Soil borings were performed with a trailer-mounted drill rig. 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. Test pit and soil boring exploration logs are presented in Appendix B. The CPT results report is presented in Appendix C. Soil descriptions and classification information are provided in Appendix D. Photo logs are presented in Appendix E.

4.1 Surface Investigation and Site Description

The approximate 4.4-acre subject site consists of a single parcel and additional unregistered land located at 2814 NW 319th Street in Ridgefield, Washington. The site is currently occupied by Paradise Truck Stop, a Shell station, and associated parking areas and drive aisles. Vegetation on the site primarily consists of manicured landscape islands around the perimeter of the site.

Field reconnaissance and review of site topographic mapping indicates relatively flat to gently rolling terrain with grades generally ranging from 0 to 10 percent. Site elevations generally range from approximately 248 feet above mean sea level (amsl) in the northwest corner to 266 feet amsl in the southeast corner.

4.2 Subsurface Exploration and Investigation

Test pit explorations TP-1 through TP-5 were advanced at the site to a maximum depth of 14 feet below ground surface (bgs). Infiltration testing was conducted at a depth of 2 feet bgs within test pit TP-3. Soil borings SB-1 and SB-2 were performed to a maximum depth of 50 feet bgs. Cone penetration tests CPT-1 and CPT-2 were advanced to a maximum depth of 62.3 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 4 to 12 inches of sod and topsoil in the exploration locations. Underlying these materials, existing fill and subsurface soils resembling native USDA Gee soil series descriptions were generally encountered. Subsurface lithology may generally be described by soil types in the following text.

Soil Type 1 – Existing FILL

Soil Type 1 was observed to generally consist of existing fill. Soil Type 1 was observed at the ground surface in test pits TP-1, TP-2 and soil boring SB-1 and below the topsoil layer in test pits TP-4 and TP-5. Within test pit TP-1 and soil boring SB-1, Soil Type 1 consisted of dark gray to black gravel mixed with topsoil and asphalt grindings and extended to a depth of 10 feet bgs where it was underlain by Soil Type 2. Within test pit TP-2, Soil Type 1 consisted of concrete chunks mixed with native lean clay with sand and extended to a depth of 2 feet bgs where it was underlain by Soil Type 2. Within test pit TP-4, brown sub-rounded to rounded gravels and cobbles, consistent with a septic drain field, were observed to a depth of 3 feet bgs where the test pit was terminated. Within test pit TP-5, Soil Type 1 consisted of brown to gray sub-rounded to rounded gravel and extended to a depth of 4 feet bgs where it was underlain by Soil Type 2. Additional recommendations associated with existing fill are presented later in Section 5.1.1, *Undocumented Fill*.

Soil Type 2 – Lean CLAY / Lean CLAY with Sand

Soil Type 2 was observed to generally consist of brown, tan, reddish-brown, and dark gray, medium stiff to hard, moist to wet lean CLAY and lean CLAY with sand. Soil Type 2 was observed underlying the topsoil layer in test pit TP-3 and soil boring SB-2 and underlying Soil Type 1 in all other explorations, with the exception of test pit TP-4. Soil Type 2 extended to the maximum depth of exploration in all locations in which it was observed.

Analytical laboratory testing conducted upon representative soil samples obtained from test pits TP-1, TP-3 and soil borings SB-1 and SB-2 indicated approximately 70 to 87 percent by weight passing the No. 200 sieve and an in situ moisture contents ranging from 23 to 40 percent. Atterberg Limits analysis indicated the tested samples of Soil Type 2 have liquid limits between 34 and 42 percent and a plasticity index ranging from 14 to 21 percent. The laboratory tested samples of Soil Type 2 are classified as CL according to USCS specifications and A-7-6(19), A-6(11), and A-6(10) according to AASHTO specifications.

4.2.2 Groundwater

Groundwater was not observed within the test pits to the maximum explored depth of 14 feet bgs. Static groundwater was not observed within the soil borings to the maximum explored depth of 50 feet bgs. However, perched groundwater layers were observed within soil borings SB-1 and SB-2 at approximately 20 and 30 feet bgs, respectively. A review of local well logs in the vicinity of the subject site indicates static groundwater was not encountered to the maximum well depth of 100 feet bgs.

Note that 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 site are near-surface fine-textured soils and undocumented fill. 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 from 4 to 12 inches.

The required stripping depth may increase in areas of unsuitable fill, heavy organics, or previously existing structures. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed. The post construction maximum depth of landscape fill placed or spread at any location onsite should not exceed one foot.

Previously disturbed soil, debris, or unconsolidated fill encountered during grading or construction activities should be removed completely and thoroughly from structural areas. This includes old foundations, basement walls, utilities, and debris. Excavated areas should be backfilled with engineered structural fill.

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

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

5.1.1 Existing Fill

As previously discussed, existing fill (Soil Type 1) was observed throughout the subject site. Subsurface exploration and field reconnaissance indicate that existing fill, in the areas observed, primarily consists of dark gray to black gravel mixed with topsoil and asphalt grindings, and concrete chunks mixed with native sandy silt. Site observations and subsurface exploration indicated that existing fill generally extended between 2 to 4 feet below ground surface with the exception of test pit TP-1 and soil boring SB-1 where it extended to approximately 10 feet below ground surface.

Existing unsuitable fill and other previously disturbed soils or debris should be removed completely and thoroughly from structural areas. In some areas existing fill may directly overlie vegetation or the original topsoil layer. This material should also be removed completely from structural areas. Upon removal of existing fill, Columbia West should observe the exposed subgrade. It should be noted that the limited scope of exploration conducted for this investigation cannot wholly eliminate uncertainty regarding the presence of unsuitable soils in areas not explored.

Based upon Columbia West's investigation, most existing fill soils do not appear to be acceptable for reuse as structural fill. Some existing fill materials, such as those encountered in test pit TP-5, may be suitable for reuse as structural fill provided materials are observed to exhibit index properties similar to those observed during this investigation and that construction adheres to the specifications presented in this report. Portions of existing fill found to contain highly organic or clayey soils, debris, boulders, or other deleterious material should be removed. Recommendations regarding the suitability of reusing existing fill soils as structural fill material should be provided in the field by Columbia West during construction.

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. For engineered structural fill placed on sloped grades, the area 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. Clean native soils may be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native soils may require addition of moisture during late summer months or after extended periods of warm dry weather. Compacted fine-textured fill soils should be covered shortly after placement.

Because they are moisture-sensitive, fine-textured native soils are often difficult to excavate and compact during wet weather construction. 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 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

Foundations are anticipated to consist of shallow continuous perimeter or column spread footings. Typical building loads are not expected to exceed approximately 6 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. Footings should be designed by a licensed structural engineer and conform to the recommendations below.

The existing ground surface should be prepared as described in Section 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 1,500 psf. Bearing capacity may be increased by one-third for transient lateral forces such as seismic or wind. The estimated coefficient of friction between in situ compacted native soil or engineered structural fill and in-place poured concrete is 0.35. Lateral forces may also be resisted by an assumed passive soil equivalent fluid pressure of 250 psf/f against embedded footings. The upper six inches of soil should be neglected in passive pressure calculations.

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

Foundations should not be permitted to bear upon unsuitable 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.

5.5 Slabs on Grade

If structures are proposed to be constructed with slab-on-grade floors, slabs should be supported on firm, competent, native, in situ soil or engineered structural fill. Disturbed soils and unsuitable fills in proposed slab locations should be removed and replaced with structural fill.

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

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.

5.7 Excavation

Soils at the site were explored to a maximum depth of 14 feet using a track-mounted excavator, 50 feet with a trailer-mounted drill rig, and 62.3 feet using a truck-mounted cone penetrometer rig. Bedrock was not encountered within the explorations and blasting or specialized rock-excavation techniques are not anticipated.

Static groundwater was not observed the explorations. However, perched groundwater layers were encountered within soil borings SB-1 and SB-2 at depths of 20 and 30 feet, respectively. Additional perched groundwater layers may exist at shallow depths depending on seasonal fluctuations of the water table or 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 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, and 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 undisturbed native soils or compacted granular fill. The recommended earth pressures do not include surcharge loads, dynamic loading, hydrostatic pressure, or seismic design.

Table 1. Lateral Earth Pressure Parameters for Level Backfill

Retained / Backfill Material	Equivalent Fluid Pressure for Level Backfill			Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive		
Undisturbed native Lean CLAY with Sand (Soil Type 2)	58 pcf	38 pcf	345 pcf	115 pcf	28°
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.*

If seismic design is required for unrestrained walls, seismic forces may be calculated by superimposing a uniform lateral force of $10H^2$ pounds per lineal foot of wall, where H is the total wall height in feet. If seismic design is required for restrained walls, seismic forces may be calculated by superimposing a uniform lateral force of $25H^2$ pounds per lineal foot of wall. The resultant force should be applied at 0.6H from the base of the wall. If sloped backfill conditions are proposed for the site, Columbia West should be contacted for additional analysis and associated recommendations.

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 *ASCE 7 Hazards 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.

The listed probabilistic ground motion values are based upon "firm rock" sites with an assumed shear wave velocity of 2,500 ft/s in the upper 100 feet of soil profile. These values should be adjusted for site class effects by applying site coefficients Fa, Fv, and F_{PGA} as defined in *ASCE 7-10, Tables 11.4-1, 11.4-2, and 11.8-1,* respectively. 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.

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

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.

The *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004) and site-specific testing indicates site soils may be represented by Site Class C. However, based upon site-specific seismic testing performed within CPT-1, the site is more accurately characterized by Site Class D. This site class designation indicates that amplification of seismic energy may occur during a seismic event because of subsurface conditions. However, this is typical for many areas within Clark County and will not prohibit development if properly accounted for during the design process.

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 to 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 or non-plastic silt 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 groundwater levels do not generally pose a liquefaction hazard. It is important to note that changes in perched groundwater elevation may occur due to project development or other factors not observed at the time of investigation.

Based upon results of laboratory analysis and site-specific testing, observed site soils do not meet the criteria described above for liquefiable soils. Therefore, the potential for liquefaction of site soils significantly impacting proposed improvements is considered to be low.

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 La Center regulations. Finished site grading should be conducted with positive drainage away from structures. Depressions or shallow areas that may retain ponding water should be avoided. Roof drains, low-point drains, and perimeter foundation drains are recommended for structures. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from foundations into 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.

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 Infiltration Testing Results and Recommendations

To facilitate design of stormwater management infrastructure, Columbia West conducted in situ infiltration testing within test pit TP-3 on August 13, 2019. Infiltration test data is presented in Table 3. The USCS soil classification presented in Table 3 is based upon laboratory analysis. The recommended infiltration rate is presented as a coefficient of permeability (k) and has been reported without application of a factor of safety.

The tests was conducted in test pit TP-3 at the indicated depth. Soils in the tested location were observed and sampled where appropriate to adequately characterize the subsurface profile. Tested native soils are classified as lean CLAY with sand (CL).

Single-ring, falling head infiltration testing was performed by inserting a three-inch diameter pipe into the soil at the noted depth. The test was conducted by filling the pipe with water and measuring time relative to changes in hydraulic head at regular intervals. Using Darcy's Law for saturated flow in homogeneous media, the coefficient of permeability (k) was then calculated.

The reported infiltration rates are approximate, reflect recommended coefficients of permeability, and do not include a factor of safety. It is important to note that site soil conditions and localized infiltration rates may be variable. The observed infiltration rates and classifications are based upon Columbia West's observations during limited subsurface exploration.

Table 3. Infiltration Test Data

Test Number	Location (See Figure 2)	Approximate Test Depth (feet bgs)	Groundwater Depth On 08-13-19	USCS Soil Type	Passing No. 200 Sieve (%)	Infiltration Rate (Coefficient of Permeability, k) (inches/hour)**
IT-3.1	TP-3	2.0	Not Observed to 12 feet bgs.	CL, Lean CLAY with Sand	70.0	< 0.1

*Indicates visual USCS soil classification.
**Infiltration rate based upon soil's approximate vertical coefficient of permeability (k).

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

5.14 Bituminous Asphalt and Portland Cement Concrete

Based upon review of preliminary site plans, proposed development includes new private parking and access drives within the subject site. Columbia West recommends adherence to City of La Center paving guidelines for roadway improvements in the public right-of-way. General recommendations for private onsite flexible pavement sections are summarized below in Table 4.

Table 4. Private Onsite Flexible Pavement Section Recommendations

Pavement Section Layer	Minimum Layer Thickness			Specifications
	Car Parking	Access Drives	*Heavy Trucks	
Asphalt concrete surface (HMA Class ½" PG 64-22)	3 inches	3 inches	4 inches	92 percent of maximum Rice density (ASTM D2041)
Base course (WSDOT 9-03.9(3) 1¼"-0 crushed aggregate)	6 inches	8 inches	10 inches	95 percent of maximum modified Proctor density (ASTM D1557)
Scarified and compacted existing subgrade material	12 inches	12 inches	12 inches	Compacted to 95 percent of maximum modified Proctor density (ASTM D1557)

*General recommendation based upon maximum traffic loading of up to 30 heavy trucks per day. If actual truck traffic substantially exceeds 30 trucks per day, reduced pavement serviceability and design life should be expected. Pavement section recommendations do not include or incorporate construction traffic loading.

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 5.15, *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 in the above table. Asphalt concrete pavement should be compacted to at least 92 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 La Center specifications.

Portland cement concrete curbs and sidewalks should be installed in accordance with City of La Center specifications. Curb and sidewalk aggregate base should 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.

5.15 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.

5.16 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, soil disturbance 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.

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 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 F. This information should be carefully read and understood by the client and other parties reviewing this document.

Sincerely,

COLUMBIA WEST ENGINEERING, Inc.

Lance V. Lehto, PE, GE
President

Jason F Merritt, PE
Project Engineer

LANCE V. LEHTO
STATE OF WASHINGTON
36301
REGISTERED
PROFESSIONAL ENGINEER

9-4-19

REFERENCES

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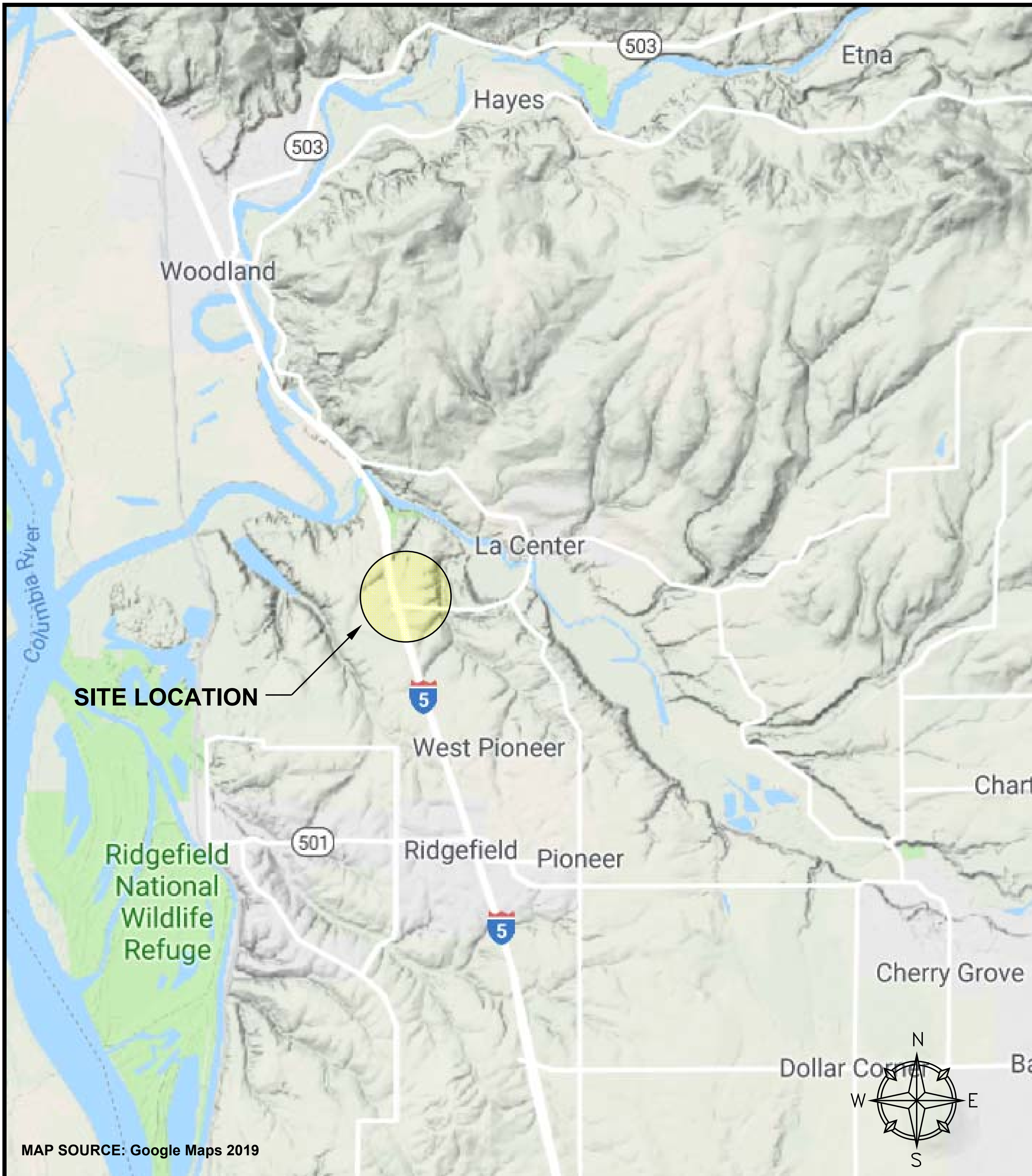
FIGURES



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Design	Drawn: JFM		
Checked: LVL	Date: 08/30/2019		
Client: MINIT	Rev	By	Date
Job No.: 19210			
CAD File: FIGURE 1			
Scale: NTS			

SITE LOCATION MAP

MINIT MANAGEMENT
COMMERCIAL DEVELOPMENT
RIDGEFIELD, WASHINGTON

FIGURE
1

Infiltration Test Results

Test Number	Location	Approximate Test Depth (feet bgs)	Groundwater Depth On 08-13-19	USCS Soil Type (*Indicates Visual Classification)	Passing No. 200 Sieve (%)	Infiltration Rate (Coefficient of Permeability, k) (inches/hour)
IT-3.1	TP-3	2	Not Observed to 12 Feet bgs	CL, Lean CLAY with Sand	70	< 0.1

NOTES:
1. SITE LOCATION: 2814 NW 319TH STREET IN RIDGEFIELD, WASHINGTON.
2. SITE CONSISTS OF PARCEL NO. 209738000 AND ADDITIONAL UNREGISTERED LAND TOTALING APPROXIMATELY 4.4 ACRES.
3. DRAWING IS NOT TO SCALE.
4. AREAL IMAGE SOURCED FROM GOOGLE EARTH.
5. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
6. CPTS BACKFILLED WITH BENTONITE ON AUGUST 9, 2019. TEST PITS BACKFILLED LOOSELY WITH ONSITE SOIL AND SOIL BORINGS BACKFILLED WITH BENTONITE ON AUGUST 13, 2019.
7. INFILTRATION RATES ARE APPROXIMATE COEFFICIENTS OF PERMEABILITY AND DO NOT INCLUDE A FACTOR OF SAFETY.

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Design:	Drawn: JFM			EXPLORATION LOCATION MAP	FIGURE 2
Checked: LVL	Date: 08/26/19				
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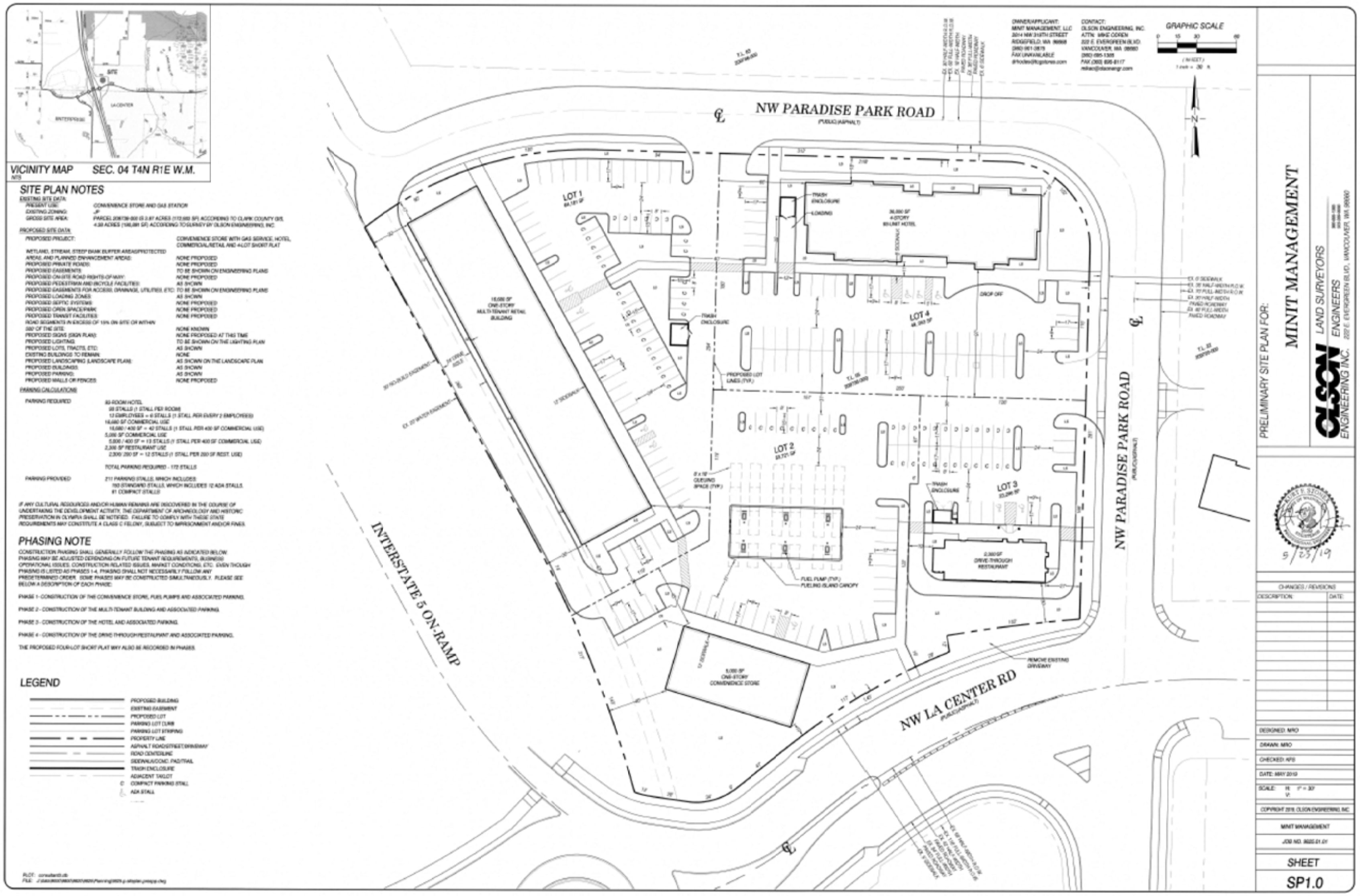
Job No: 19210					
CAD File: FIGURE 2					
Scale: NONE					



NOTES:
1. BASE MAP PROVIDED BY OLSON ENGINEERING, INC.

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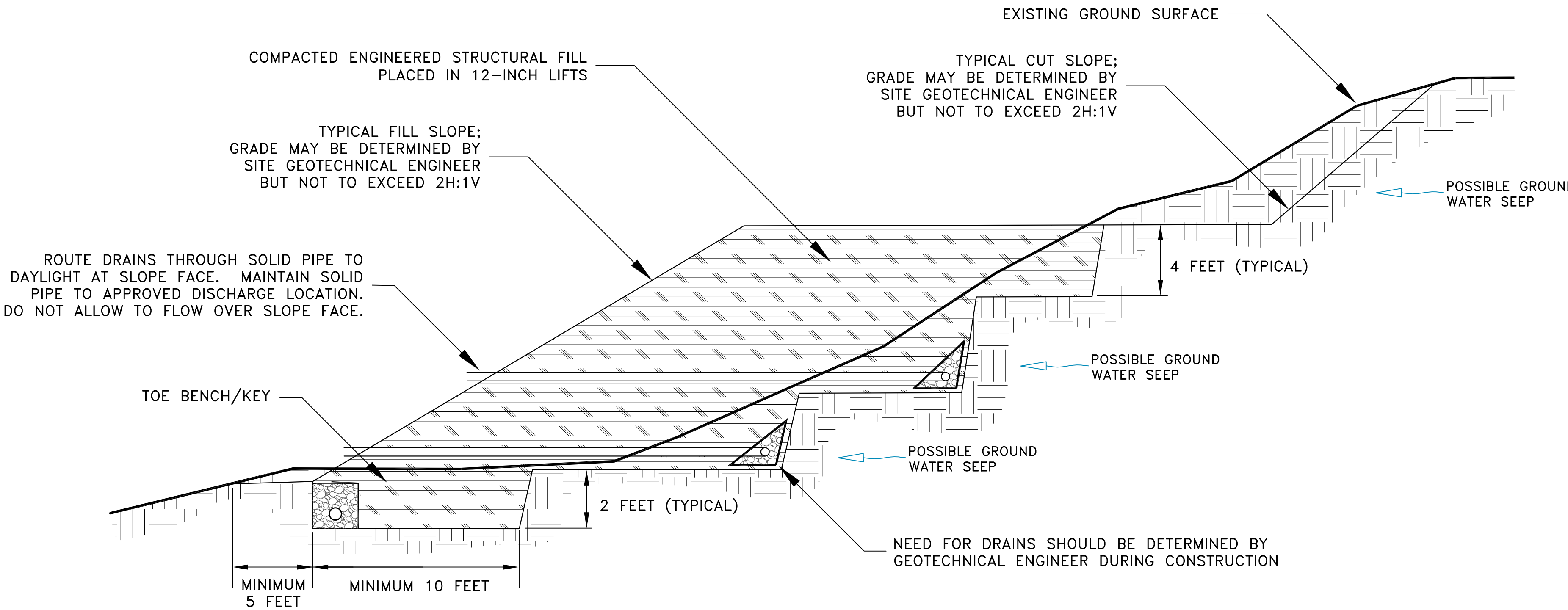
Checked: LVL	Date: 08/30/19			
Client: MINIT	Rev	By	Date	
Job No: 19210				
CAD File: FIGURE 2A				
Scale: NONE				

PROPOSED DEVELOPMENT PLAN

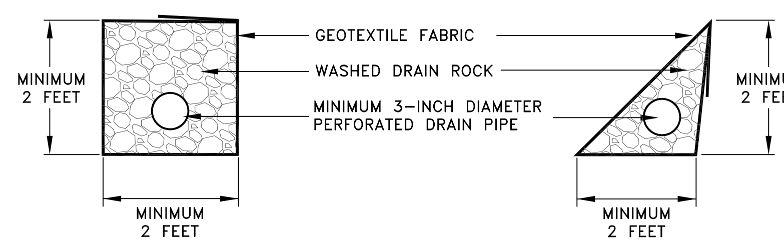
MINIT MANAGEMENT
COMMERCIAL DEVELOPMENT
RIDGEFIELD, WASHINGTON

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: JFM			TYPICAL CUT AND FILL SLOPE CROSS-SECTION
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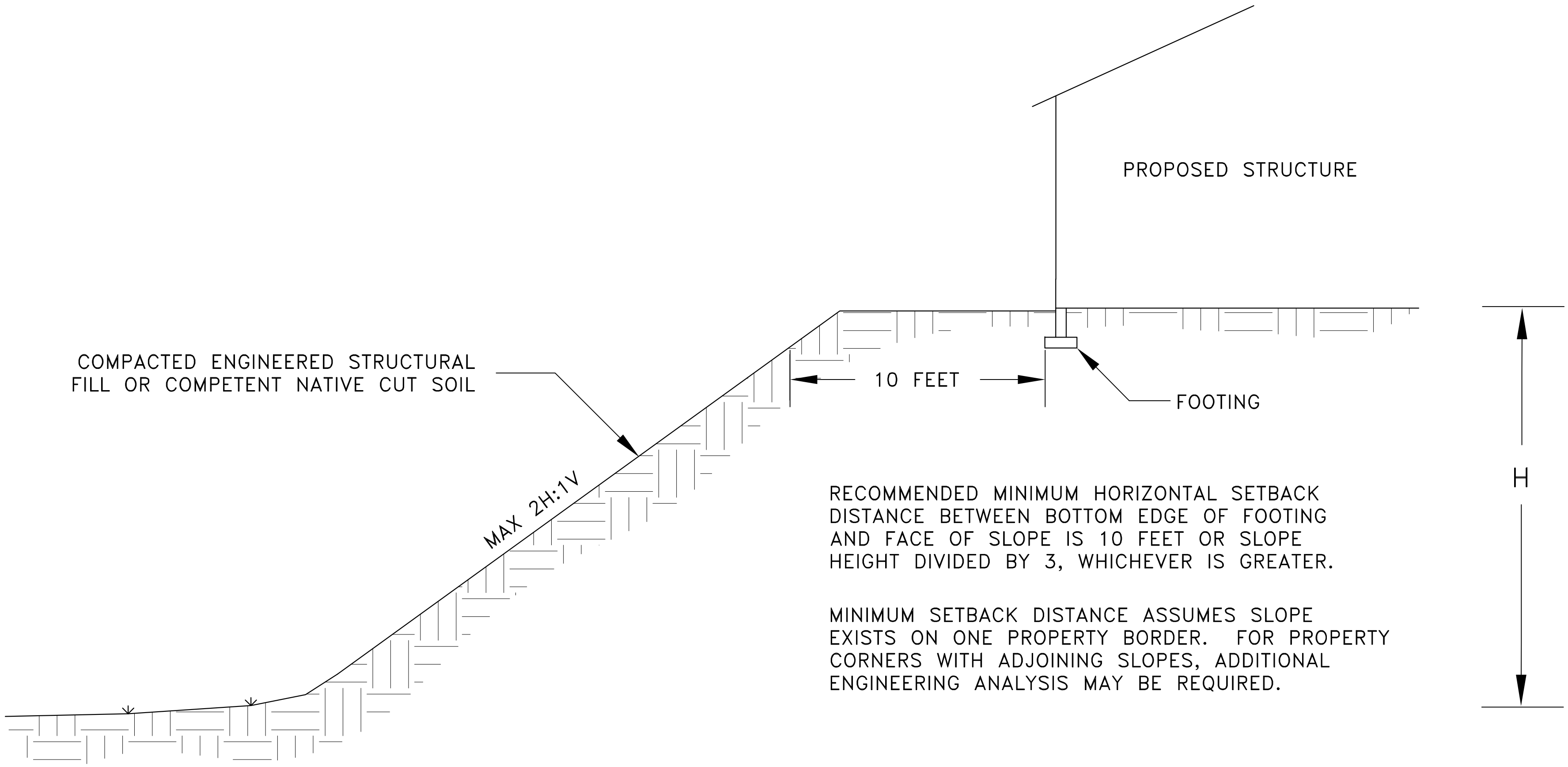
Job No: 19210				MINIT MANAGEMENT COMMERCIAL DEVELOPMENT
CAD File: FIGURE 3				RIDGEFIELD, WASHINGTON
Scale: NONE				

FIGURE 3

MINIMUM FOUNDATION SLOPE SETBACK DETAIL



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

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

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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Job No: 19210				
CAD File: FIGURE 4				
Scale: NONE				

TYPICAL MINIMUM SLOPE SETBACK DETAIL

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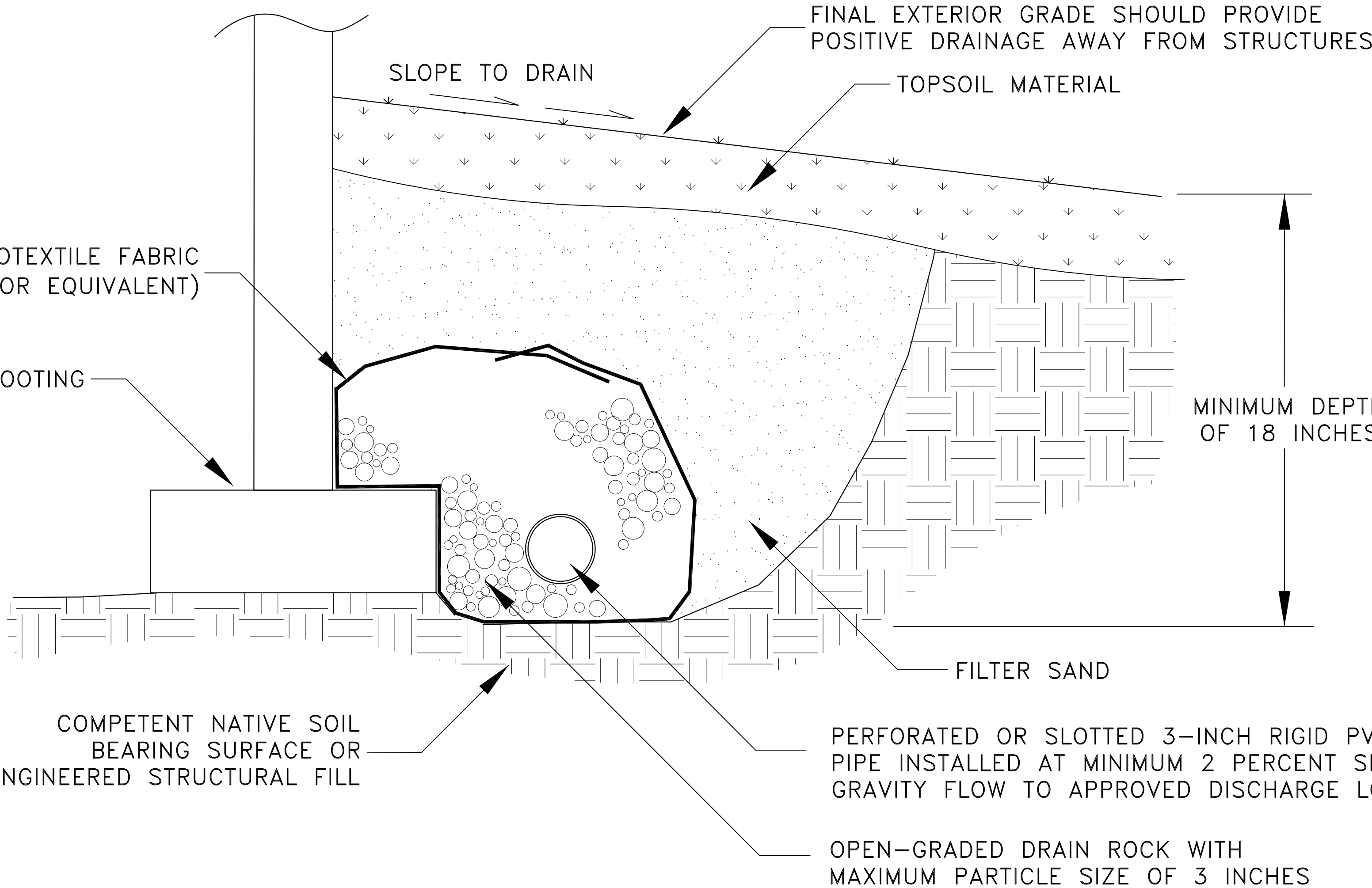
RIDGEFIELD, WASHINGTON

FIGURE 4

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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Design:	Drawn: JFM			TYPICAL PERIMETER FOOTING DRAIN DETAIL
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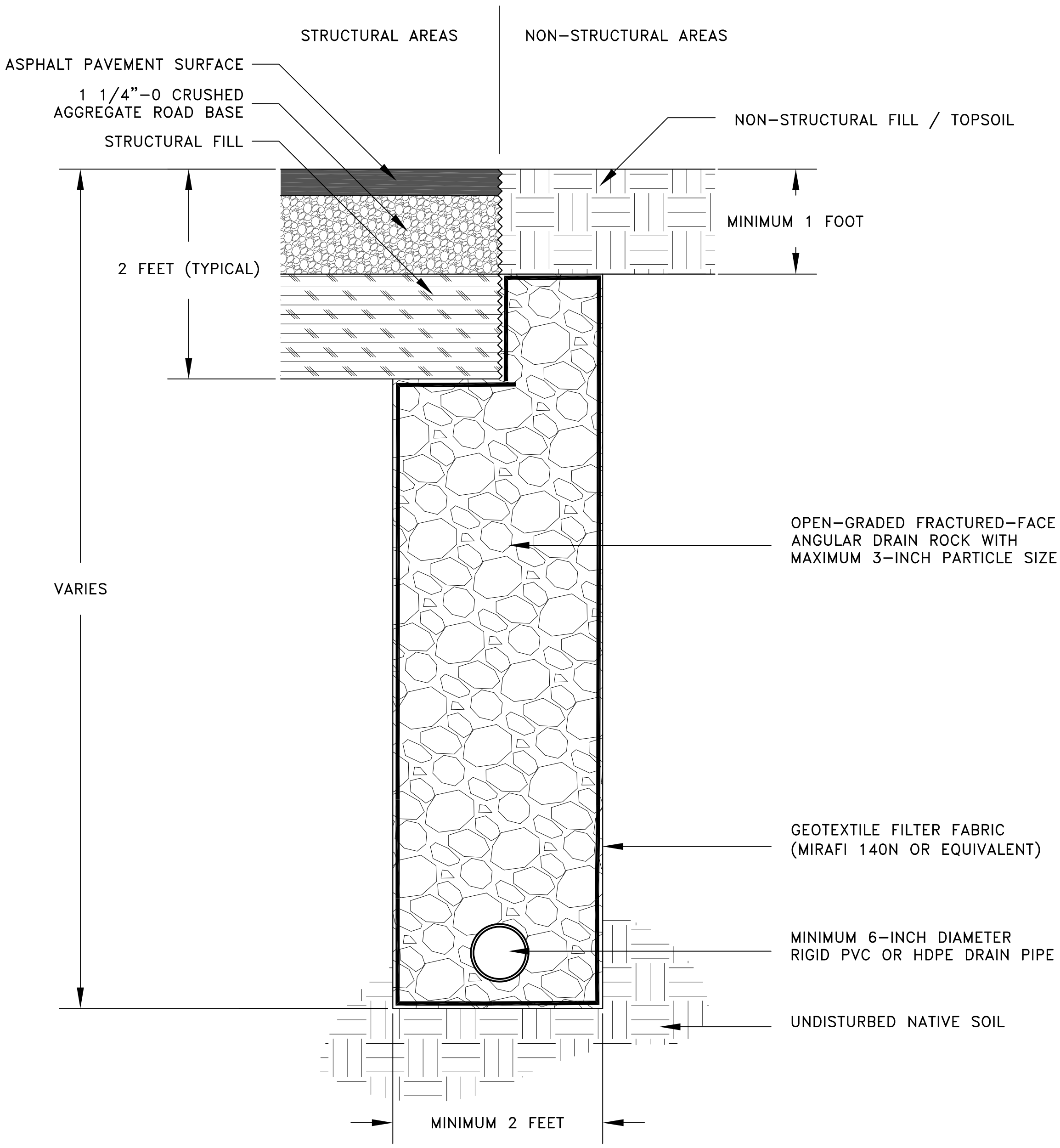
Client: MINIT MANAGEMENT	Rev	By	Date	
Job No: 19210				MINIT MANAGEMENT COMMERCIAL DEVELOPMENT
CAD File: FIGURE 5				
Scale: NONE				RIDGEFIELD, WASHINGTON

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: JFM			TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL
Checked: LVL	Date: 08/26/19			
Client: MINIT MANAGEMENT	Rev	By	Date	MINIT MANAGEMENT COMMERCIAL DEVELOPMENT RIDGEFIELD, WASHINGTON
Job No: 19210				
CAD File: FIGURE 6				
Scale: NONE				

FIGURE 6

APPENDIX A
LABORATORY TEST RESULTS

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MOISTURE CONTENT

PROJECT	CLIENT	PROJECT NO.	REPORT DATE
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	08/26/19
		DATE SAMPLED: 08/13/19	
		SAMPLED BY: JFM/CTB	

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Despatch LEB2	ASTM D2216, Method B

LAB ID	CONTAINER MASS	MOIST MASS + PAN	DRY MASS + PAN	MATERIAL DESCRIPTION	FIELD ID	SAMPLE DEPTH	MOISTURE CONTENT
S19-799	87.70	392.93	335.08	Lean CLAY with Sand	TP1.1	11 feet	23.4%
S19-800	87.36	355.13	301.74	Lean CLAY with Sand	TP3.1	2 feet	24.9%
S19-801	87.61	210.23	187.50	clay with sand	SB1.4	10 feet	22.8%
S19-802	86.47	269.82	227.46	clay	SB1.5	15 feet	30.0%
S19-803	86.69	302.01	245.01	Lean CLAY	SB1.6	20 feet	36.0%
S19-804	87.86	284.21	241.56	clay	SB1.7	25 feet	27.7%
S19-805	87.50	295.85	246.79	clay	SB1.8	30 feet	30.8%
S19-806	87.25	284.49	228.15	silt	SB1.9	35 feet	40.0%
S19-807	86.85	299.21	255.72	clay	SB1.10	40 feet	25.8%
S19-808	87.41	300.84	245.04	clay	SB1.12	50 feet	35.4%
S19-809	85.78	292.27	251.03	clay with sand	SB2.2	5 feet	25.0%
S19-810	87.23	304.10	254.26	clay	SB2.3	7.5 feet	29.8%

NOTES:	DATE TESTED	TESTED BY
	08/23/19	KMS/BTT/JJC

COLUMBIA WEST ENGINEERING, INC. authorized signature

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MOISTURE CONTENT

PROJECT	CLIENT	PROJECT NO.	REPORT DATE
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	08/26/19
		DATE SAMPLED: 08/13/19	
		SAMPLED BY: JFM/CTB	

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Despatch LEB2	ASTM D2216, Method B

LAB ID	CONTAINER MASS	MOIST MASS + PAN	DRY MASS + PAN	MATERIAL DESCRIPTION	FIELD ID	SAMPLE DEPTH	MOISTURE CONTENT
S19-811	87.07	266.68	229.35	clay	SB2.4	10 feet	26.2%
S19-812	85.29	285.33	245.68	clay	SB2.5	15 feet	24.7%
S19-813	87.83	270.36	235.14	clay	SB2.6	20 feet	23.9%
S19-814	87.94	288.99	243.44	clay	SB2.7	25 feet	29.3%
S19-815	86.57	279.37	232.98	clay	SB2.8	30 feet	31.7%
S19-816	88.01	299.02	247.18	Lean CLAY with Sand	SB2.9	35 feet	32.6%
S19-817	86.83	271.70	220.54	silt with sand	SB2.10	40 feet	38.3%
S19-818	87.98	255.90	214.36	silt	SB2.11	45 feet	32.9%
S19-819	85.96	250.59	206.13	silt	SB2.12	50 feet	37.0%

NOTES:	DATE TESTED	TESTED BY
	08/23/19	KMS/BTT/JJC

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	S19-799
		REPORT DATE: 08/26/19	FIELD ID: TP1.1
		DATE SAMPLED: 08/13/19	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-01 depth = 11 feet	CL, Lean Clay with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-6(11)

LABORATORY TEST DATA

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

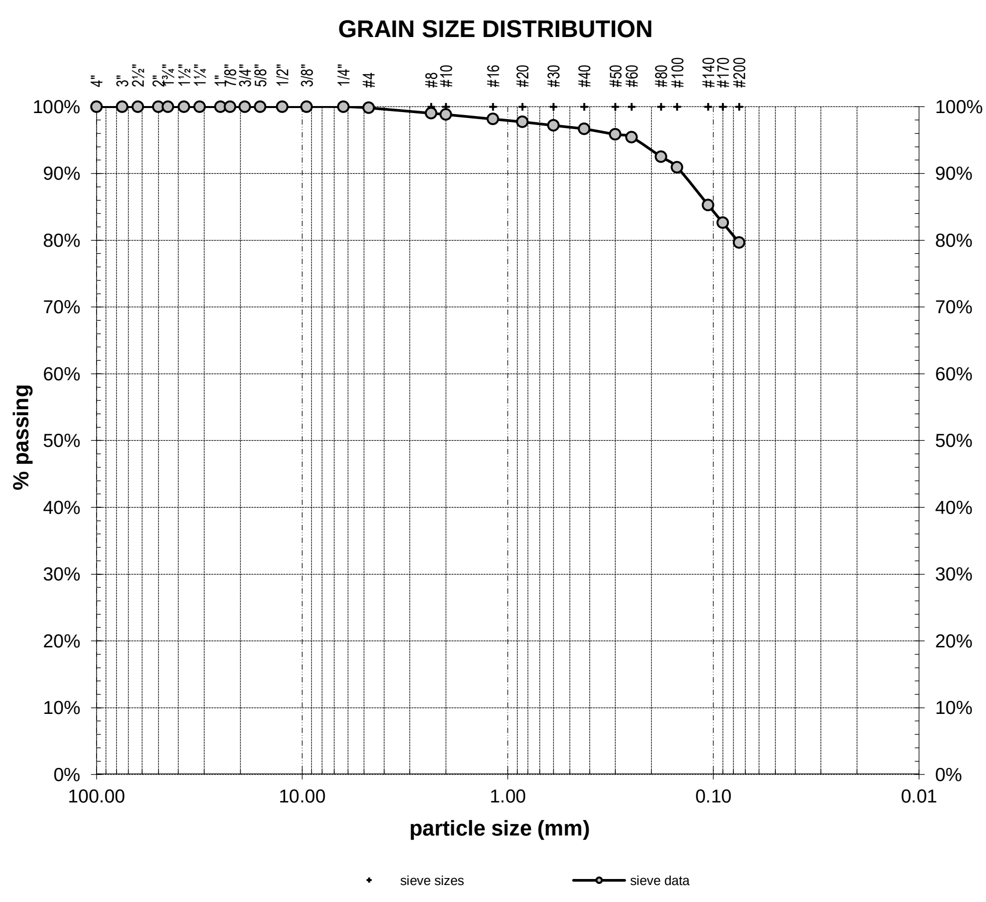
ADDITIONAL DATA

initial dry mass (g) =	244.65		
as-received moisture content =	23.4%	coefficient of curvature, C_C =	n/a
liquid limit =	34	coefficient of uniformity, C_U =	n/a
plastic limit =	19	effective size, $D_{(10)}$ =	n/a
plasticity index =	15	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.2%
% sand = 20.2%
% silt and clay = 79.6%

	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		99%		
	#10	2.00	99%			
	#16	1.18		98%		
	#20	0.850	98%			
	#30	0.600		97%		
	#40	0.425	97%			
	#50	0.300		96%		
	#60	0.250	95%			
	#80	0.180		93%		
	#100	0.150	91%			
	#140	0.106		85%		
	#170	0.090		83%		
	#200	0.075	80%			



DATE TESTED	TESTED BY
08/22/19	BTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	S19-799
		REPORT DATE: 08/26/19	FIELD ID: TP1.1
		DATE SAMPLED: 08/13/19	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-01 depth = 11 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

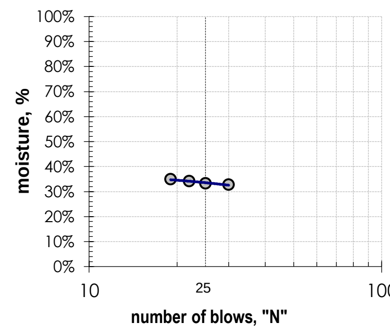
ATTERBERG LIMITS

liquid limit =	34
plastic limit =	19
plasticity index =	15

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.93	32.67	31.85	32.94
dry soil + pan weight, g =	29.18	29.66	29.07	29.80
pan weight, g =	20.77	20.62	20.92	20.80
N (blows) =	30	25	22	19
moisture, % =	32.7 %	33.3 %	34.1 %	34.9 %

LIQUID LIMIT



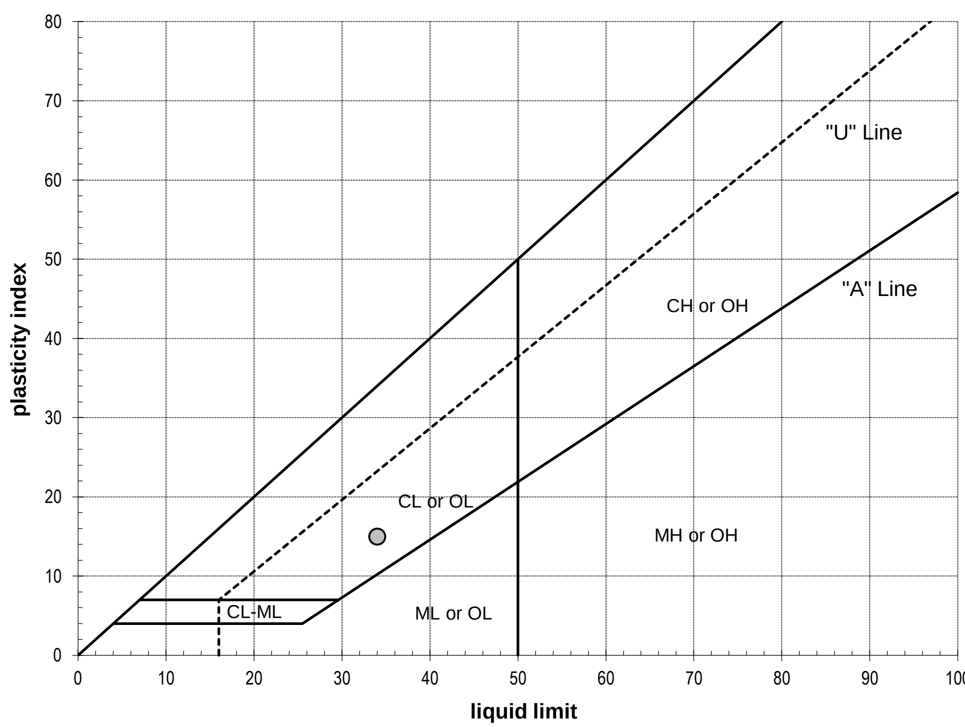
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.32	28.27		
dry soil + pan weight, g =	26.29	27.09		
pan weight, g =	20.86	20.68		
moisture, % =	19.0 %	18.4 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.2%
% sand =	20.2%
% silt and clay =	79.6%
% silt =	n/a
% clay =	n/a
moisture content =	23.4%

DATE TESTED	TESTED BY
08/23/19	KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	S19-800
		REPORT DATE: 08/26/19	FIELD ID: TP3.1
		DATE SAMPLED: 08/13/19	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-03 depth = 2 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) =	201.31		
as-received moisture content =	24.9%	coefficient of curvature, C_C =	n/a
liquid limit =	38	coefficient of uniformity, C_U =	n/a
plastic limit =	22	effective size, $D_{(10)}$ =	n/a
plasticity index =	16	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

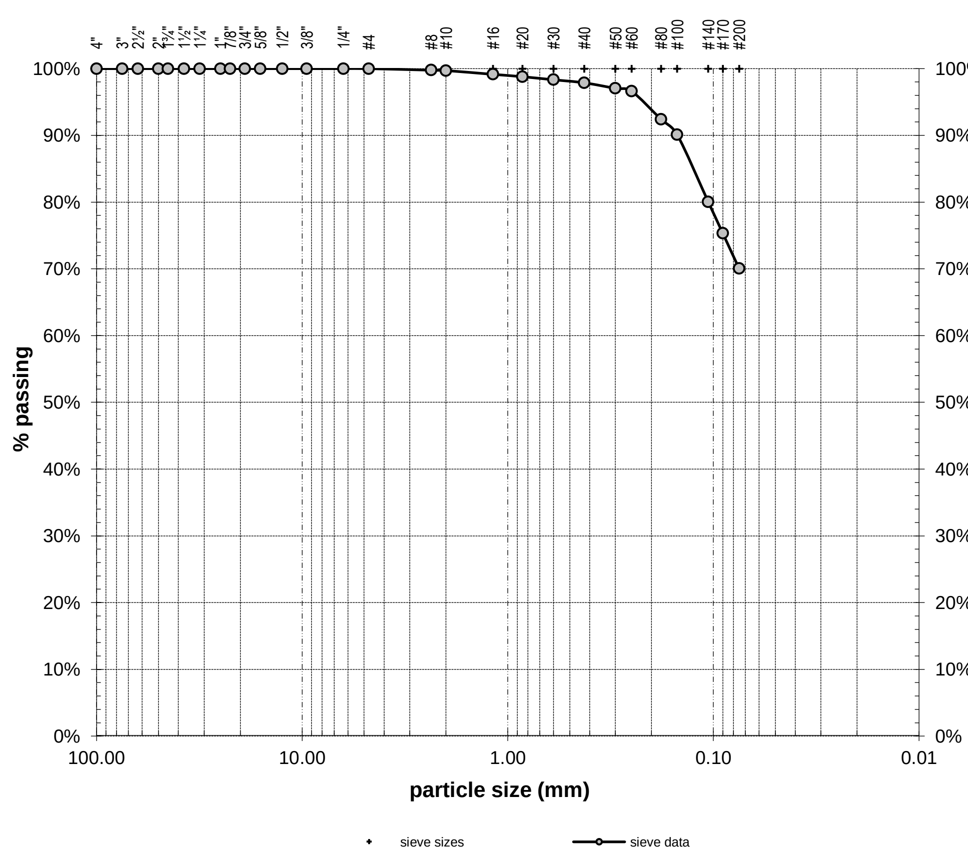
SIEVE DATA

% gravel = 0.0%
% sand = 30.0%
% silt and clay = 70.0%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		99%		
	#20	0.850	99%			
	#30	0.600		98%		
	#40	0.425	98%			
	#50	0.300		97%		
	#60	0.250	97%			
	#80	0.180		92%		
	#100	0.150	90%			
	#140	0.106		80%		
	#170	0.090		75%		
	#200	0.075	70%			

DATE TESTED	TESTED BY
08/22/19	BTT

GRAIN SIZE DISTRIBUTION



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	S19-800
		REPORT DATE: 08/26/19	FIELD ID: TP3.1
		DATE SAMPLED: 08/13/19	SAMPLED BY: HDG

MATERIAL DATA

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

LABORATORY TEST DATA

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

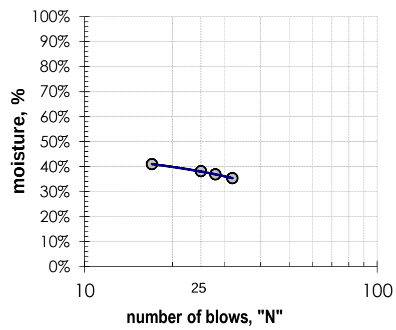
ATTERBERG LIMITS

liquid limit = 38
plastic limit = 22
plasticity index = 16

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.32	33.45	33.58	33.96
dry soil + pan weight, g =	30.79	30.06	30.08	30.15
pan weight, g =	20.81	20.85	20.90	20.83
N (blows) =	32	28	25	17
moisture, % =	35.4 %	36.8 %	38.1 %	41.0 %

LIQUID LIMIT



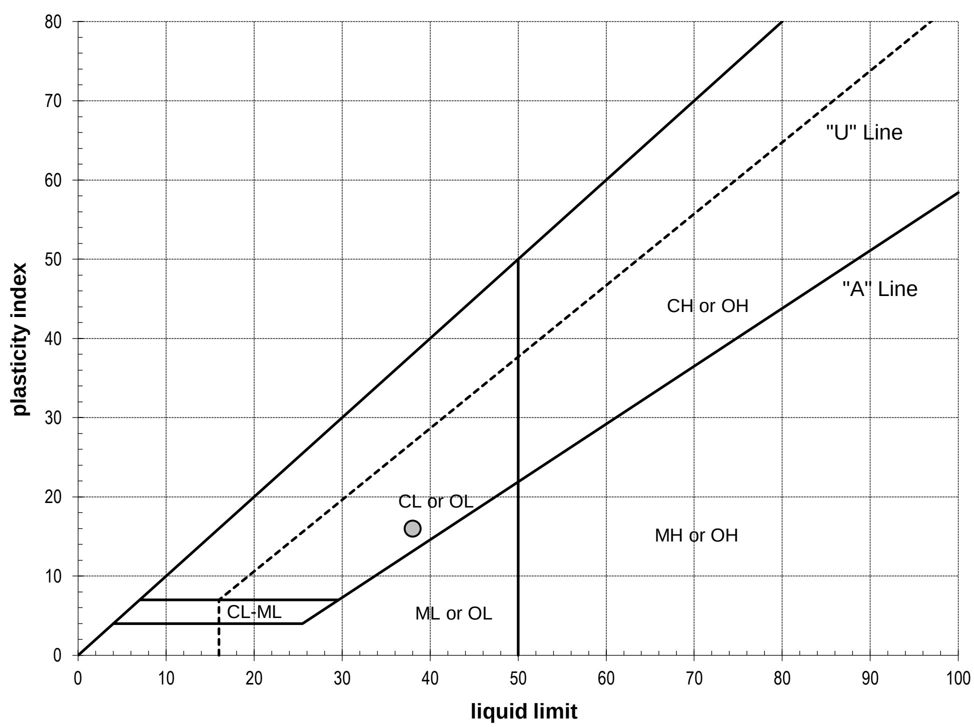
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.83	27.10		
dry soil + pan weight, g =	26.54	25.96		
pan weight, g =	20.75	20.87		
moisture, % =	22.3 %	22.4 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 30.0%
% silt and clay = 70.0%
% silt = n/a
% clay = n/a
moisture content = 24.9%

DATE TESTED	TESTED BY
08/23/19	KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	S19-803
		REPORT DATE: 08/26/19	FIELD ID: SB1.6
		DATE SAMPLED: 08/13/19	SAMPLED BY: JFM/CTB

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(19)

LABORATORY TEST DATA

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

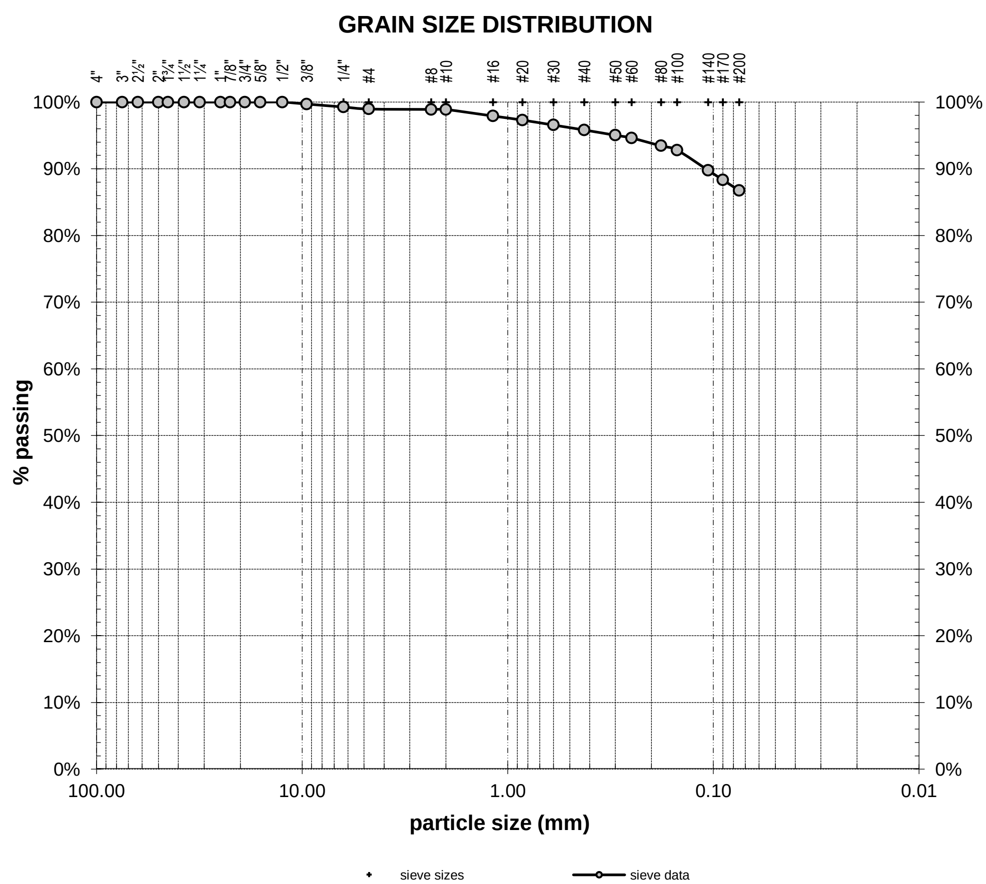
ADDITIONAL DATA

initial dry mass (g) =	174.32		
as-received moisture content =	36.0%	coefficient of curvature, C_C =	n/a
liquid limit =	42	coefficient of uniformity, C_U =	n/a
plastic limit =	21	effective size, $D_{(10)}$ =	n/a
plasticity index =	21	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 1.0%
% sand = 12.2%
% silt and clay = 86.7%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5	100%			
	3/8"	9.50		100%		
	1/4"	6.30		99%		
	#4	4.75	99%			
SAND	#8	2.36		99%		
	#10	2.00	99%			
	#16	1.18		98%		
	#20	0.850	97%			
	#30	0.600		97%		
	#40	0.425	96%			
	#50	0.300		95%		
	#60	0.250	95%			
	#80	0.180		93%		
	#100	0.150	93%			
	#140	0.106		90%		
	#170	0.090		88%		
	#200	0.075	87%			



DATE TESTED	TESTED BY
08/23/19	BTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	S19-803
		REPORT DATE: 08/26/19	FIELD ID: SB1.6
		DATE SAMPLED: 08/13/19	SAMPLED BY: JFM/CTB

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

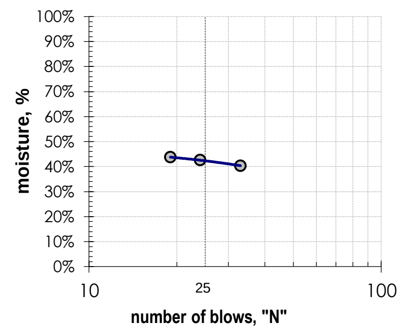
ATTERBERG LIMITS

liquid limit = 42
plastic limit = 21
plasticity index = 21

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.58	31.60	30.61	
dry soil + pan weight, g =	28.52	28.39	27.62	
pan weight, g =	20.93	20.86	20.78	
N (blows) =	33	24	19	
moisture, % =	40.3 %	42.6 %	43.7 %	

LIQUID LIMIT



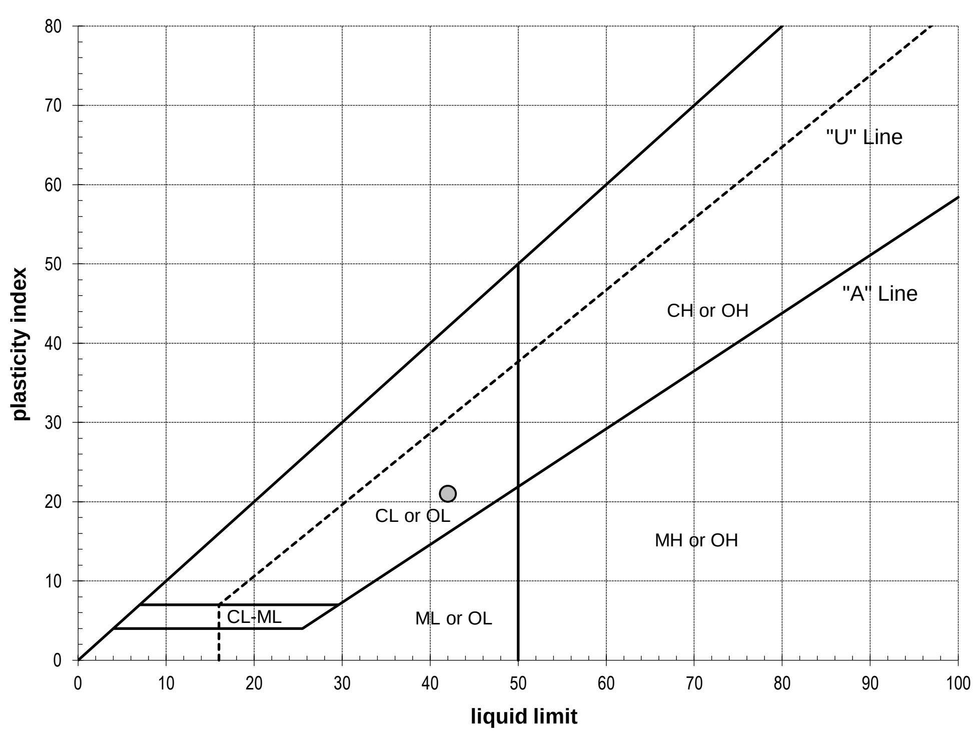
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.66	28.32		
dry soil + pan weight, g =	26.47	26.98		
pan weight, g =	20.74	20.60		
moisture, % =	20.8 %	21.0 %		

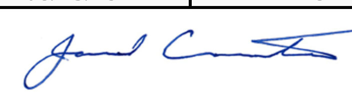
PLASTICITY CHART



ADDITIONAL DATA

% gravel = 1.0%
% sand = 12.2%
% silt and clay = 86.7%
% silt = n/a
% clay = n/a
moisture content = 36.0%

DATE TESTED	TESTED BY
08/23/19	KMS



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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	S19-816
		REPORT DATE: 08/26/19	FIELD ID: SB2.9
		DATE SAMPLED: 08/13/19	SAMPLED BY: JFM/CTB

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Soil Boring SB-02 depth = 35 feet	CL, Lean Clay with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-6(11)

LABORATORY TEST DATA

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

ADDITIONAL DATA

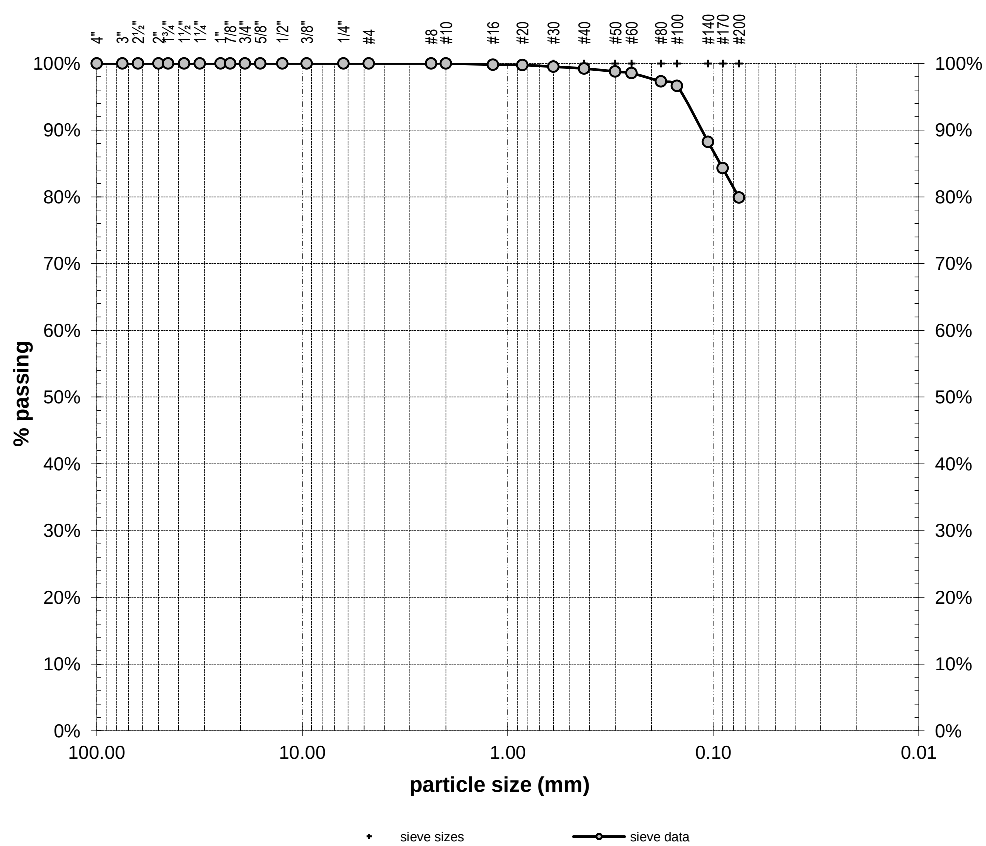
initial dry mass (g) =	165.02		
as-received moisture content =	32.6%	coefficient of curvature, C_C =	n/a
liquid limit =	36	coefficient of uniformity, C_U =	n/a
plastic limit =	22	effective size, $D_{(10)}$ =	n/a
plasticity index =	14	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 20.1%
% silt and clay = 79.9%

	SIEVE SIZE US	mm	PERCENT PASSING SIEVE act.	interp.	SPECS max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		100%		
	#20	0.850	100%			
	#30	0.600		99%		
	#40	0.425	99%			
	#50	0.300		99%		
	#60	0.250	99%			
	#80	0.180		97%		
	#100	0.150	97%			
	#140	0.106		88%		
	#170	0.090		84%		
	#200	0.075	80%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Minit Management Commercial Development 2814 NW 319th Street Ridgefield, Washington	Minit Management, LLC P.O. Box 5889 Vancouver, Washington 98668	19210	S19-816
		REPORT DATE: 08/26/19	FIELD ID: SB2.9
		DATE SAMPLED: 08/13/19	SAMPLED BY: JFM/CTB

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Soil Boring SB-02 depth = 35 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

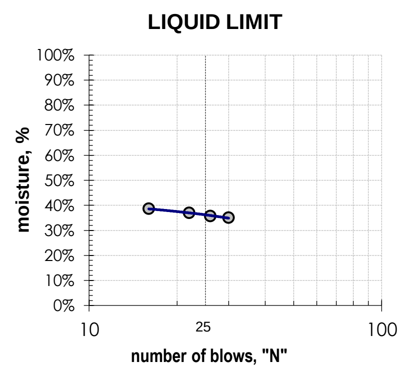
ATTERBERG LIMITS

liquid limit =	36
plastic limit =	22
plasticity index =	14

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	33.87	33.52	32.68	33.47
dry soil + pan weight, g =	30.46	30.16	29.39	29.95
pan weight, g =	20.73	20.76	20.48	20.85
N (blows) =	30	26	22	16
moisture, % =	35.1 %	35.7 %	36.9 %	38.7 %

LIQUID LIMIT



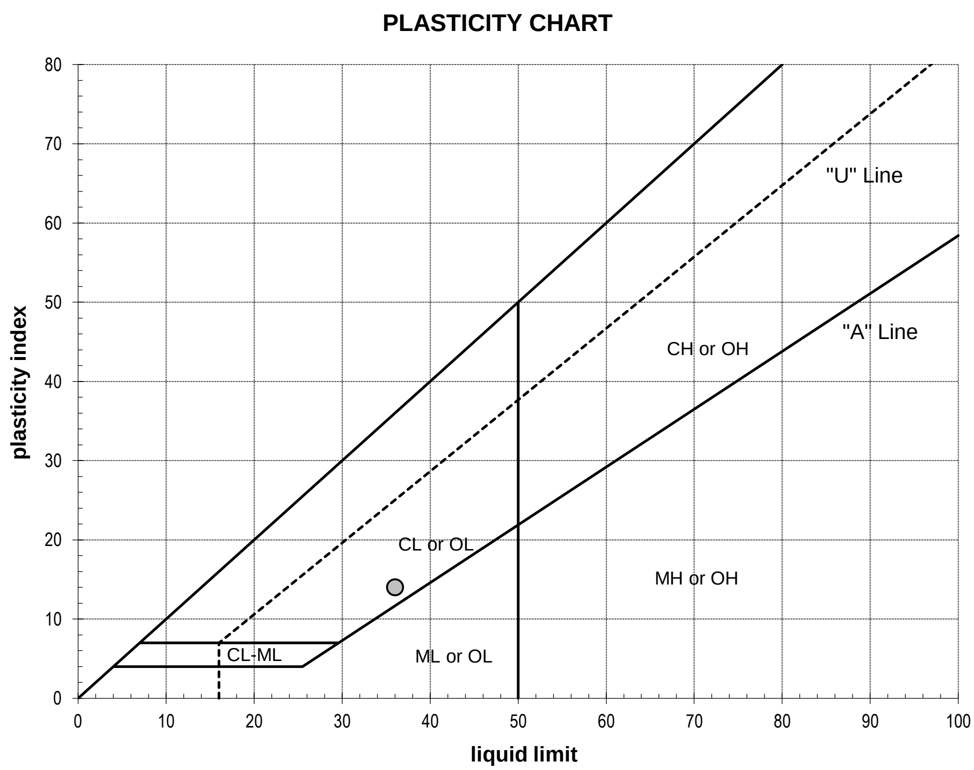
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.64	27.45		
dry soil + pan weight, g =	26.40	26.24		
pan weight, g =	20.80	20.60		
moisture, % =	22.1 %	21.5 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	20.1%
% silt and clay =	79.9%
% silt =	n/a
% clay =	n/a
moisture content =	32.6%

DATE TESTED	TESTED BY
08/23/19	KMS

COLUMBIA WEST ENGINEERING, INC. authorized signature

APPENDIX B
TEST PIT AND SOIL BORING EXPLORATION LOGS

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SOIL BORING LOG

PROJECT NAME	CLIENT		PROJECT NO.	BORING NO.
Minit Management Commercial Dev.	Minit Management, LLC		19210	SB-1
PROJECT LOCATION: Ridgefield, Washington	DRILLING CONTRACTOR: Dan Fischer Excavating	DRILL RIG: Trailer Mount	TECHNICIAN: CTB	PAGE NO.: 1 of 1
BORING LOCATION: See Figure 2	DRILLING METHOD: Solid Stem	SAMPLING METHOD: SPT	START DATE: 08/14/19	START TIME: 0924
REMARKS: none	APPROX. SURFACE ELEVATION: 252 ft amsl	GROUNDWATER DEPTH ON 08-14-19: See Text	FINISH DATE: 08/14/19	FINISH TIME: 1200

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
0	252						FILL. Dark gray to black gravel mixed with topsoil and asphalt grindings, moist, medium dense [Soil Type 1].					
		SPT SB1.1	27									
5	247	SPT SB1.2	23									
		SPT SB1.3	8									
10	242	SPT SB1.4	9				Brown, tan, and reddish-brown lean CLAY with sand, moist, stiff [Soil Type 2]. Interbedded silt lenses and layers throughout.		22.8			
15	237	SPT SB1.5	11				Sand content decreases with depth.		30.0			
20	232	SPT SB1.6	6	▼			Perched groundwater observed at 20 feet. Becomes wet and medium stiff.		36.0	86.7	42	21
25	227	SPT SB1.7	19				Becomes moist and very stiff at 25 feet.		27.7			
30	222	SPT SB1.8	34	CL	A-7-6(19)		Becomes hard at 30 feet.		30.8			
35	217	SPT SB1.9	10				Becomes stiff and very moist at 35 feet.		40.0			
40	212	SPT SB1.10	13						25.8			
45	207						Becomes very stiff at 50 feet.					
50	202	SPT	17				Soil boring terminated at 50 feet bgs. Perched groundwater observed at 20 feet.		35.4			

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SOIL BORING LOG

PROJECT NAME	CLIENT		PROJECT NO.	BORING NO.
Minit Management Commercial Dev.	Minit Management, LLC		19210	SB-2
PROJECT LOCATION	**DRILLING CONTRACTOR**	**DRILL RIG**	**TECHNICIAN**	**PAGE NO.**
Ridgefield, Washington	Dan Fischer Excavating	Trailer Mount	CTB	1 of 1
BORING LOCATION	**DRILLING METHOD**	**SAMPLING METHOD**	**START DATE**	**START TIME**
See Figure 2	Solid Stem	SPT	08/14/19	1205
REMARKS	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH ON 08-14-19**	**FINISH DATE**	**FINISH TIME**
none	262 ft amsl	See Text	08/14/19	1430

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
0							Approximately 8 to 10 inches of topsoil and grass.					
							Brown, tan, and reddish-brown lean CLAY with sand, moist to very moist, very stiff [Soil Type 2].					
		SPT SB2.1	24									
5							Interbedded silt lenses and layers throughout.					
		SPT SB2.2	20						25.0			
							Becomes medium stiff to stiff at 7.5 feet.					
		SPT SB2.3	8						29.8			
10	252						Sand content decreases with depth.					
		SPT SB2.4	9						26.2			
15	247						Becomes hard at 15 feet.					
		SPT SB2.5	31						24.7			
20	242											
		SPT SB2.6	35						23.9			
25	237			CL	A-6(11)		Becomes very stiff at 25 feet.					
		SPT SB2.7	18						29.3			
30	232			▼			Perched groundwater layer observed at 30 feet.					
		SPT SB2.8	17						31.7			
35	227						Becomes stiff at 35 feet.					
		SPT SB2.9	11						32.6	79.9	36	14
40	222						Becomes stiff to very stiff at 40 feet.					
		SPT SB2.10	15						38.3			
45	217											
		SPT SB2.11	16						32.9			
							Soil boring terminated at 50 feet bgs.					
50	212	SPT	16				Perched groundwater observed at 30 feet.		37.0			

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Minit Management Commercial Development	Minit Management, LLC		19210	TP-1
PROJECT LOCATION: Ridgefield, Washington	CONTRACTOR: L&S	EQUIPMENT: Excavator	TECHNICIAN: HDG	DATE: 08/13/19
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 254 feet amsl	GROUNDWATER DEPTH: Not Observed	START TIME: 0805	FINISH TIME: 0835

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						FILL. Dark gray to black gravel mixed with topsoil and asphalt grindings [Soil Type 1].					
5											
						Heavy organics from 8 to 10 feet.					
10				CL		Brown lean CLAY with sand, moist, medium stiff [Soil Type 2].					
	TP1.1		A-6(11)				23.4	79.6	34	15	
						Bottom of test pit at 13 feet bgs. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Minit Management Commercial Development	Minit Management, LLC		19210	TP-2
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**TECHNICIAN**	**DATE**
Ridgefield, Washington	L&S	Excavator	HDG	08/13/19
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	257 feet amsl	Not Observed	0845	0910

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						FILL. Concrete chunks mixed with native lean clay with sand [Soil Type 1].					
		Gee Silt Loam	A-6	CL		Brown lean CLAY with sand, moist, medium stiff [Soil Type 2].					
5											
						Organic odor throughout soil.					
10											
						Bottom of test pit at 14 feet bgs. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Minit Management Commercial Development	Minit Management, LLC		19210	TP-3
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Ridgefield, Washington	L&S	Excavator	HDG	08/13/19
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	263 feet amsl	Not Observed	1030	1100

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 8 to 10 inches of topsoil and grass.					
		Gee Silt Loam		CL		Brown lean CLAY with sand, light gray mottling, moist, medium stiff [Soil Type 2].					
	TP3.1		A-6(10)				24.9	70.0	38	16	IT-3.1 2.0-ft bgs k < 0.1 in/hr
5											
10											
						Bottom of test pit at 12 feet bgs. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Minit Management Commercial Development	Minit Management, LLC		19210	TP-4
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Ridgefield, Washington	L&S	Excavator	HDG	08/13/19
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	266 feet amsl	Not Observed	1000	1020

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 10 to 12 inches of topsoil and grass.					
						FILL. Brown sub-rounded to rounded gravels and cobbles consistent with a septic drain field [Soil Type 1].					
						Bottom of test pit at 3 feet bgs. Groundwater not observed.					
5											
10											
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Minit Management Commercial Development	Minit Management, LLC		19210	TP-5
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Ridgefield, Washington	L&S	Excavator	HDG	08/13/19
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	264 feet amsl	Not Observed	0920	0950

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 4 to 6 inches of topsoil and grass.					
						FILL. Brown to gray subrounded to rounded gravel, moist, medium dense [Soil Type 1].					
		Gee Silt Loam	A-6	CL		Brown to dark gray lean CLAY with sand, moist, medium stiff [Soil Type 2].					
5											
						Organic odor, sticks, and roots from 8.5 to 13 feet.					
10											
						Bottom of test pit at 13 feet bgs. Groundwater not observed.					
15											

APPENDIX C
CPT RESULTS REPORT

PRESENTATION OF SITE INVESTIGATION RESULTS

Minit Management Commercial Development

Prepared for:

Columbia West Engineering

ConeTec Job No: 19-59031

Project Start Date: 09-AUG-2019
Project End Date: 09-AUG-2019
Report Date: 19-AUG-2019

Prepared by:

ConeTec Inc.
1508 O st SW – Unit 104
Auburn, WA 98001

Tel: (253) 397-4861

Email: ConeTecWA@conetec.com
www.conetec.com
www.conetecdataservices.com

CONETEC

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for Columbia West Engineering at 2814 NW 319th Street, Ridgefield, WA 98642. The program consisted of cone penetration tests (CPT) and seismic cone penetration tests (SCPT).

Project Information

Project	
Client	Columbia West Engineering
Project	Minit Management Commercial Development
ConeTec project number	19-59031

A map from Google Earth including the CPT test locations is presented below.

Rig Description	Deployment System	Test Type
C20 – 25Ton Truck Rig	Integrated Ramset	SCPT/CPT

Coordinates		
Test Type	Collection Method	EPSG Number
SCPT/CPT	Consumer Grade GPS	4326

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Advanced plots with Ic, Su(Nkt), Phi and N(60)Ic, Seismic Vs plots as well as Soil Behavior Type (SBT) Scatter plots have been included in the data release package.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
595:T1500F15U500	595	15	225	1500	15	500
Cone 595 was used for all CPT soundings						

Interpretation Tables	
Additional information	The Normalized Soil Behavior Type Chart based on Q_{tn} (SBT Qtn) (Robertson 2009) was used to classify the soil for this project. A detailed set of calculated CPT interpretations have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip (q_t), sleeve friction (f_s) and pore pressure (u_2) at each data point. Effective stresses are calculated based on unit weights that have been assigned to the individual soil behavior type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behavior Type Chart (Robertson 2009). Calculations for both drained and undrained parameters have been included for materials that classified as silts mixtures (zone 4).

Limitations

This report has been prepared for the exclusive use of Columbia West Engineering (Client) for the project titled "Minit Management Commercial Development ". The report's contents may not be relied upon by any other party without the express written permission of ConeTec Inc. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first Appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meet or exceed those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

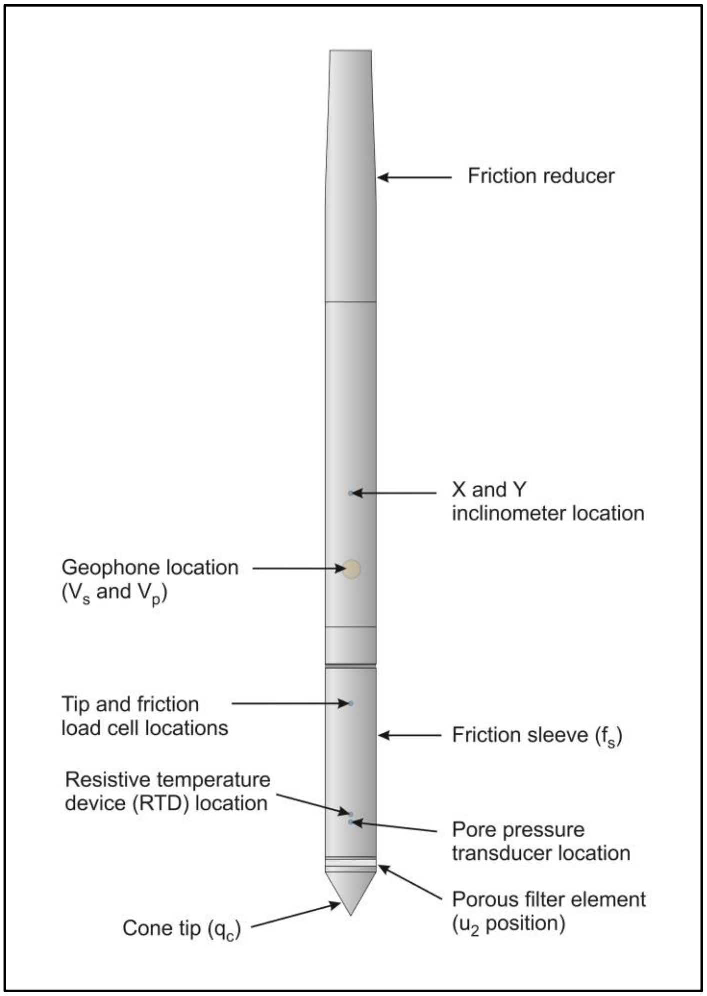

Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerin or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerin under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave (Vp) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

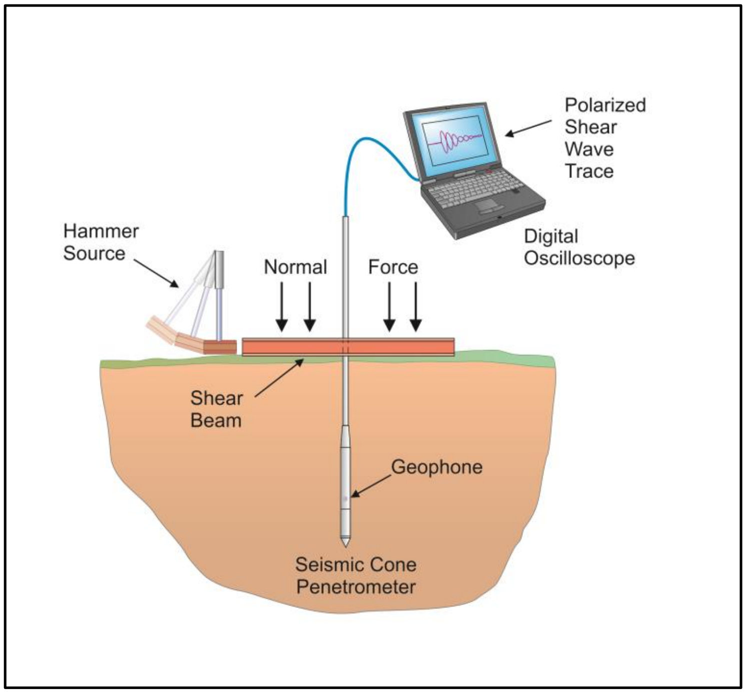

Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

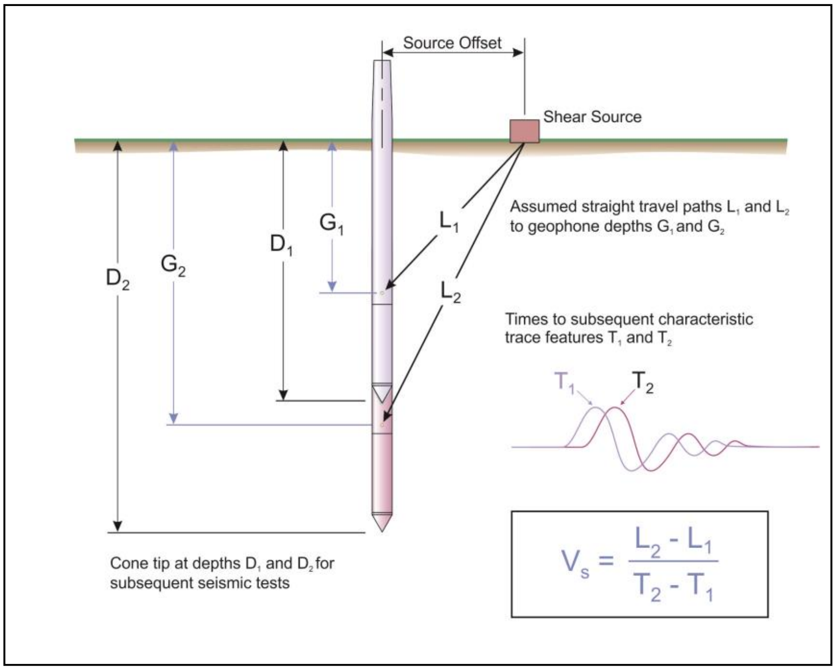

Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 100 feet (30 meters) ($\bar{v}_s$) has been calculated and provided for all applicable soundings using the following equation presented in ASCE, 2010.

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)
d_i = the thickness of any layer between 0 and 100 ft (30 m)
v_{si} = the shear wave velocity in ft/s (m/s)
$\sum_{i=1}^{n} d_i$ = 100 ft (30 m)

Average shear wave velocity, $\bar{v}_s$ is also referenced to V_{s100} or V_{s30}.

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

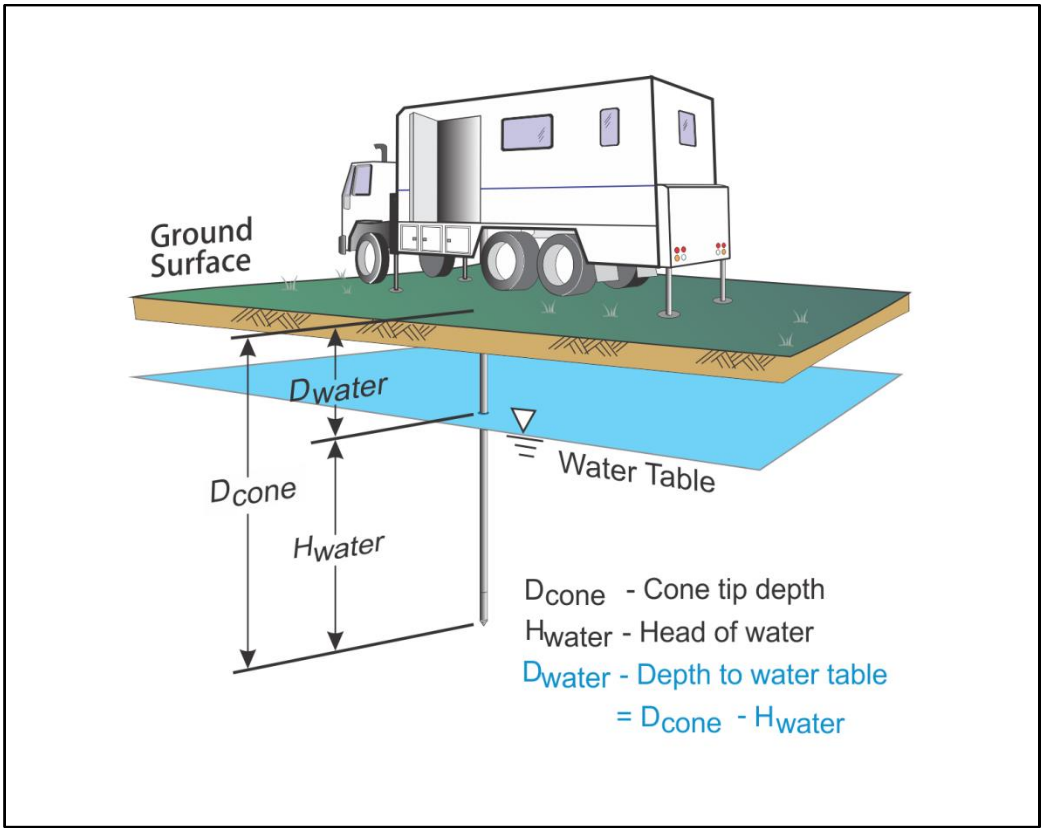

Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

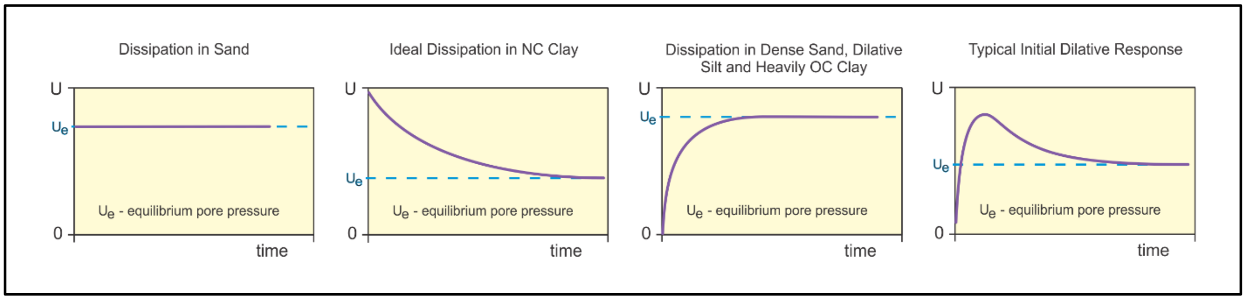

Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T* is the dimensionless time factor (Table Time Factor)
a is the radius of the cone
I_r is the rigidity index
t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

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Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, "Estimating coefficient of consolidation from piezocone tests", Canadian Geotechnical Journal, 29(4): 551-557.

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Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Cone Penetration Test Advanced Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Wave Traces
- Cone Penetration Test Soil Behavior Type Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and

Standard Cone Penetration Test Plots

Job No: 19-59031
Client: Columbia West Engineering
Project: Minit Management Commercial Development
Start Date: 09-Aug-2019
End Date: 09-Aug-2019

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface (ft)	Final Depth (ft)	Latitude[2] (Deg)	Longitude[2] (Deg)
SCPT19-01	19-59031_SP01	09-Aug-2019	595:T1500F15U500		62.3	45.85370	-122.70083
CPT19-02	19-59031_CP02	09-Aug-2019	595:T1500F15U500		62.3	45.85257	-122.70112
Totals	2 soundings				124.7		

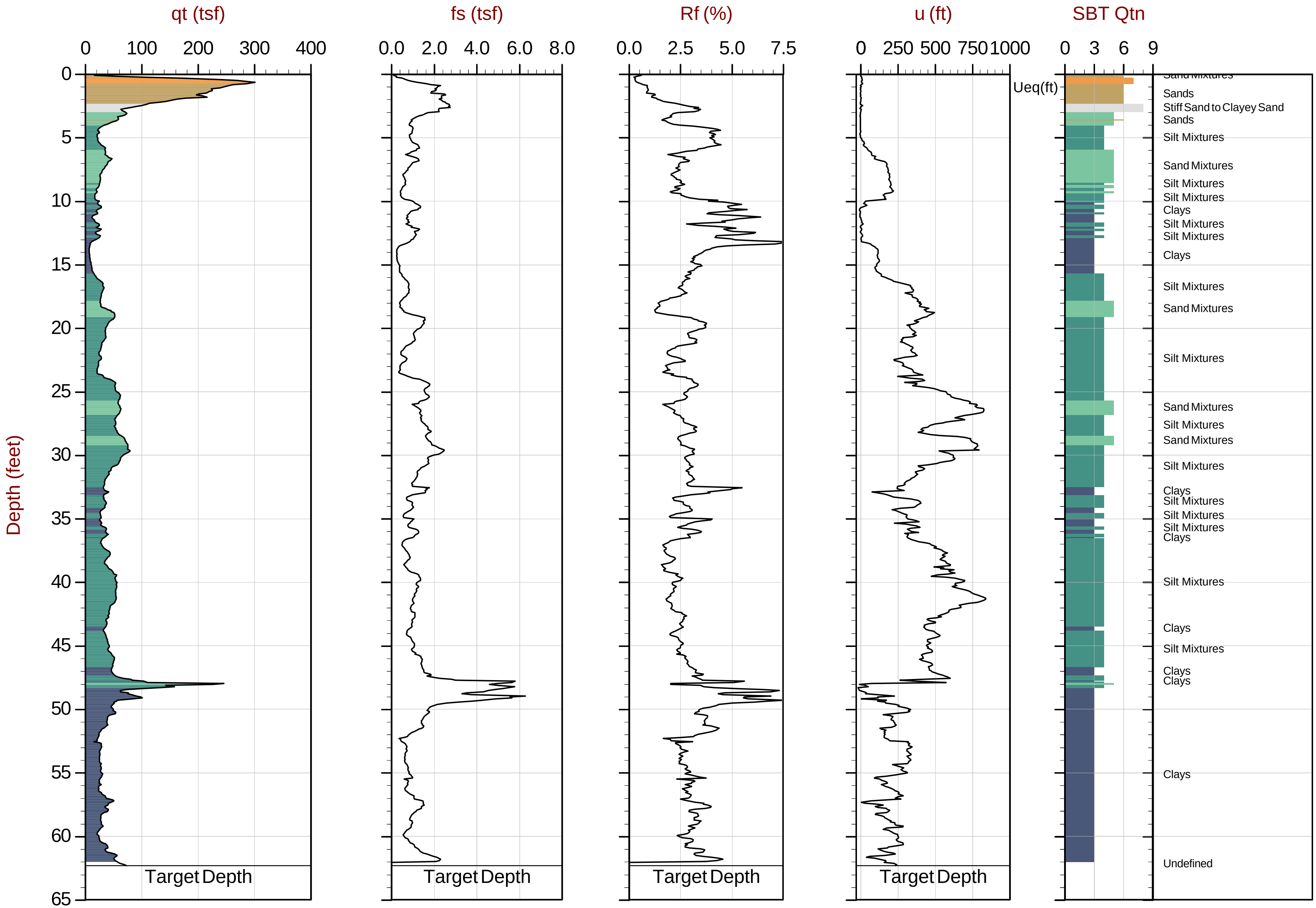
1. Phreatic surface assumed to be below final testing depth
2. Coordinates were collected using a handheld GPS - WGS 84 Lat/Long

CONETEC

Columbia West Eng.

Job No: 19-59031
Date: 2019-08-09 10:54
Site: Minit Managemen

Sounding: SCPT19-01
Cone: 595:T1500F15U500



Max Depth: 19.000 m / 62.34 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-59031_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.85370 Long: -122.70083

△ Dissipation with estimated Ueq value △ Dissipation, equilibrium not achieved ● Equilibrium Pore Pressure (Ueq) — Hydrostatic Line

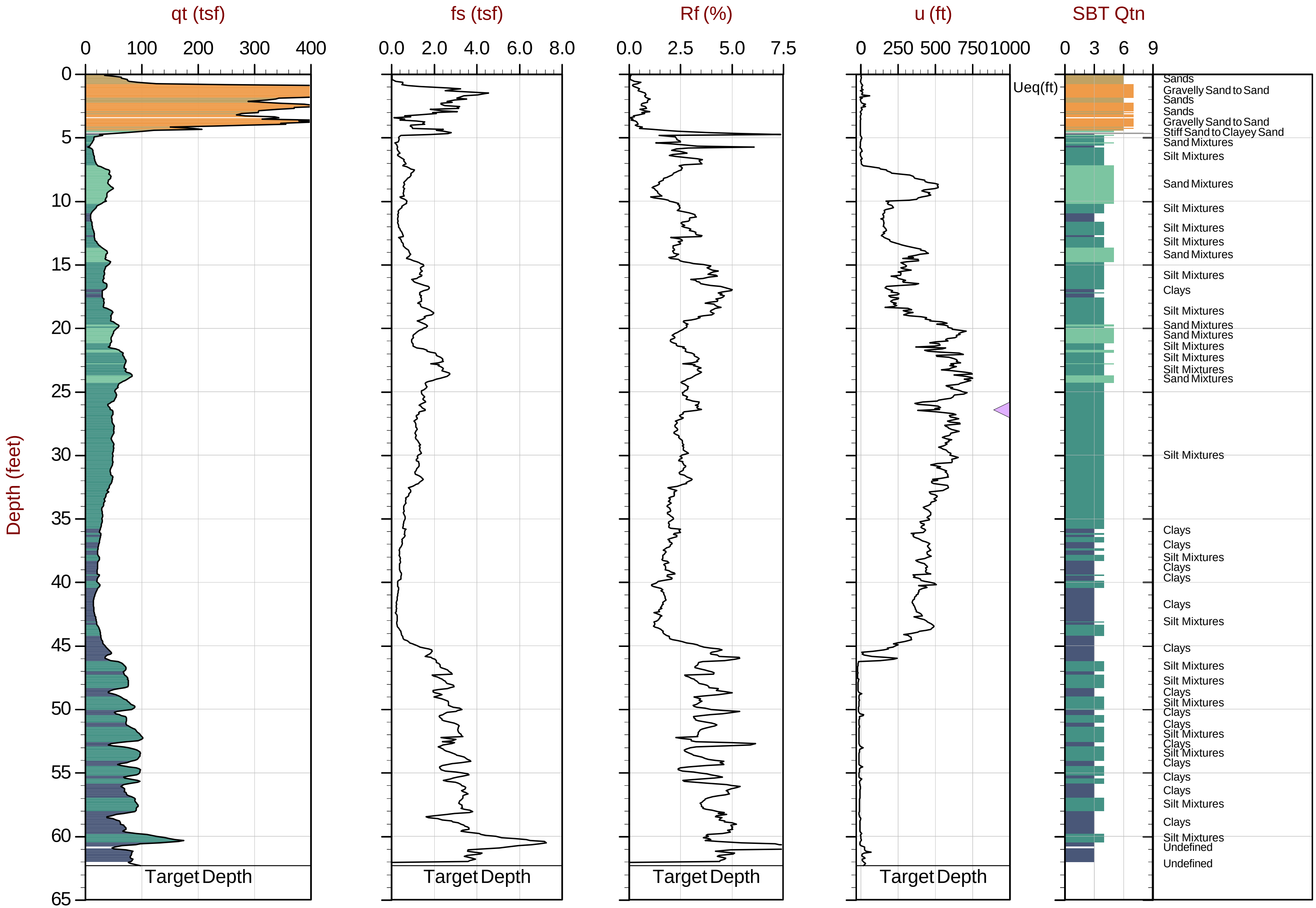
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Columbia West Eng.

Job No: 19-59031
Date: 2019-08-09 12:50
Site: Minit Managemen

Sounding: CPT19-02
Cone: 595:T1500F15U500



Max Depth: 19.000 m / 62.34 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-59031_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.85257 Long: -122.70112

△ Dissipation with estimated Ueq value △ Dissipation, equilibrium not achieved ○ Equilibrium Pore Pressure (Ueq) — Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

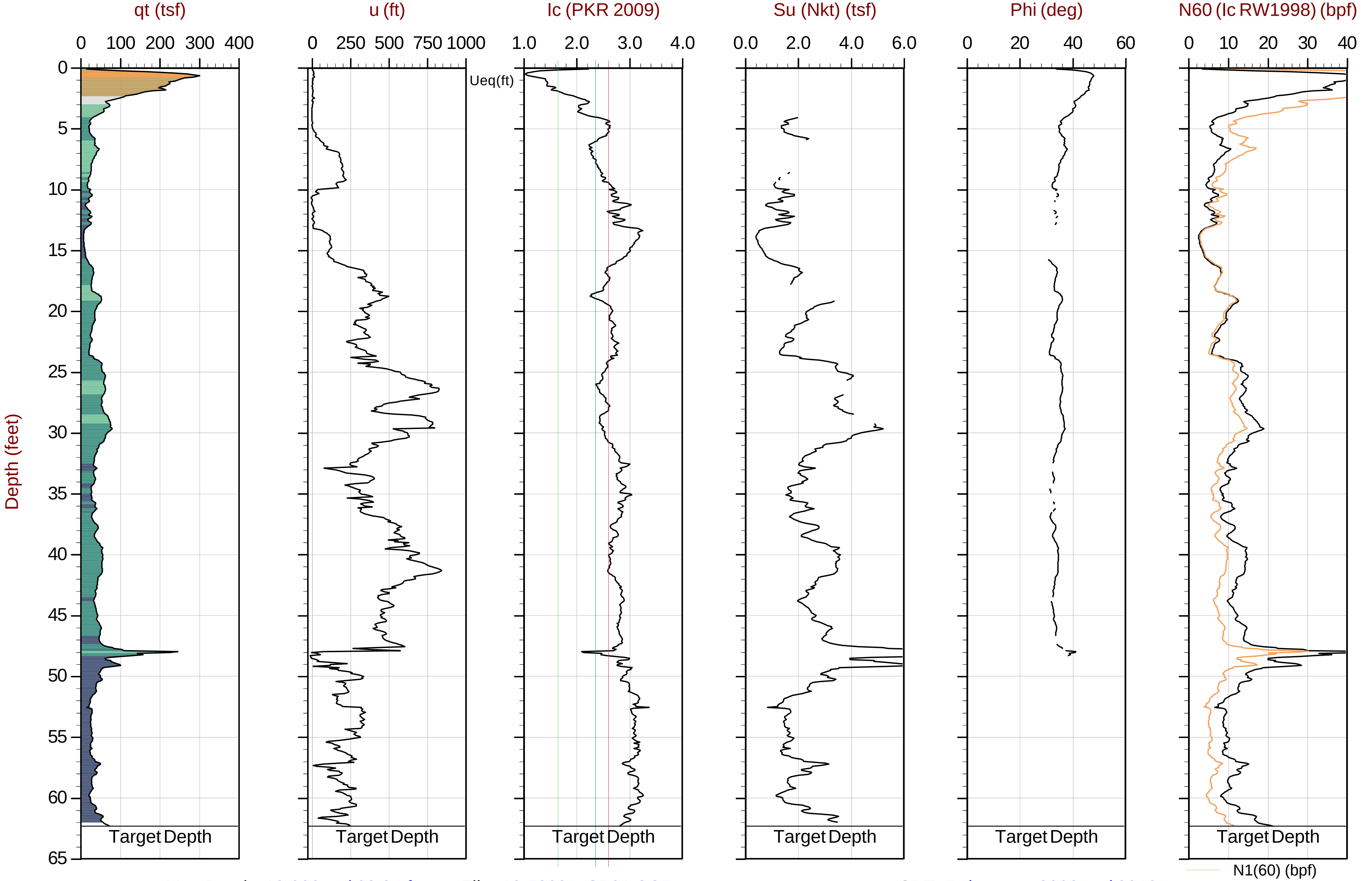
Cone Penetration Test Advanced Plots

CONETEC

Columbia West Eng.

Job No: 19-59031
Date: 2019-08-09 10:54
Site: Minit Managemen

Sounding: SCPT19-01
Cone: 595:T1500F15U500



Max Depth: 19.000 m / 62.34 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-59031_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.85370 Long: -122.70083

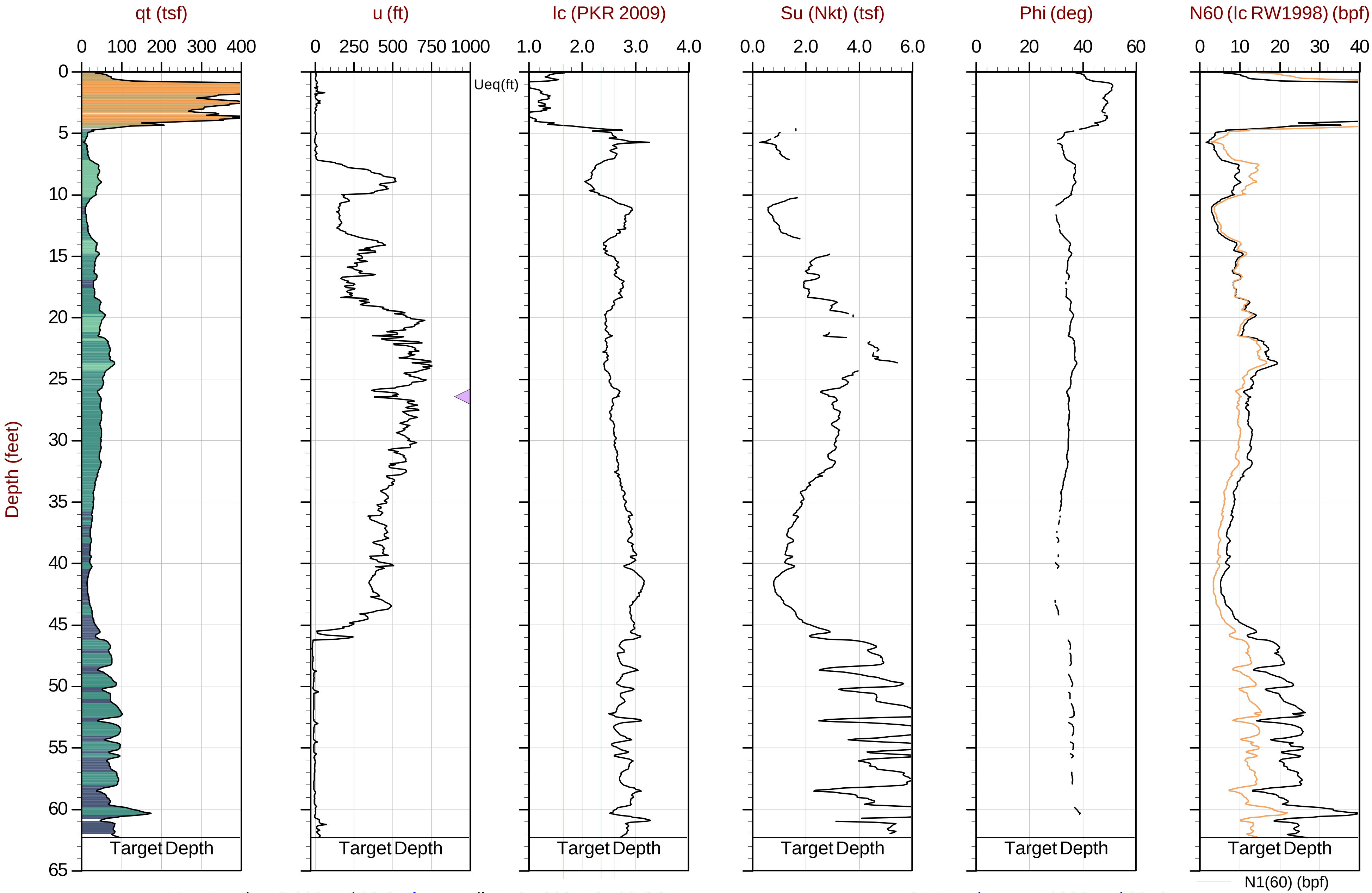
△ Dissipation with estimated Ueq value △ Dissipation, equilibrium not achieved ○ Equilibrium Pore Pressure (Ueq) — Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Columbia West Eng.

Job No: 19-59031
Date: 2019-08-09 12:50
Site: Minit Managemen

Sounding: CPT19-02
Cone: 595:T1500F15U500



Max Depth: 19.000 m / 62.34 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-59031_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.85257 Long: -122.70112

△ Dissipation with estimated Ueq value △ Dissipation, equilibrium not achieved ○ Equilibrium Pore Pressure (Ueq) — Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

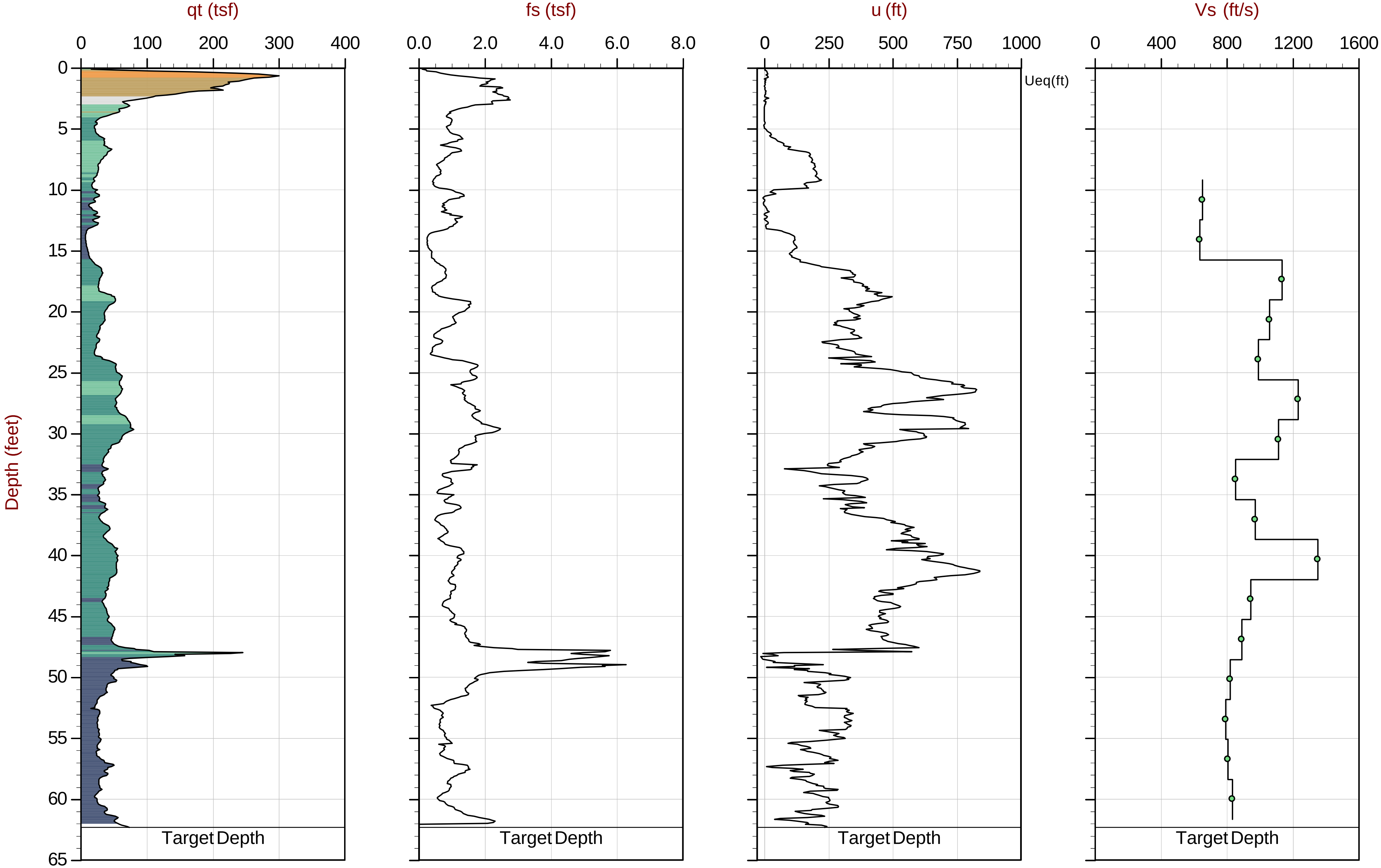
Seismic Cone Penetration Test Plots

CONETEC

Columbia West Eng.

Job No: 19-59031
Date: 2019-08-09 10:54
Site: Minit Managemen

Sounding: SCPT19-01
Cone: 595:T1500F15U500



Max Depth: 19.000 m / 62.34 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-59031_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.85370 Long: -122.70083

△ Dissipation with estimated Ueq value △ Dissipation, equilibrium not achieved ○ Equilibrium Pore Pressure (Ueq) — Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Tabular Results

CONETEC

Job No: 19-59031
Client: Columbia West Engineering
Project: Minit Management Commercial Development
Sounding ID: SCPT19-01
Date: 09-Aug-2019

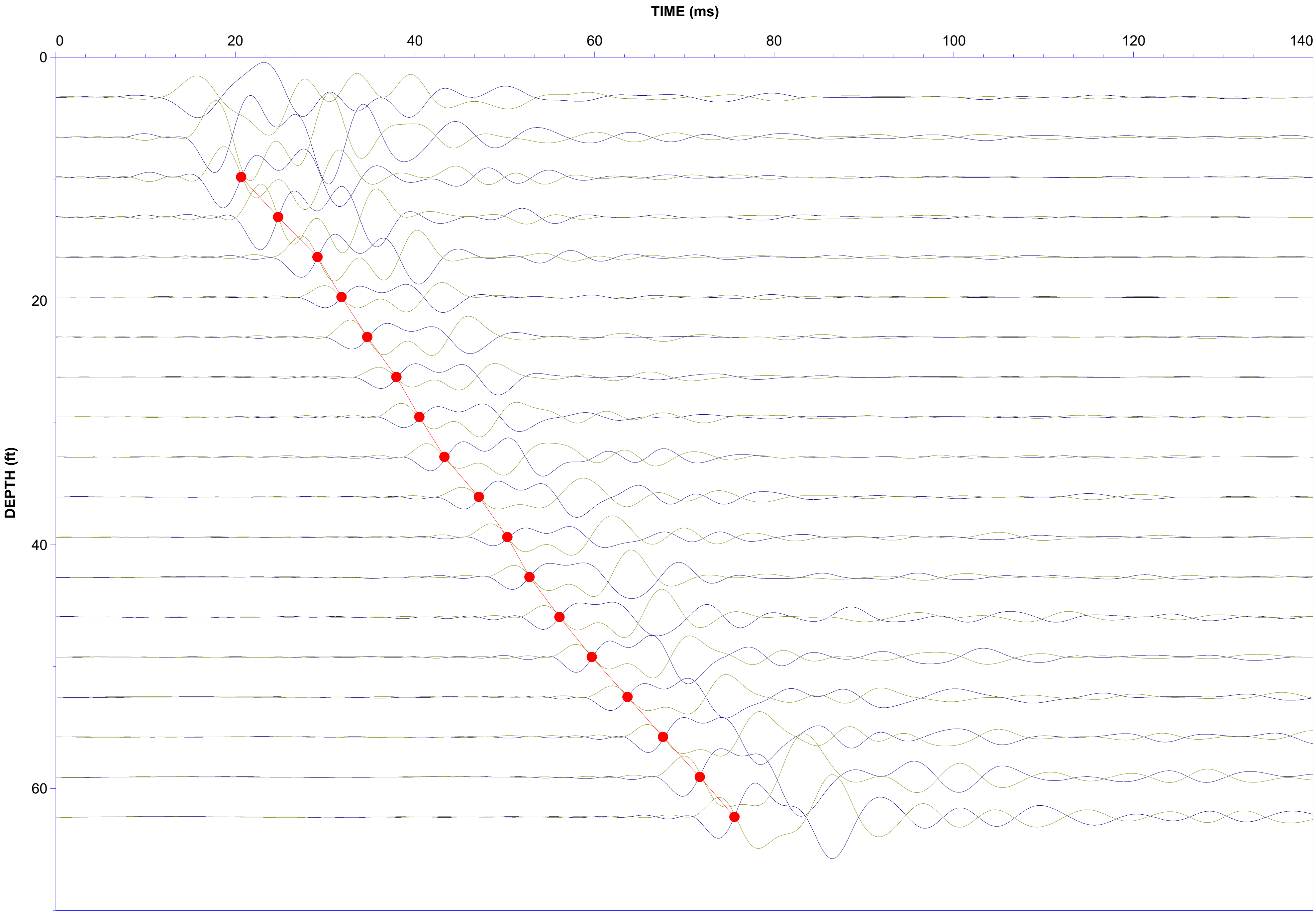
Seismic Source: Beam
Source Offset (ft): 8.17
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
9.84	9.19	12.29			
13.12	12.47	14.90	2.61	4.01	651
16.40	15.75	17.74	2.84	4.45	637
19.69	19.03	20.71	2.97	2.61	1135
22.97	22.31	23.76	3.05	2.88	1058
26.25	25.59	26.86	3.10	3.14	990
29.53	28.87	30.00	3.14	2.55	1232
32.81	32.15	33.17	3.17	2.85	1114
36.09	35.43	36.36	3.19	3.74	853
39.37	38.71	39.57	3.20	3.30	972
42.65	41.99	42.78	3.22	2.38	1352
45.93	45.28	46.01	3.22	3.41	946
49.21	48.56	49.24	3.23	3.63	891
52.49	51.84	52.48	3.24	3.95	820
55.77	55.12	55.72	3.24	4.09	794
59.06	58.40	58.97	3.25	4.03	806
62.34	61.68	62.22	3.25	3.89	835

Seismic Cone Penetration Filtered Wave Traces

Job No: 19-59031 Client: Columbia West Engineering Project Title: Minit Management Operator: JMAR Sounding ID: SCPT19-01
Site: Minit Management Date: Aug 18, 2019



Cone Penetration Test Soil Behavior Type Plots

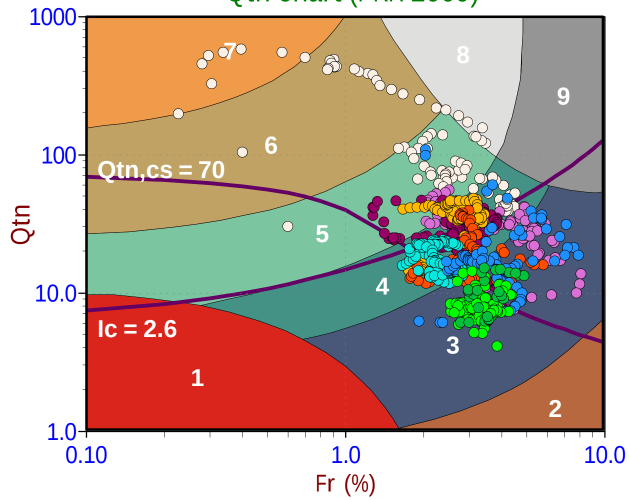
CONETEC

Columbia West Eng.

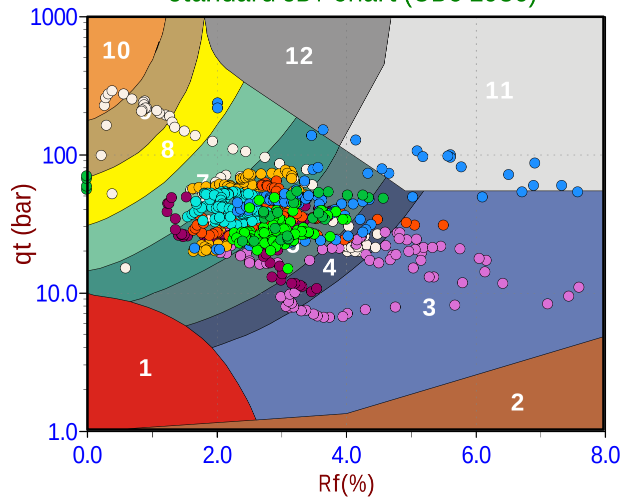
Job No: 19-59031
Date: 2019-08-09 10:54
Site: Minit Managemen

Sounding: SCPT19-01
Cone: 595:T1500F15U500

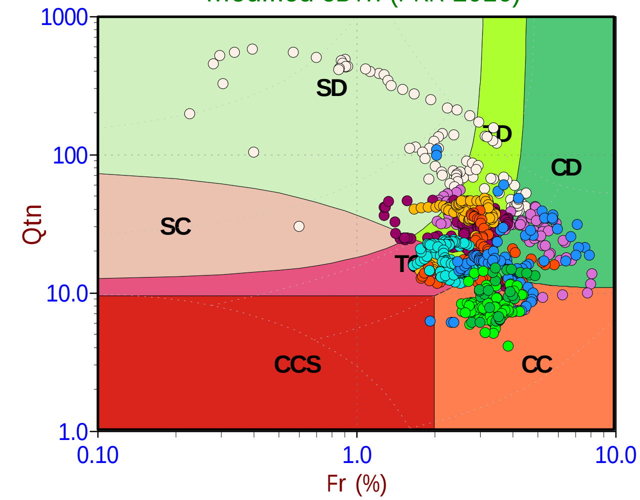
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend

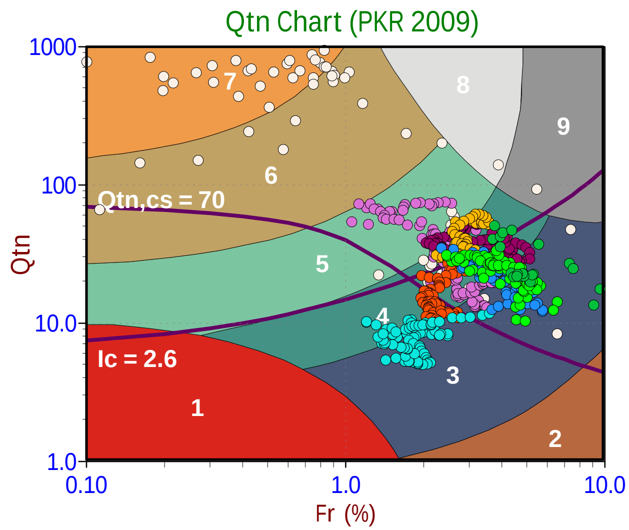
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

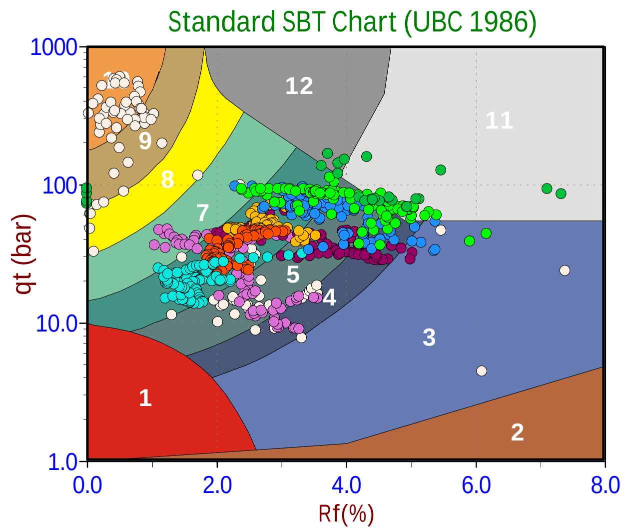
ConeTec

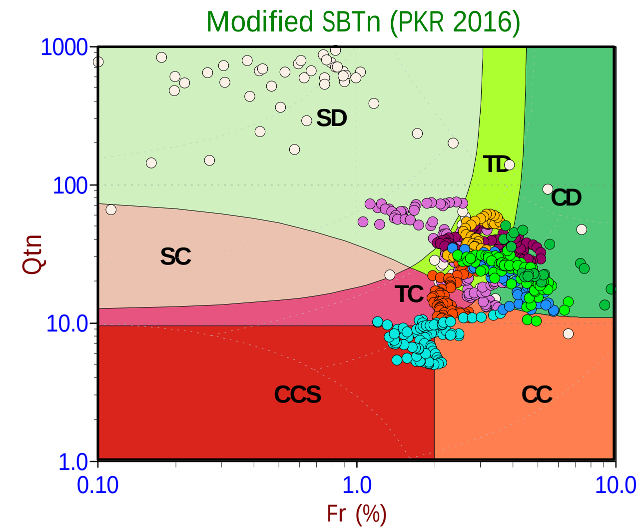
Columbia West Eng.

Job No: 19-59031
Date: 2019-08-09 12:50
Site: Minit Managemen

Sounding: CPT19-02
Cone: 595:T1500F15U500







Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Pore Pressure Dissipation Summary and

Pore Pressure Dissipation Plots

Job No: 19-59031
Client: Columbia West Engineering
Project: Minit Management Commercial Development
Start Date: 9-Aug-19
End Date: 9-Aug-19

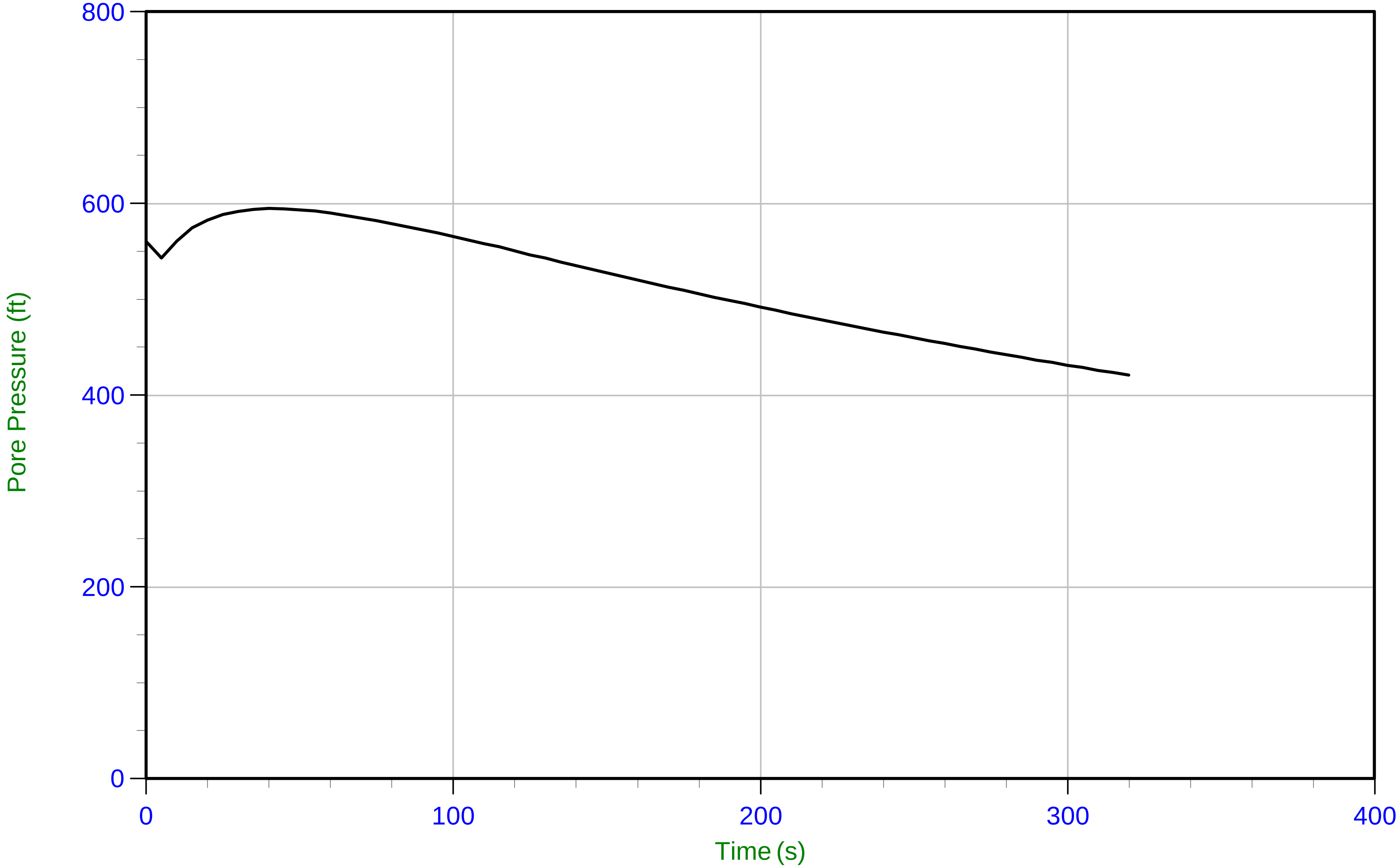
CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
CPT19-02	19-59031_CP02.PPD	15.0	320	26.4		
Totals			5.3	(min)		

Columbia West Eng.

Job No: 19-59031
Date: 08/09/2019 12:50
Site: Minit Managemen

Sounding: CPT19-02
Cone: 595:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-59031_CP02.PPD
Depth: 8.050 m / 26.410 ft
Duration: 320.0 s

U Min: 420.9 ft
U Max: 594.7 ft

APPENDIX D
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.

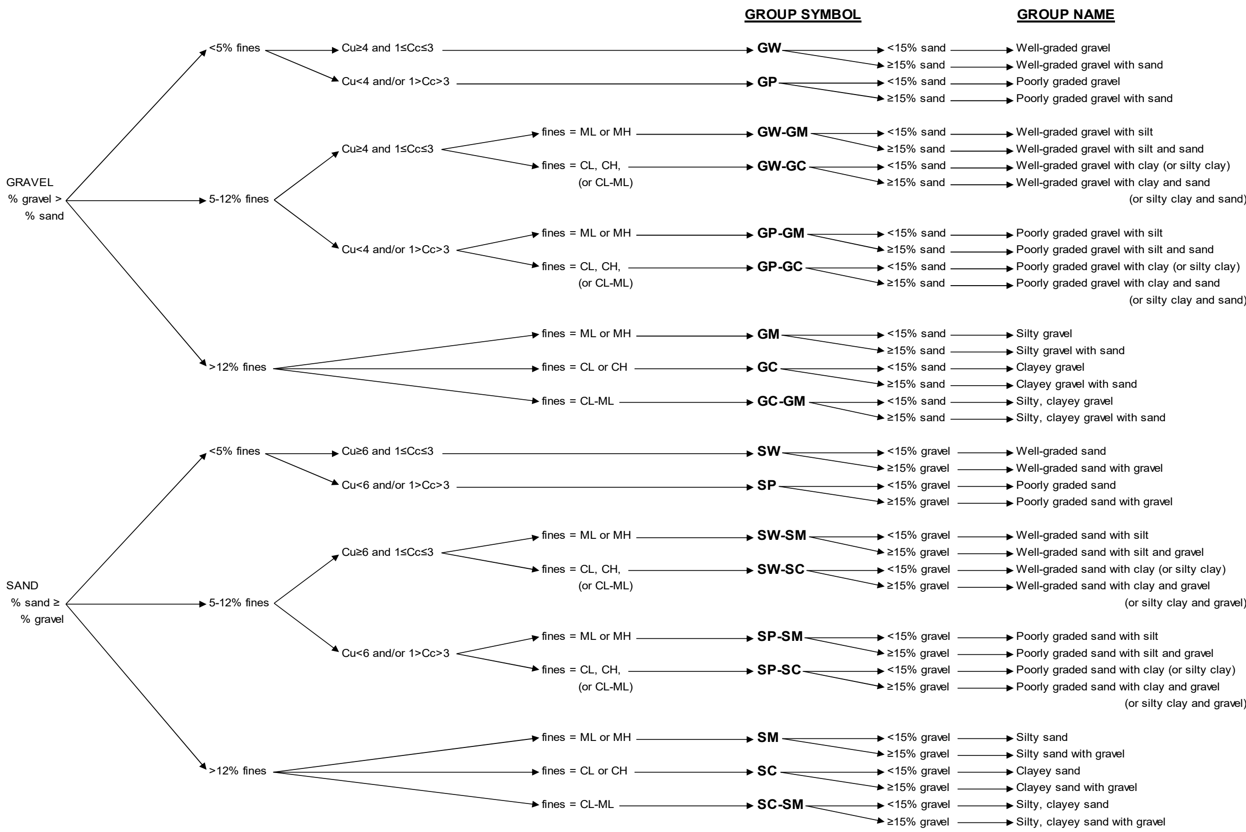
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

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			

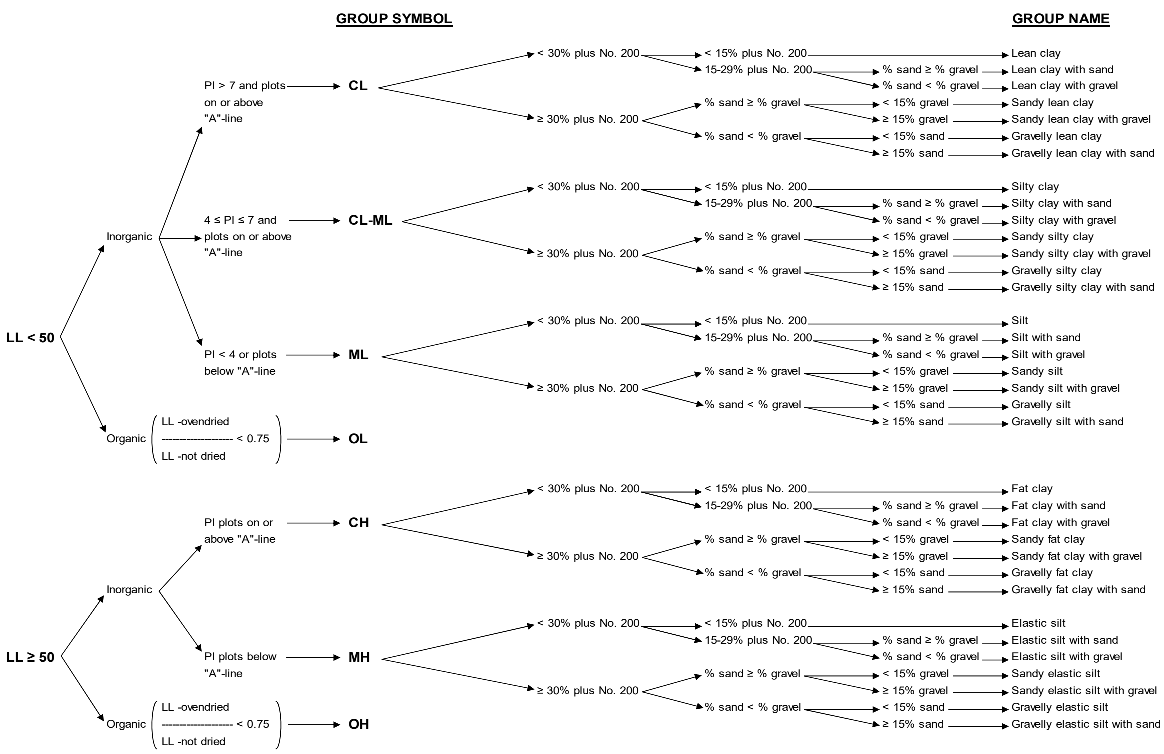
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

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 E
PHOTO LOG

MINIT MANAGEMENT COMMERCIAL DEVELOPMENT
RIDGEFIELD, WASHINGTON
PHOTO LOG

Test Pit Exploration Activity, TP-1

Test Pit Profile, TP-1

MINIT MANAGEMENT COMMERCIAL DEVELOPMENT
RIDGEFIELD, WASHINGTON
PHOTO LOG

Site View From TP-5, Facing Southeast

Test Pit Profile, TP-5

APPENDIX F
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

Date: September 4, 2019
Project: Minit Management Commercial Development
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.

Report Ownership

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Consultant Responsibility

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