Geotechnical Site Investigation

Sunrise Terrace

La Center, Washington

June 26, 2015

Geotechnical ■ Environmental ■ Special Inspections
Columbia West Engineering, Inc

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




www.columbiawestengineering.com

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GEOTECHNICAL SITE INVESTIGATION
SUNRISE TERRACE
LA CENTER, WASHINGTON

Prepared For:	**RK Land Development, LLC** **c/o Mr. Ed Greer** **1520 SW Eaton Blvd.** **Battle Ground, Washington 98604**
Site Location:	**1908 NE Lockwood Creek Road** **Parcel Nos. 209062000, 986027189, 986027188, 209047000** **La Center, Washington**
Prepared By:	**Columbia West Engineering, Inc.** **11917 NE 95th Street** **Vancouver, Washington 98682** **Phone: 360-823-2900** **Fax: 360-823-2901**
Date Prepared:	**June 26, 2015**

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GEOTECHNICAL SITE INVESTIGATION
SUNRISE TERRACE
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1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by RK Land Development, LLC to conduct a geotechnical site investigation for the proposed Sunrise Terrace residential development located in La Center, Washington. The purpose of the investigation was to observe and assess subsurface soil conditions at specific locations and provide geotechnical engineering analyses, planning, and design recommendations for proposed development. The specific scope of services was outlined in a proposal contract dated April 29, 2015. This report summarizes the investigation and provides field assessment documentation and laboratory analytical test reports. This report is subject to the limitations expressed in Section 6.0, *Conclusion and Limitations*, and Appendix E.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located at 1908 NE Lockwood Creek Road in La Center, Washington. The site consists of tax parcels 209047000, 986027188, 986027189, and a portion of parcel 209062000 contributing to a proposed development area of approximately 33.7 acres. The regulatory jurisdictional agency is the City of La Center, Washington. The approximate latitude and longitude are N 45° 51’ 50” and W 122° 38’ 58”, and the legal description is a portion of the NW ¼ of Section 02, T4N, R1E, Willamette Meridian.

1.2 Proposed Development

Review of preliminary site plans indicates that proposed development will consist of a single-family residential subdivision that will include approximately 120 building lots, asphalt concrete roadways, underground utilities, and stormwater management facilities. Columbia West understands that cut and fill will likely be proposed at the property. This report is based upon proposed development as described above and may not be applicable if modified.

2.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

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

According to the *Geologic Map of the Ridgefield Quadrangle, Clark and Cowlitz Counties, Washington (*Russell C. Evarts, USGS Geological Survey, 2004*)*, near-surface soils are expected to consist of Pleistocene aged, unconsolidated, rhythmically bedded periglacial deposits of sand, silt, and clay derived from catastrophic outburst floods of Glacial Lake Missoula (Qfs) transitioning to Pleistocene to Pliocene, semi-consolidated, pebble to cobble sedimentary conglomerate (QTc) in the northwest portion of the site. Previously published geologic mapping has identified the conglomerate as the Troutdale Formation.

The *Web Soil Survey (*United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2013 Website*)* identifies surface soils primarily as Gee silt loam with smaller areas mapped as Hillsboro silt loam and Odne silt loam. Although soil conditions may vary from the broad USDA descriptions, these soils generally consist of fine-textured sand, silt, and clay with very low to 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 upon slope grade. According to Clark County GIS, Odne soils mapped at the site are classified as hydric.

3.0 REGIONAL SEISMOLOGY

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

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

<u>Portland Hills Fault Zone</u>

The Portland Hills Fault Zone consists of several northwest-trending faults located along the northeastern margin of the Tualatin Mountains, also known as the Portland Hills, and the southwest margin of the Portland Basin. The fault zone is approximately 25 to 30 miles in length and is located approximately 17 miles southwest of the site. According to *Seismic Design Mapping, State of Oregon (Geomatrix Consultants, 1995),* there is no definitive consensus among geologists as to the zone fault type. Several alternate interpretations have been suggested.

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

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

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

Located approximately 40 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 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 21 miles southeast of the site, and form part of the northeastern margin of the Portland basin. According to *Geology and Groundwater Conditions of Clark County Washington (USGS Water Supply Paper 1600, Mundorff, 1964)* and the *Geologic Map of the Lake Oswego Quadrangle (Oregon DOGAMI Series GMS-59, 1989)*, the Lacamas Lake fault zone consists of shear contact between the Troutdale Formation and underlying Oligocene andesite-basalt bedrock. Secondary shear contact associated with the fault zone may have produced a series of prominent northwest-southeast geomorphic lineaments in proximity to the site.

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

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

Cascadia Subduction Zone

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

4.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance, twelve test pits (TP-1 through TP-12), and one infiltration test (IT-1) was conducted at the site on June 10, 2015. Test pits were excavated with a track-mounted excavator. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications. Disturbed soil samples were collected from relevant soil horizons and submitted for laboratory analysis. Analytical laboratory test results are presented in Appendix A. Exploration locations are indicated on Figure 2. Subsurface exploration logs are presented in Appendix B. Soil descriptions and classification information are provided in Appendix C. A photo log is presented in Appendix D.

4.1 Surface Investigation and Site Description

The approximate 33.7-acre subject site consists of four tax parcels located at 1908 NE Lockwood Creek Road in La Center, Washington. The subject site is generally surrounded by rural farmland and may be accessed through frontage along NE 339th Street to the north, NE 24th Avenue to the east, and NE Lockwood Creek Road to the south. Site reconnaissance and review of topographic mapping indicates gently rolling to moderately southwest sloping terrain with site elevations ranging from 294 feet above mean sea level (amsl) adjacent to 339th Street to 152 ft amsl at Lockwood Creek Road. Slopes of 5 to 10 percent characterize most of the site with localized steeper grades observed in the east-central area. The site appears to be currently utilized for agricultural purposes as evidenced by perimeter fencing, grassy pastures, and recently tilled farmland. Towards the southern end of the site, an existing residential structure with two outbuildings joins Lockwood Creek Road via a gravel driveway. With the exception of recently tilled areas, open areas of the site were primarily covered with grass. Vegetation also included occasional trees, shrubs, and vines observed along fence lines and near the existing residence.

4.2 Subsurface Exploration and Investigation

Test pit explorations TP-1 through TP-12 were advanced to a maximum depth of 15 feet below ground surface (bgs). Exploration locations were selected to observe subsurface soil characteristics in proximity to proposed development areas and are indicated on Figure 2.

4.2.1 Soil Type Description

The field investigation indicated the southernmost parcel is generally covered with approximately 12 inches of sod and topsoil in the observed locations. In areas of the site where recent tilling had occurred, exploration indicated that disturbed, organic-rich topsoil extended to depths of 14 to 16 inches. Underlying the topsoil layer, subsurface soils resembling native USDA Gee soil series descriptions were encountered. Subsurface lithology was reasonably consistent at all explored locations and may generally be described by soil types identified in the following text.

Soil Type 1 – Lean CLAY with Sand / SILT with Sand

Soil Type 1 was observed to primarily consist of light brown to mottled brown and gray, moist to wet, medium stiff to stiff, low plasticity lean CLAY with sand and SILT with sand. Soil Type 1 was observed underlying the topsoil layer in all test pits and extended to depths of 6 to 13.5 feet below ground surface. In several test pits, Soil Type 1 was observed to the maximum depth of exploration.

Analytical laboratory testing conducted upon representative soil samples obtained from test pits TP-1 and TP-12 indicated approximately 77 to 84 percent by weight passing the No. 200 sieve and in situ moisture contents ranging from 29 to 36 percent. Atterberg Limits analysis indicated a liquid limit ranging from 33 to 38 percent and a plasticity index ranging from 8 to 14 percent. Soil Type 1 is classified CL and ML according to USCS specifications and A-6(12), A-6(11), and A-4(6) according to AASHTO specifications.

Soil Type 2 – Sandy Lean CLAY

Soil Type 2 was observed to primarily consist of blueish gray, wet, medium stiff, low plasticity sandy lean CLAY. Soil Type 2 was observed below Soil Type 1 in test pits TP-1 and TP-3 and extended to the maximum depth of exploration.

Analytical laboratory testing conducted upon a representative soil sample obtained from test pit TP-1 indicated approximately 66 percent by weight passing the No. 200 sieve and an in situ moisture content of approximately 32 percent. Atterberg Limits analysis indicated a liquid limit of 32 percent and a plasticity index of 14 percent. Soil Type 2 is classified CL according to USCS specifications and A-6(7) according to AASHTO specifications.

Soil Type 3 – Gravelly Lean CLAY with Sand (Apparent Sedimentary Conglomerate)

Soil Type 3 was observed to primarily consist of weathered orange to black, moist, stiff to very stiff, cemented, medium plasticity gravelly lean CLAY with sand. The degree of weathering and cementation varied throughout the soil unit and the presence of pebbles, gravels, and cobbles ranged trace to abundant. Soil Type 3 may represent Pleistocene to Pliocene, semi-consolidated, deeply weathered pebble to boulder sedimentary conglomerate (QTc) mapped by Evarts (2004). Soil Type 3 was observed below Soil Type 1 in test pits TP-8, TP-9, and TP-12 and extended to the maximum depth of exploration.

Analytical laboratory testing conducted upon a representative soil sample obtained from test pit TP-9 indicated approximately 57 percent by weight passing the No. 200 sieve and an in situ moisture content of approximately 19 percent. Atterberg Limits analysis indicated a liquid limit of 44 percent and a plasticity index of 25 percent. Soil Type 3 is classified CL according to USCS specifications and A-7-6(11) according to AASHTO specifications.

4.2.2 Groundwater

Groundwater was encountered within several test pits at depths ranging from 6 to 12 feet below ground surface. Review of nearby well logs obtained from the State of Washington Department of Ecology indicates that static groundwater levels in the area may vary significantly. Variations in ground water elevations likely reflect the screened interval depth of these wells, changes in ground surface elevation, and the presence of multiple aquifers and confining units. 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 fine-textured soils, shallow groundwater, and drainage. 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 anticipated stripping depth for sod and highly organic topsoil is anticipated to vary between 12 and 16 inches. The required stripping depth may increase in areas of existing fill, heavy organics, deep till zones, 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 gravel driveways, old foundations, basement walls, utilities, associated soft soils, and debris. Excavation areas should be backfilled with engineered structural fill.

Test pits excavated during site exploration activities were backfilled loosely with onsite soils. Test pits located within structural areas should be properly backfilled with engineered fill during site improvements construction.

Site grading activities should be performed in accordance with requirements specified in the 2012 *International Building Code* (IBC), Chapter 18 and Appendix J, with exceptions noted in the text herein. Site preparation, soil stripping, and grading activities should be observed and documented by Columbia West.

5.2 Engineered Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in the preceding text. Surface soils should then be scarified and compacted prior to additional fill placement. Engineered structural fill should be placed in loose lifts not exceeding 12 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within two percentage points of optimum conditions. A field density at least equal to 95 percent of the maximum dry density, obtained from the standard Proctor moisture-density relationship test (ASTM D698), is recommended for structural fill placement. 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. Most clean native soils may be suitable for use as structural fill if adequately moisture-conditioned to achieve recommended compaction specifications. Native clay soils with a plasticity index greater than 20 (Soil Type 3) should be evaluated and approved by Columbia West prior to re-use as structural fill. Because they are moisture-sensitive, fine-textured soils are often difficult to excavate and compact during wet weather conditions. If adequate compaction is not achievable with clean native soils, import structural fill consisting of well-graded granular material with a maximum particle size of three inches and no more than five percent passing the No. 200 sieve is recommended.

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

5.3 Cut and Fill Slopes

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

Final cut or fill slopes at the site should not exceed 2H:1V 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

Based upon correspondence with the client, residential foundations are anticipated to consist of shallow continuous perimeter or column spread footings. Footings should be designed by a licensed structural engineer and conform to the recommendations below. Typical building loads are not expected to exceed approximately 2 to 3 kips per foot for perimeter footings or 10 kips per column. If actual loading exceeds anticipated loading, additional analysis should be conducted for the specific load conditions and proposed footing dimensions.

The existing ground surface should be prepared as described in Section 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 modulus of subgrade reaction is estimated to be 250 psi/inch. The estimated coefficient of friction between in situ compacted native soil or engineered structural fill and in-place poured concrete is 0.45. 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 may require over-excavation of saturated subgrade soils and granular structural backfill prior to concrete placement. Over-excavation recommendations should be provided by Columbia West during foundation excavation and construction. Excavations adjacent to foundations should not extend within a 1.5H:1V angle projected down from the outside bottom footing edge without additional geotechnical analysis.

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

5.5 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.

5.6 Excavation

Soils at the site were explored to a maximum depth of 15 feet using a track-mounted excavator. Blasting or specialized rock-excavation techniques are not anticipated.

Groundwater was encountered within several test pits at depths ranging from 6 to 12 feet, however, perched groundwater layers may exist at shallow depths depending on seasonal fluctuations of the water table. Recommendations as described in Section 5.7 *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.

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.

5.7 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 some areas proposed for development. Utility trenches in shallow groundwater areas or excavations and cuts that remain open for even short periods of time may undermine or collapse due to groundwater effects. Placement of layers of riprap or quarry spalls in localized areas on shallow excavation side slopes may be required to limit instability. Over-excavation and stabilization of pipe trenches or other excavations with imported crushed aggregate or gabion rock may also be necessary to provide adequate subgrade support.

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

5.8 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 in situ 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 existing soils or compacted granular backfill. 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 Soil	Equivalent Fluid Pressure for Level Backfill			Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive		
Undisturbed native SILT and CLAY (Soil Types 1 and 2)	60 pcf	41 pcf	293 pcf	110 pcf	27°
Undisturbed native Gravelly Lean CLAY with Sand (Soil Type 3)	62 pcf	42 pcf	346 pcf	120 pcf	29°
Approved Structural Backfill Material					
WSDOT 9-03.12(2) compacted aggregate backfill	52 pcf	32 pcf	568 pcf	135 pcf	38°

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

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

A continuous one-foot-thick zone of free-draining, washed, open-graded 1-inch by 2-inch drain rock and a 4-inch perforated gravity 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 5.10, *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.

5.9 Seismic Design Considerations

According to the *United States Geologic Survey (USGS) 2012 Seismic Design Maps Summary Report,* the anticipated peak ground and maximum considered earthquake spectral response accelerations resulting from seismic activity for the subject site are summarized in Table 2.

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

	2% Probability of Exceedance in 50 yrs
Peak Ground Acceleration	0.39 g
0.2 sec Spectral Acceleration	0.89 g
1.0 sec Spectral Acceleration	0.40 g

The listed probabilistic ground motion values are based upon "firm rock" sites with an assumed shear wave velocity of 2,500 ft/s in the upper 100 feet of soil profile. These values should be adjusted for site class effects by applying site coefficients Fa and Fv as defined in 2012 IBC Tables 1613.3.3(1) and (2). The site coefficients are intended to more accurately characterize estimated peak ground and respective earthquake spectral response accelerations by considering site-specific soil characteristics and index properties.

The *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), indicates the northeast and southwest portions of the site may be represented by Site Class D and C respectively in *2012 IBC Section 1613.3.2*. Based upon site-specific testing and review of well logs and local geologic maps, site soils may be considered to be Site Class C. This site class designation indicates that some amplification of seismic energy may occur during a seismic event because of subsurface conditions.

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

Identification of specific seismic response spectra is beyond the scope of this investigation. If site structures are designed in accordance with recommendations specified in the 2012 IBC, the potential for peak ground accelerations in excess of the adjusted and amplified values should be understood.

5.10 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.

5.11 Infiltration Testing Results

To investigate the feasibility of subsurface disposal of stormwater, Columbia West conducted in situ infiltration testing at one location within the project area on June 10, 2015. Results of in situ infiltration testing are presented in Table 3. The soil classification presented in Table 3 is based upon laboratory analysis. The measured infiltration rate is presented as a coefficient of permeability (k) and has been reported without application of a factor of safety.

As indicated in Table 3, the test was conducted within test pit TP-1 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 SILT with sand. Soil laboratory analytical test reports are provided in Appendix A.

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

Table 3. Infiltration Test Data

Infiltration Test No.	Depth to Groundwater (feet bgs)	Location	Soil Type	Measured Infiltration Rate*	Approximate Test Depth	Passing No. 200 Sieve (%)
IT-1	12.0	TP-1, See Figure 2	ML, SILT with Sand	< 0.1 in/hr	7.0 feet	77.3

*Infiltration rate as defined by soil's approximate vertical coefficient of permeability (k).

As indicated in Table 3, soils in the tested location exhibited a very low coefficient of permeability. Due to the presence of shallow groundwater and fine-textured soils, the site has limited potential for infiltration. If infiltration is considered, a gravity overflow should be provided to an appropriate discharge location.

5.12 Bituminous Asphalt and Portland Cement Concrete

Based upon review of preliminary site plans, proposed development will include new asphalt concrete roadways. Columbia West recommends adherence to City of La Center paving guidelines unless a site-specific pavement design is conducted.

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 later in Section 5.13, *Wet Weather Construction Methods and Techniques*. Subgrade conditions should be evaluated and tested by Columbia West prior to placement of crushed aggregate base. Subgrade evaluation should include nuclear gauge density testing and wheel proof-roll observations conducted with a 12-cubic yard, double-axle dump truck or equivalent. Nuclear gauge density testing should be conducted at 150-foot intervals or as determined by the onsite geotechnical engineer. Subgrade soil should be compacted to at least 95 percent of the modified Proctor dry density, as determined by ASTM D1557. Areas of observed deflection or rutting during proof-roll evaluation should be excavated to a firm surface and replaced with compacted crushed aggregate.

Crushed aggregate base should be compacted and tested in accordance with the specifications outlined above. Asphalt concrete pavement should be compacted to at least 91 percent of maximum Rice density. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with Washington Department of Transportation and City of 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 specimen at a frequency of four cylinders per 100 cubic yards of poured concrete. Recommended field and analytical laboratory concrete testing includes slump, air entrainment, temperature, and unit weight.

5.13 Wet Weather Construction Methods and Techniques

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

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

Construction during wet weather conditions may require increased base thickness. Over-excavation may be necessary to provide a firm base upon which to place crushed aggregate. Geotextile filter fabric is also recommended. 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 future pavement performance.

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

5.14 Erosion Control Measures

Based upon field observations and laboratory testing, the erosion hazard for site soils in flat to shallow-gradient portions of the property is likely to be low. The potential for erosion generally increases in sloped areas. Therefore, disturbance to vegetation in sloped areas should be minimized during construction activities. Soil is also prone to erosion if unprotected and unvegetated during periods of 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 tactifier, or other means, as appropriate. Temporary slopes or exposed areas may be covered with straw, crushed aggregate, or riprap in localized areas to minimize erosion. Erosion and water runoff during wet weather conditions may be controlled by application of strategically placed channels and small detention depressions with overflow pipes.

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

5.15 Soil Shrink/Swell Potential

Based upon laboratory analysis, near-surface soils contain as much as 85 percent by weight passing the No. 200 sieve and exhibit a plasticity index ranging from 8 to 25 percent. This indicates potential for soil shrinking or swelling and underscores the importance of proper moisture-conditioning during fill placement. Medium plasticity soils should be placed and compacted at a moisture content at least two percent above optimum as determine by laboratory analysis.

5.16 Utility Installation

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

Utilities should be installed in general accordance with manufacturer's recommendations. Utility trench backfill should consist of crushed aggregate or other coarse-textured, free-draining material acceptable to the client, City of La Center, and Columbia West. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). The remaining backfill should be compacted to at least 95 percent of maximum dry density as determined by the standard Proctor moisture-density test (ASTM D698). Clean, free-draining, fine bedding sand is recommended for use in the pipe zone. With exception of the pipe zone, backfill should be placed in loose lifts not exceeding 12 inches in thickness.

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

6.0 CONCLUSION AND LIMITATIONS

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

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

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

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

Sincerely,

COLUMBIA WEST ENGINEERING, Inc.

Lance V. Lehto, PE, GE
President

LANCE V. LEHTO
STATE OF WASHINGTON
36301
REGISTERED
PROFESSIONAL ENGINEER
6-29-15

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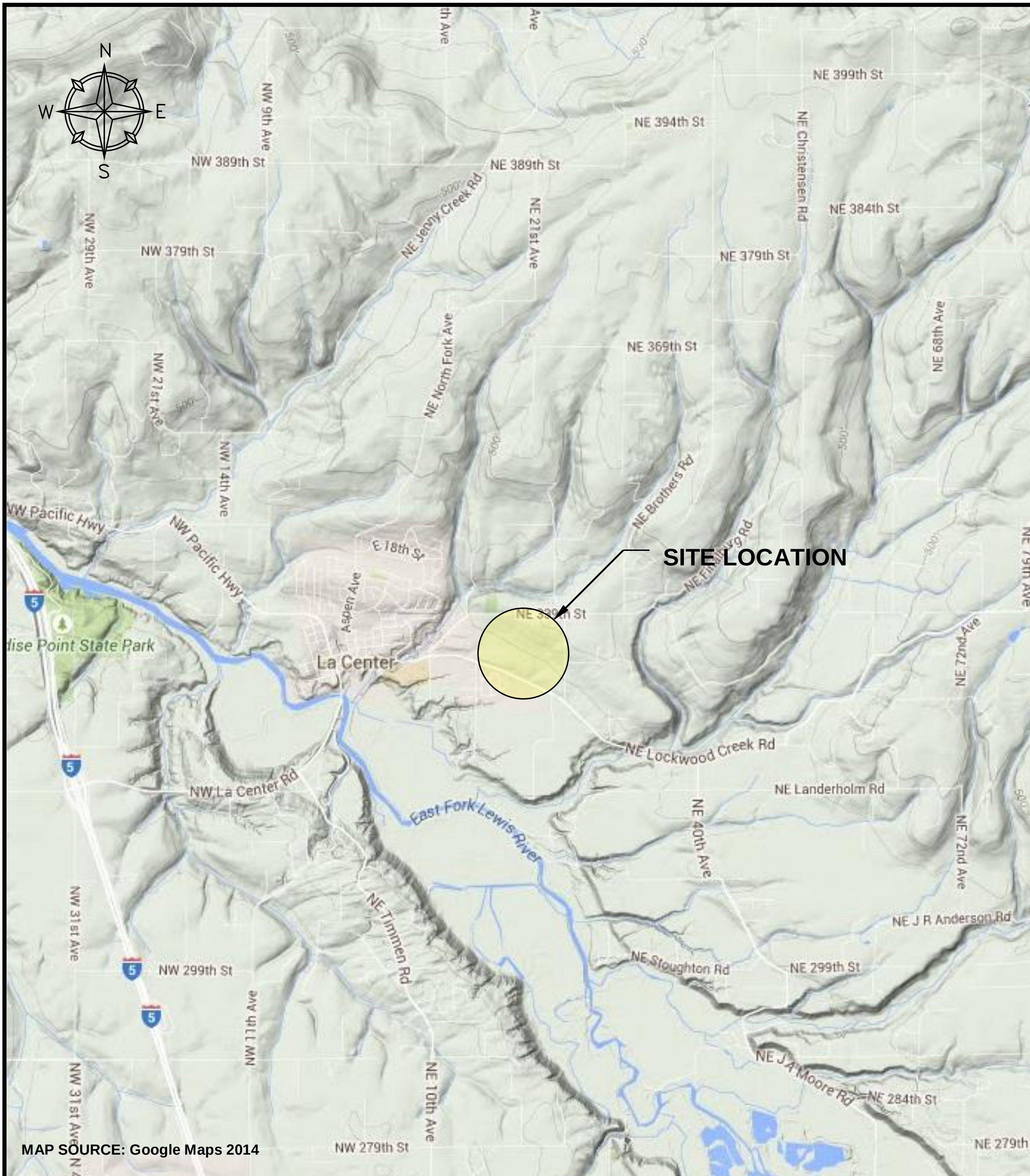
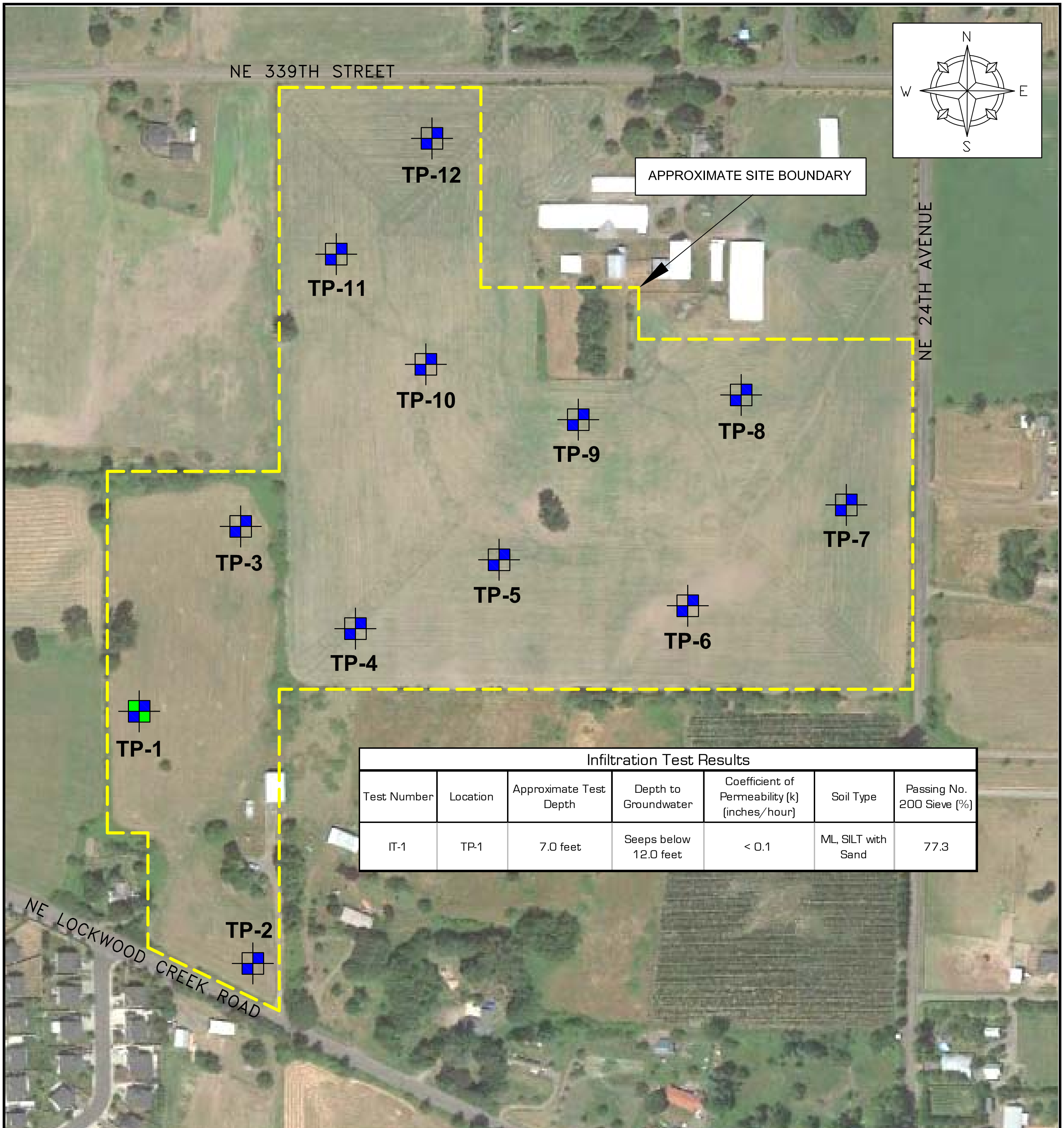
FIGURES



Geotechnical ▪ Environmental ▪ Special Inspections **Columbia West** Engineering, Inc 11917 NE 95th Street Vancouver, Washington 98682 Phone: 360-823-2900, Fax: 360-823-2901 www.columbiawestengineering.com	Design Checked: LVL Client: RK LD, LLC Job No.: 15159 CAD File: FIGURE 1 Scale: ~1:50,000	Drawn: GLW Date: 06/22/2015 Rev / By / Date	**SITE LOCATION MAP** SUNRISE TERRACE LA CENTER, WASHINGTON	FIGURE 1



Infiltration Test Results						
Test Number	Location	Approximate Test Depth	Depth to Groundwater	Coefficient of Permeability (k) (inches/hour)	Soil Type	Passing No. 200 Sieve (%)
IT-1	TP-1	7.0 feet	Seeps below 12.0 feet	< 0.1	ML, SILT with Sand	77.3

NOTES:
1. SITE LOCATION: 1908 NE LOCKWOOD CREEK ROAD, LA CENTER, WASHINGTON, PARCEL NOS. 209062000, 986027189, 986027188, 209047000
2. SITE IS APPROXIMATELY 33.7 ACRES IN SIZE.
3. DRAWING IS NOT TO SCALE.
4. BASE MAP OBTAINED FROM GOOGLE EARTH.
5. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
6. TEST PITS BACKFILLED LOOSELY WITH ONSITE SOIL ON 6/9/2015.

APPROXIMATE LOCATION OF TEST PIT EXPLORATION

APPROXIMATE LOCATION OF INFILTRATION TEST

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Client: RK LD, LLC
Job No: 15159
CAD File: FIGURE 2
Scale: NONE

Drawn: GLW
Date: 6/15/15

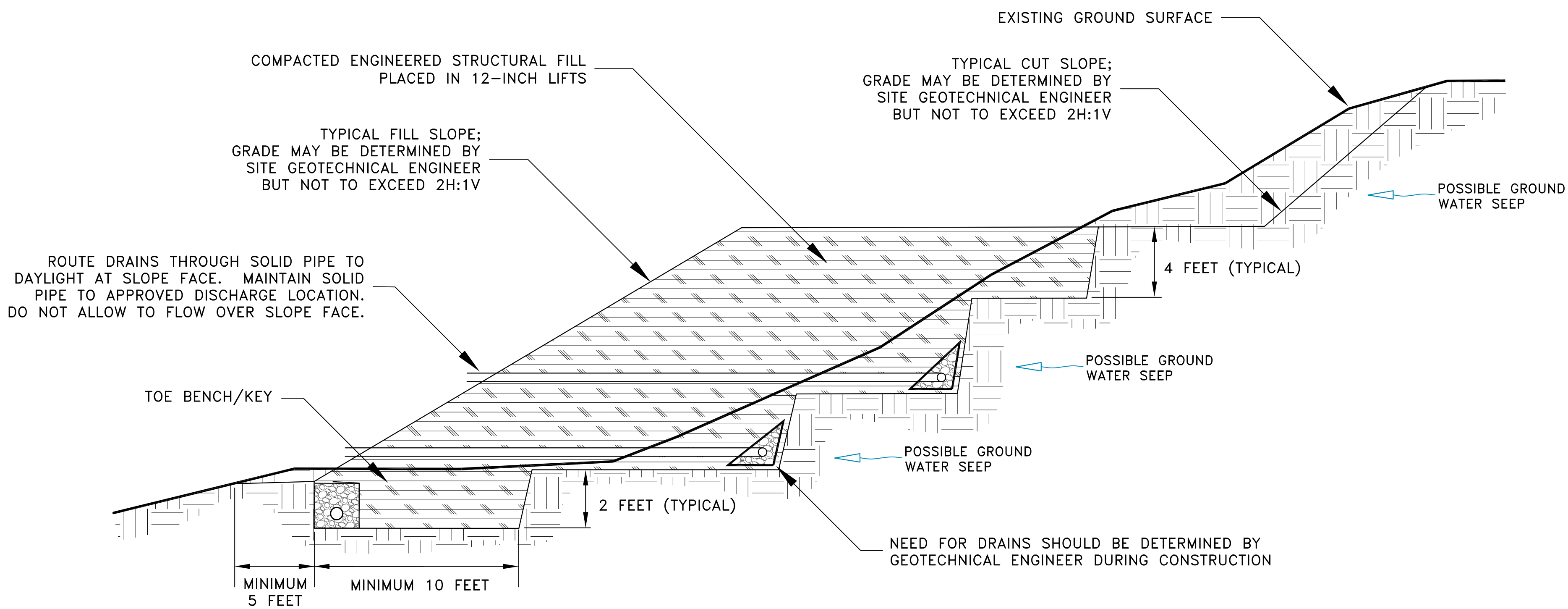
Rev	By	Date

EXPLORATION LOCATION MAP

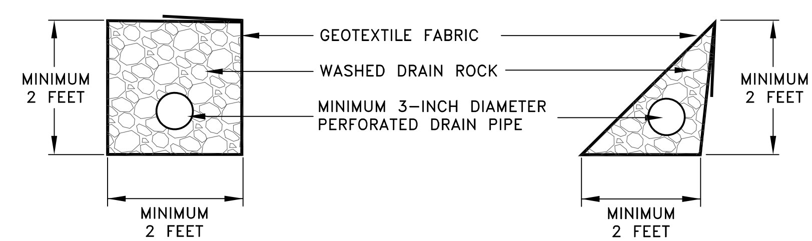
SUNRISE TERRACE
LA CENTER, WASHINGTON

FIGURE 2

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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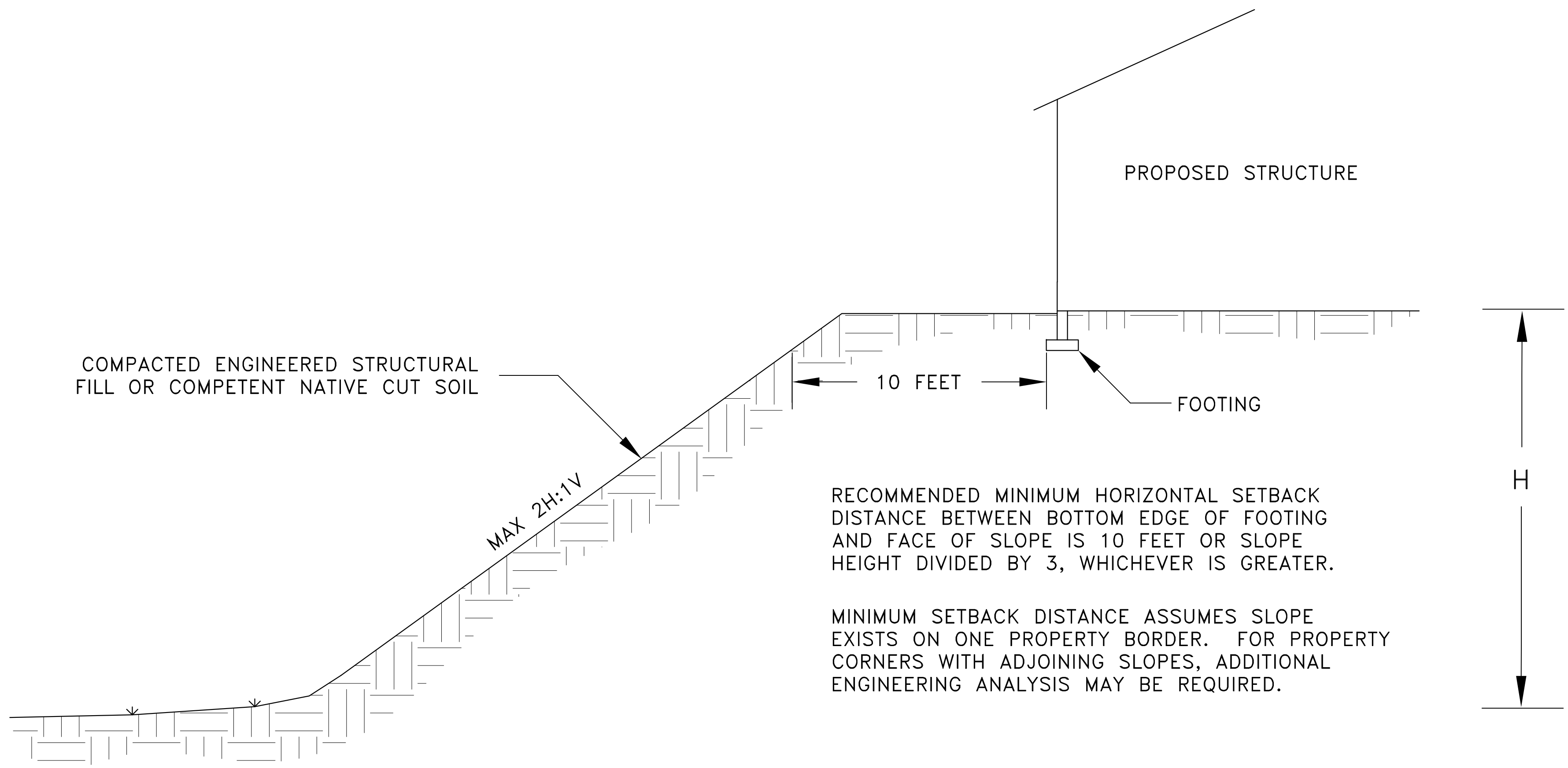
Client: RK LD, LCC	Rev	By	Date	
Job No: 15159				
CAD File: FIGURE 3				
Scale: NONE				

TYPICAL CUT AND FILL SLOPE CROSS-SECTION

SUNRISE TERRACE
LA CENTER, WASHINGTON

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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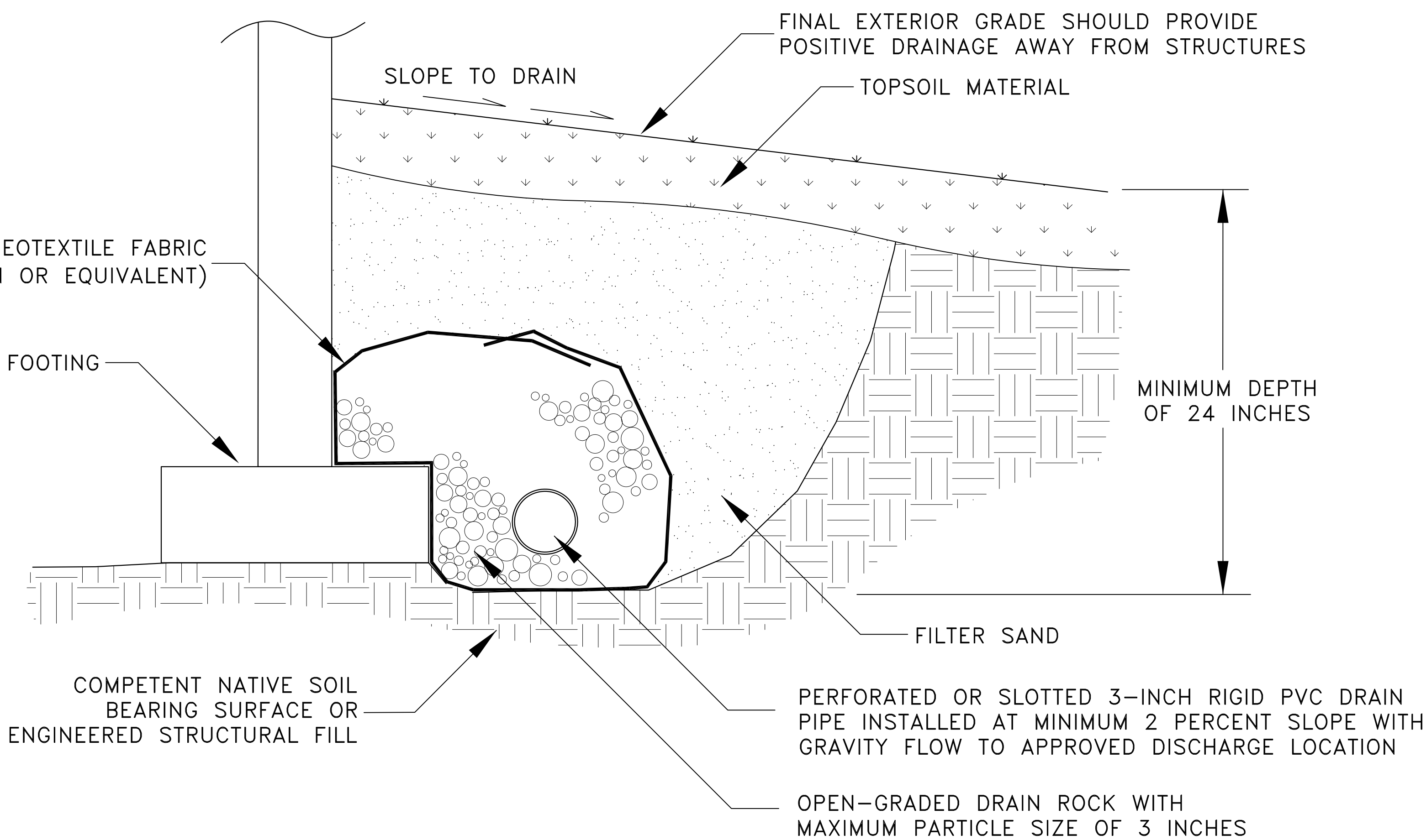
Design:	Drawn: GLW			MINIMUM FOUNDATION SLOPE SETBACK DETAIL
Checked: LVL	Date: 6/22/15			
Client: RK LD, LCC	Rev	By	Date	SUNRISE TERRACE LA CENTER, WASHINGTON
Job No: 15159				
CAD File: FIGURE 4				
Scale: NONE				

FIGURE 4

TYPICAL PERMITER 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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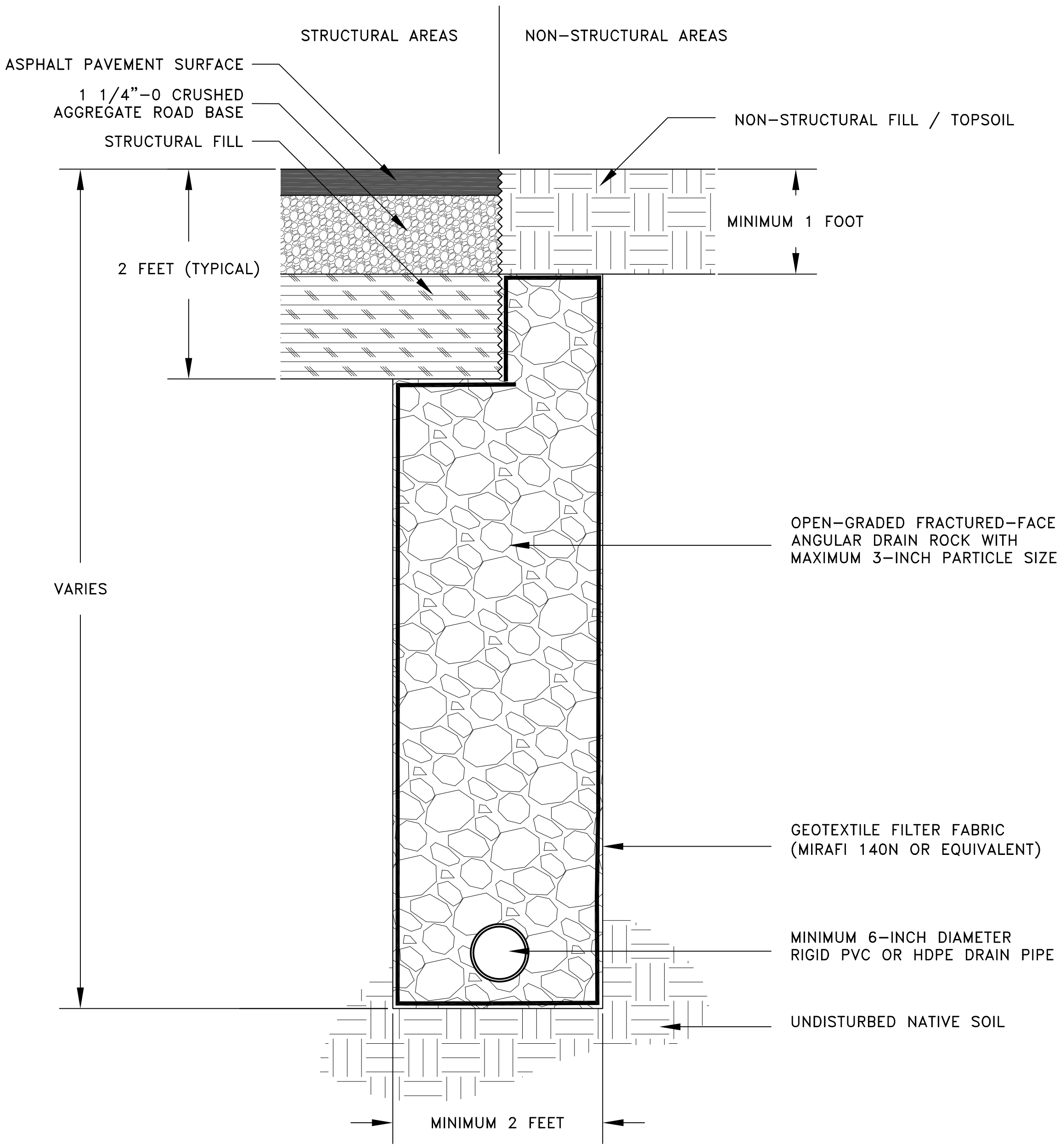
Design:	Drawn: GLW			TYPICAL PERIMETER FOOTING DRAIN DETAIL
Checked: LVL	Date: 6/22/15			
Client: RK LD, LCC	Rev	By	Date	SUNRISE TERRACE LA CENTER, WASHINGTON
Job No: 15159				
CAD File: FIGURE 5				
Scale: NONE				

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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Client: RK LD, LCC	Rev	By	Date
Job No: 15159			
CAD File: FIGURE 6			
Scale: NONE			

TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL

SUNRISE TERRACE
LA CENTER, WASHINGTON

FIGURE 6

APPENDIX A
LABORATORY TEST RESULTS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sunrise Terrace Residential Subdivision La Center, Washington	RK Land Development, LLC c/o Mr. Ed Greer 1520 SW Eaton Blvd Battle Ground, WA 98604	15159	S15-384
		REPORT DATE: 06/18/15	FIELD ID: TP1.1
		DATE SAMPLED: 06/10/15	SAMPLED BY: GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-01 depth = 2.5 feet	CL, Lean Clay with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-6(12)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913, D422

ADDITIONAL DATA

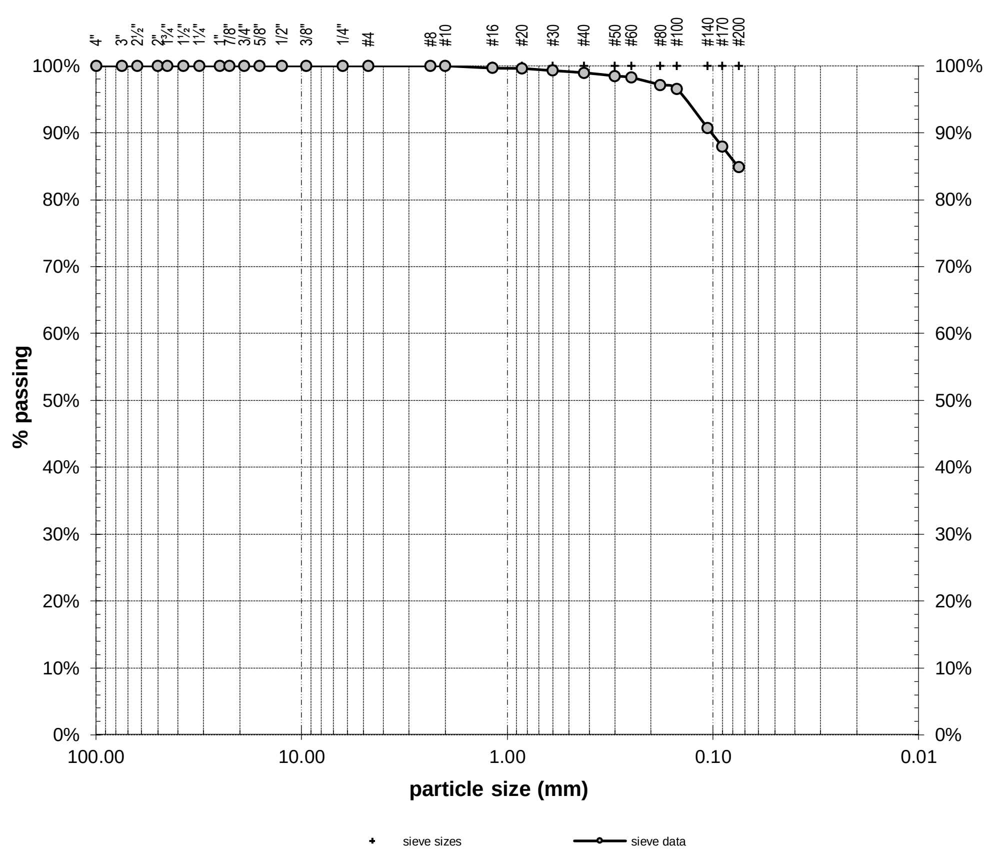
initial dry mass (g) =	192.0		
as-received moisture content =	32.6%	coefficient of curvature, C_C =	n/a
liquid limit =	38	coefficient of uniformity, C_U =	n/a
plastic limit =	24	effective size, $D_{(10)}$ =	n/a
plasticity index =	14	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 15.2%
% silt and clay = 84.8%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		100.0%		
	#10	2.00	100.0%			
	#16	1.18		99.7%		
	#20	0.850	99.6%			
	#30	0.600		99.3%		
	#40	0.425	99.0%			
	#50	0.300		98.5%		
	#60	0.250	98.3%			
	#80	0.180		97.2%		
	#100	0.150	96.6%			
	#140	0.106		90.7%		
	#170	0.090		87.9%		
	#200	0.075	84.8%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
06/16/15	JMR/JJC

COLUMBIA WEST ENGINEERING, INC. authorized signature

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
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		REPORT DATE: 06/18/15	FIELD ID: TP1.1
		DATE SAMPLED: 06/10/15	SAMPLED BY: GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-01 depth = 2.5 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

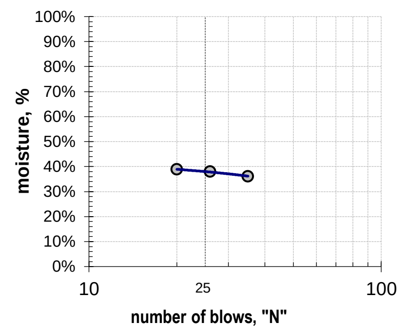
ATTERBERG LIMITS

liquid limit = 38
plastic limit = 24
plasticity index = 14

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.47	35.60	34.74	
dry soil + pan weight, g =	28.63	31.51	30.83	
pan weight, g =	20.77	20.73	20.76	
N (blows) =	35	26	20	
moisture, % =	36.1 %	37.9 %	38.8 %	

LIQUID LIMIT



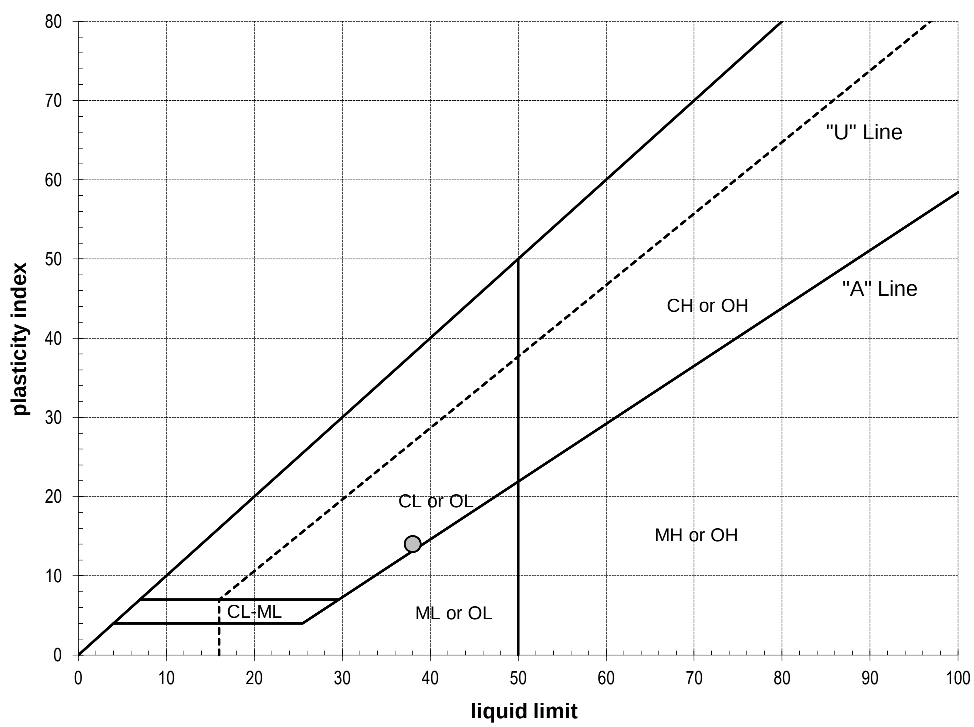
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.38	29.26		
dry soil + pan weight, g =	26.91	27.58		
pan weight, g =	20.57	20.57		
moisture, % =	23.2 %	24.0 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 15.2%
% silt and clay = 84.8%
% silt = n/a
% clay = n/a
moisture content = 32.6%

DATE TESTED	TESTED BY
06/16/15	JJC/JMR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sunrise Terrace Residential Subdivision La Center, Washington	RK Land Development, LLC c/o Mr. Ed Greer 1520 SW Eaton Blvd Battle Ground, WA 98604	15159	S15-385
		REPORT DATE: 06/18/15	FIELD ID: TP1.2
		DATE SAMPLED: 06/10/15	SAMPLED BY: GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
SILT with Sand	Test Pit TP-01 depth = 7 feet	ML, Silt with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-4(6)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913, D422

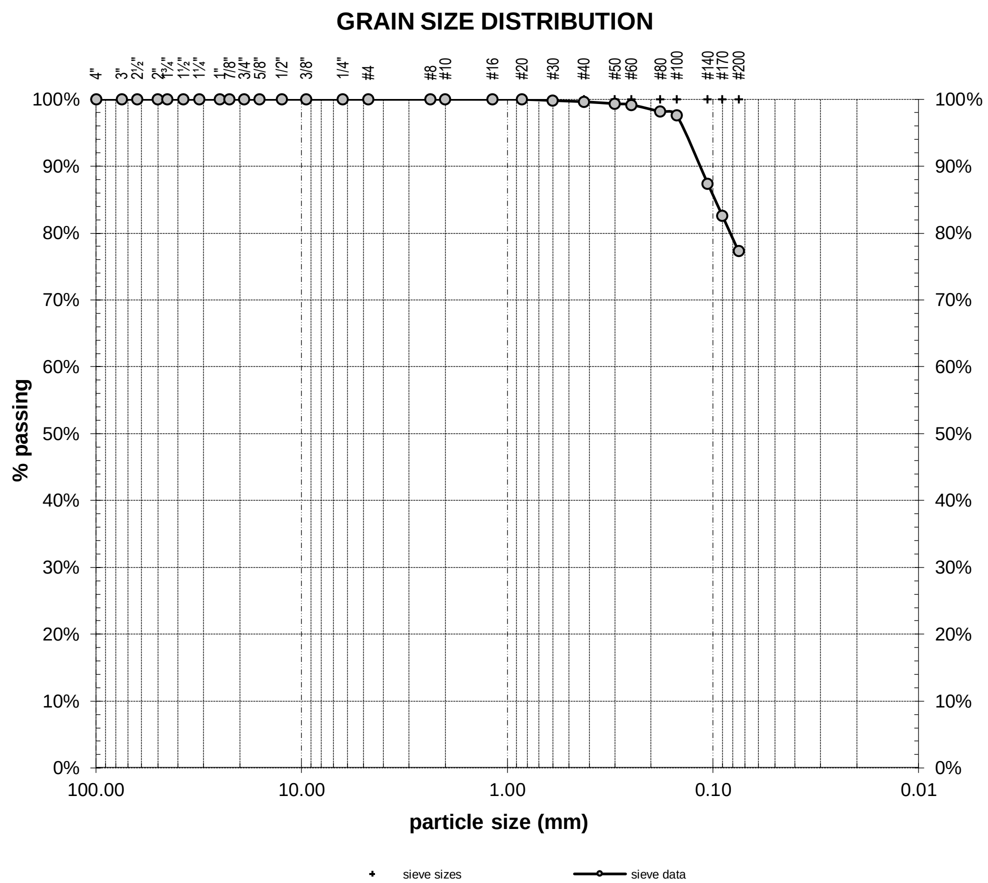
ADDITIONAL DATA

initial dry mass (g) =	124.1		
as-received moisture content =	36.0%	coefficient of curvature, C_C =	n/a
liquid limit =	33	coefficient of uniformity, C_U =	n/a
plastic limit =	25	effective size, $D_{(10)}$ =	n/a
plasticity index =	8	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 22.7%
% silt and clay = 77.3%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		100.0%		
	#10	2.00	100.0%			
	#16	1.18		100.0%		
	#20	0.850	100.0%			
	#30	0.600		99.8%		
	#40	0.425	99.6%			
	#50	0.300		99.3%		
	#60	0.250	99.2%			
	#80	0.180		98.2%		
	#100	0.150	97.6%			
	#140	0.106		87.4%		
	#170	0.090		82.6%		
	#200	0.075	77.3%			



DATE TESTED	TESTED BY
06/16/15	JMR/JJC

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sunrise Terrace Residential Subdivision La Center, Washington	RK Land Development, LLC c/o Mr. Ed Greer 1520 SW Eaton Blvd Battle Ground, WA 98604	15159	S15-385
		REPORT DATE	**FIELD ID**
		06/18/15	TP1.2
		DATE SAMPLED	**SAMPLED BY**
		06/10/15	GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
SILT with Sand	Test Pit TP-01 depth = 7 feet	ML, Silt with Sand

LABORATORY TEST DATA

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

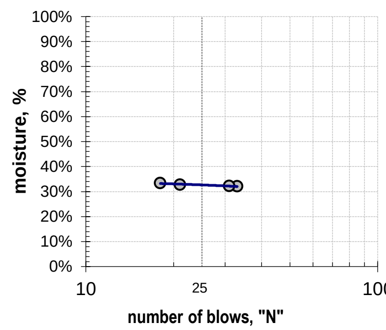
ATTERBERG LIMITS

liquid limit =	33
plastic limit =	25
plasticity index =	8

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	35.87	38.50	38.27	35.74
dry soil + pan weight, g =	32.20	34.17	33.92	31.97
pan weight, g =	20.76	20.72	20.63	20.69
N (blows) =	33	31	21	18
moisture, % =	32.1 %	32.2 %	32.7 %	33.4 %

LIQUID LIMIT



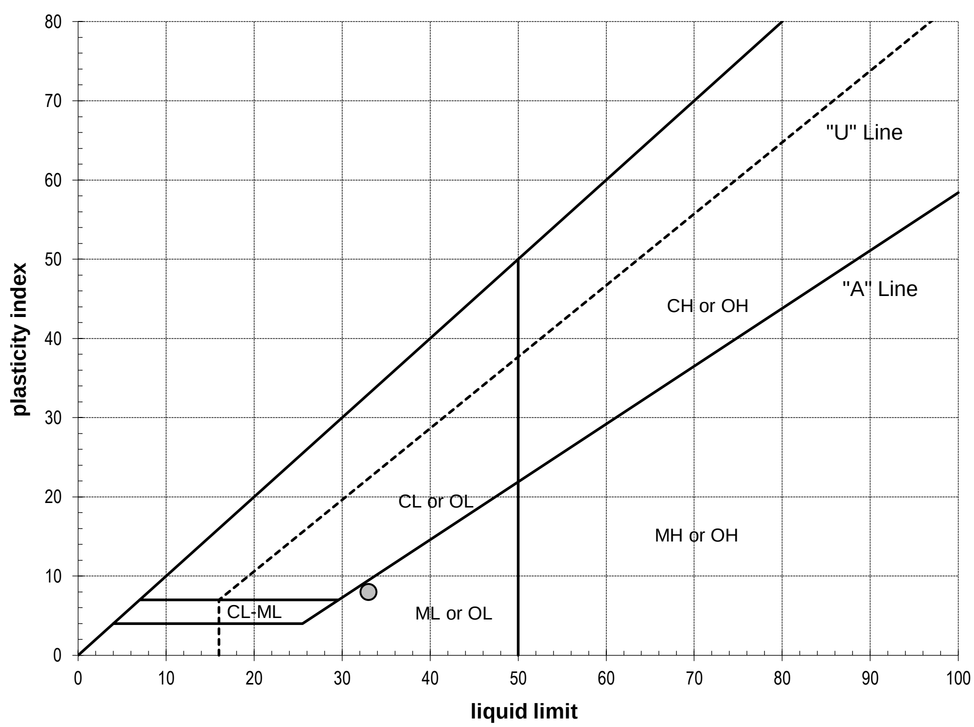
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	29.12	28.78		
dry soil + pan weight, g =	27.46	27.15		
pan weight, g =	20.77	20.65		
moisture, % =	24.8 %	25.1 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	22.7%
% silt and clay =	77.3%
% silt =	n/a
% clay =	n/a
moisture content =	36.0%

DATE TESTED	TESTED BY
06/17/15	JJC

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sunrise Terrace Residential Subdivision La Center, Washington	RK Land Development, LLC c/o Mr. Ed Greer 1520 SW Eaton Blvd Battle Ground, WA 98604	15159	S15-386
		REPORT DATE: 06/18/15	FIELD ID: TP1.3
		DATE SAMPLED: 06/10/15	SAMPLED BY: GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Lean CLAY	Test Pit TP-01 depth = 14 feet	CL, Sandy Lean Clay

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-6(7)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913, D422

ADDITIONAL DATA

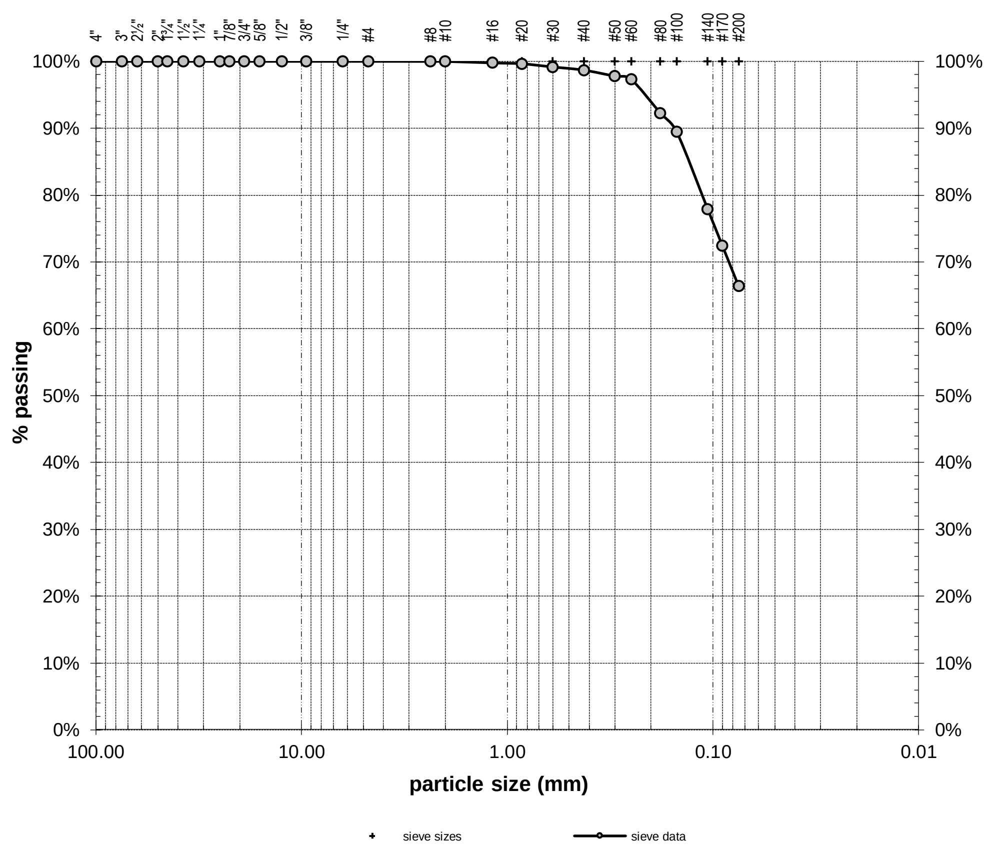
initial dry mass (g) =	151.8		
as-received moisture content =	31.9%	coefficient of curvature, C_C =	n/a
liquid limit =	32	coefficient of uniformity, C_U =	n/a
plastic limit =	18	effective size, $D_{(10)}$ =	n/a
plasticity index =	14	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 33.6%
% silt and clay = 66.4%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		100.0%		
	#10	2.00	100.0%			
	#16	1.18		99.8%		
	#20	0.850	99.6%			
	#30	0.600		99.2%		
	#40	0.425	98.7%			
	#50	0.300		97.8%		
	#60	0.250	97.3%			
	#80	0.180		92.3%		
	#100	0.150	89.5%			
	#140	0.106		77.9%		
	#170	0.090		72.5%		
	#200	0.075	66.4%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
06/16/15	JMR/JJC

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sunrise Terrace Residential Subdivision La Center, Washington	RK Land Development, LLC c/o Mr. Ed Greer 1520 SW Eaton Blvd Battle Ground, WA 98604	15159	S15-386
		REPORT DATE: 06/18/15	FIELD ID: TP1.3
		DATE SAMPLED: 06/10/15	SAMPLED BY: GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Lean CLAY	Test Pit TP-01 depth = 14 feet	CL, Sandy Lean Clay

LABORATORY TEST DATA

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

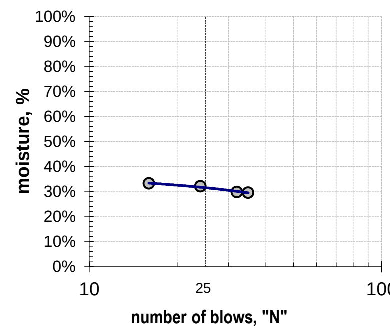
ATTERBERG LIMITS

liquid limit = 32
plastic limit = 18
plasticity index = 14

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	39.19	41.37	38.65	34.68
dry soil + pan weight, g =	35.01	36.66	34.32	31.15
pan weight, g =	20.89	20.86	20.86	20.52
N (blows) =	35	32	24	16
moisture, % =	29.6 %	29.8 %	32.2 %	33.2 %

LIQUID LIMIT



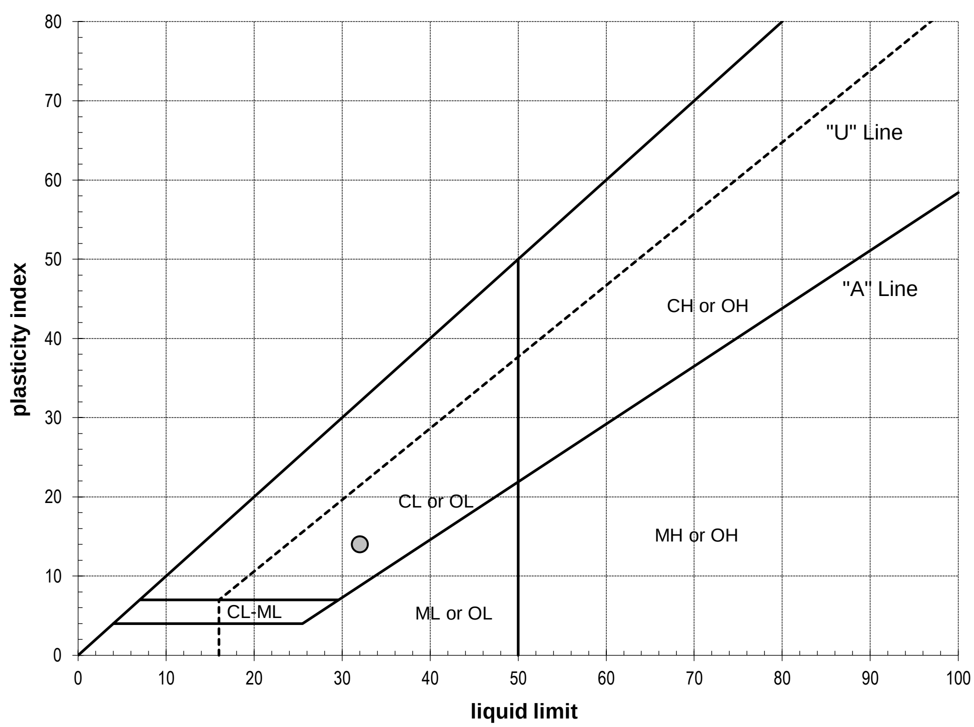
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.50	27.65		
dry soil + pan weight, g =	26.48	26.58		
pan weight, g =	20.78	20.73		
moisture, % =	17.9 %	18.3 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 33.6%
% silt and clay = 66.4%
% silt = n/a
% clay = n/a
moisture content = 31.9%

DATE TESTED	TESTED BY
06/17/15	MJR/JMR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sunrise Terrace Residential Subdivision La Center, Washington	RK Land Development, LLC c/o Mr. Ed Greer 1520 SW Eaton Blvd Battle Ground, WA 98604	15159	S15-387
		REPORT DATE: 06/18/15	FIELD ID: TP9.1
		DATE SAMPLED: 06/10/15	SAMPLED BY: GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Gravelly Lean CLAY with Sand	Test Pit TP-09 depth = 11 feet	CL, Gravelly Lean Clay with Sand

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

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913, D422

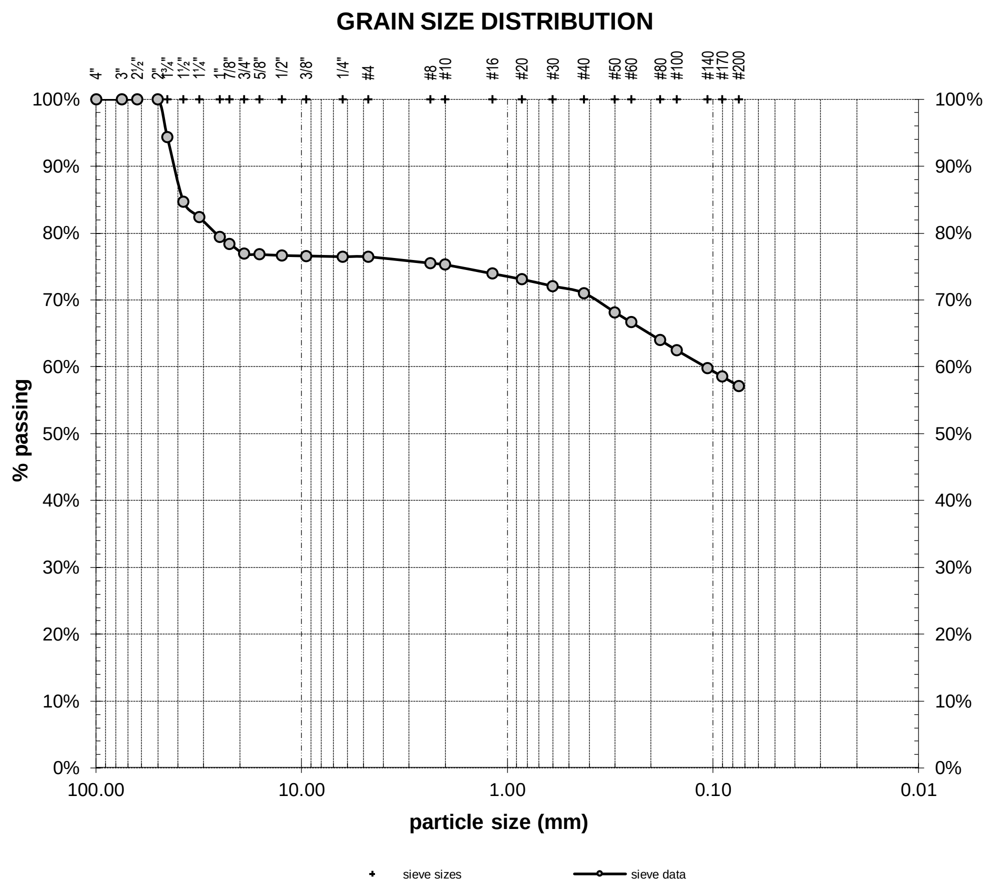
ADDITIONAL DATA

initial dry mass (g) =	1517.2		
as-received moisture content =	19.4%	coefficient of curvature, C_C =	n/a
liquid limit =	44	coefficient of uniformity, C_U =	n/a
plastic limit =	19	effective size, $D_{(10)}$ =	n/a
plasticity index =	25	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	0.109 mm

SIEVE DATA

% gravel = 23.5%
% sand = 19.3%
% silt and clay = 57.1%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0	100.0%			
	1.75"	45.0		94.4%		
	1.50"	37.5	84.7%			
	1.25"	31.5		82.4%		
	1.00"	25.0	79.4%			
	7/8"	22.4		78.4%		
	3/4"	19.0	76.9%			
	5/8"	16.0		76.8%		
	1/2"	12.5	76.7%			
	3/8"	9.50		76.6%		
	1/4"	6.30	76.5%			
	#4	4.75	76.5%			
SAND	#8	2.36		75.5%		
	#10	2.00	75.3%			
	#16	1.18		73.9%		
	#20	0.850	73.1%			
	#30	0.600		72.1%		
	#40	0.425	71.0%			
	#50	0.300		68.2%		
	#60	0.250	66.7%			
	#80	0.180		64.0%		
	#100	0.150	62.5%			
	#140	0.106		59.8%		
	#170	0.090		58.5%		
	#200	0.075	57.1%			



DATE TESTED	TESTED BY
06/16/15	BTT/JJC

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sunrise Terrace Residential Subdivision La Center, Washington	RK Land Development, LLC c/o Mr. Ed Greer 1520 SW Eaton Blvd Battle Ground, WA 98604	15159	S15-387
		REPORT DATE	**FIELD ID**
		06/18/15	TP9.1
		DATE SAMPLED	**SAMPLED BY**
		06/10/15	GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Gravelly Lean CLAY with Sand	Test Pit TP-09 depth = 11 feet	CL, Gravelly Lean Clay with Sand

LABORATORY TEST DATA

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

ATTERBERG LIMITS

liquid limit = 44
plastic limit = 19
plasticity index = 25

LIQUID LIMIT DETERMINATION

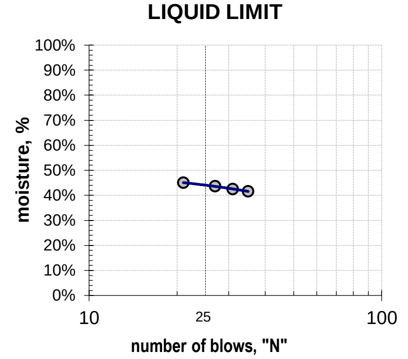
	❶	❷	❸	❹
wet soil + pan weight, g =	33.26	33.18	34.26	34.05
dry soil + pan weight, g =	29.61	29.45	30.16	29.89
pan weight, g =	20.83	20.68	20.75	20.66
N (blows) =	35	31	27	21
moisture, % =	41.6 %	42.5 %	43.6 %	45.1 %

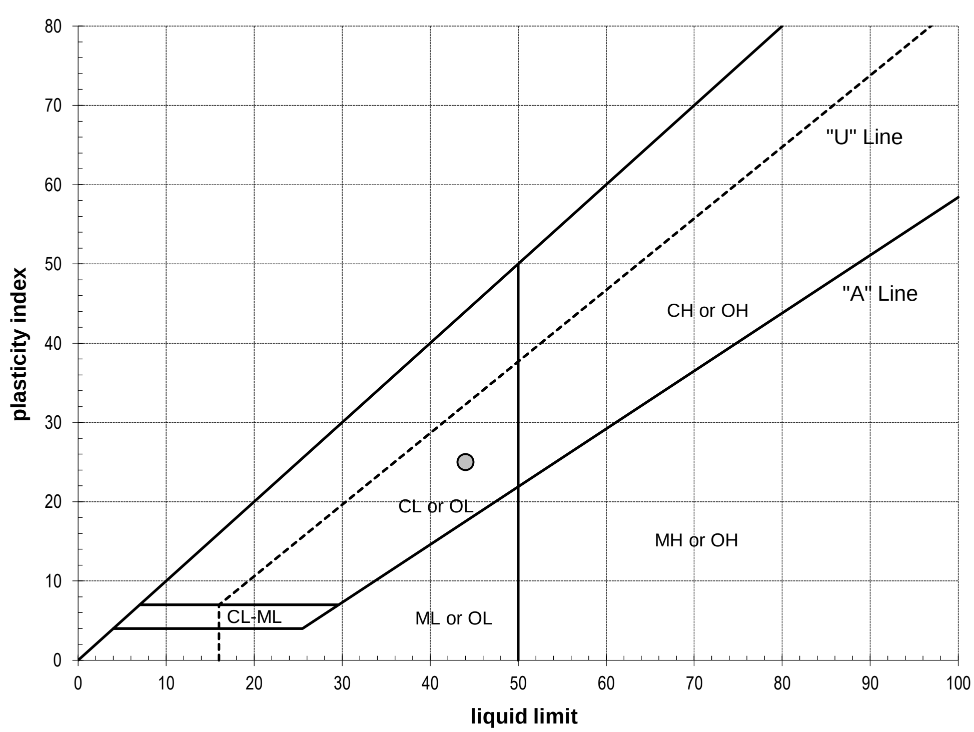
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.93	28.58		
dry soil + pan weight, g =	27.62	27.35		
pan weight, g =	20.57	20.82		
moisture, % =	18.6 %	18.8 %		





ADDITIONAL DATA

% gravel = 23.5%
% sand = 19.3%
% silt and clay = 57.1%
% silt = n/a
% clay = n/a
moisture content = 19.4%

DATE TESTED	TESTED BY
06/17/15	JJC

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sunrise Terrace Residential Subdivision La Center, Washington	RK Land Development, LLC c/o Mr. Ed Greer 1520 SW Eaton Blvd Battle Ground, WA 98604	15159	S15-388
		REPORT DATE: 06/18/15	FIELD ID: TP12.1
		DATE SAMPLED: 06/10/15	SAMPLED BY: GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-12 depth = 3.5 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, D422

ADDITIONAL DATA

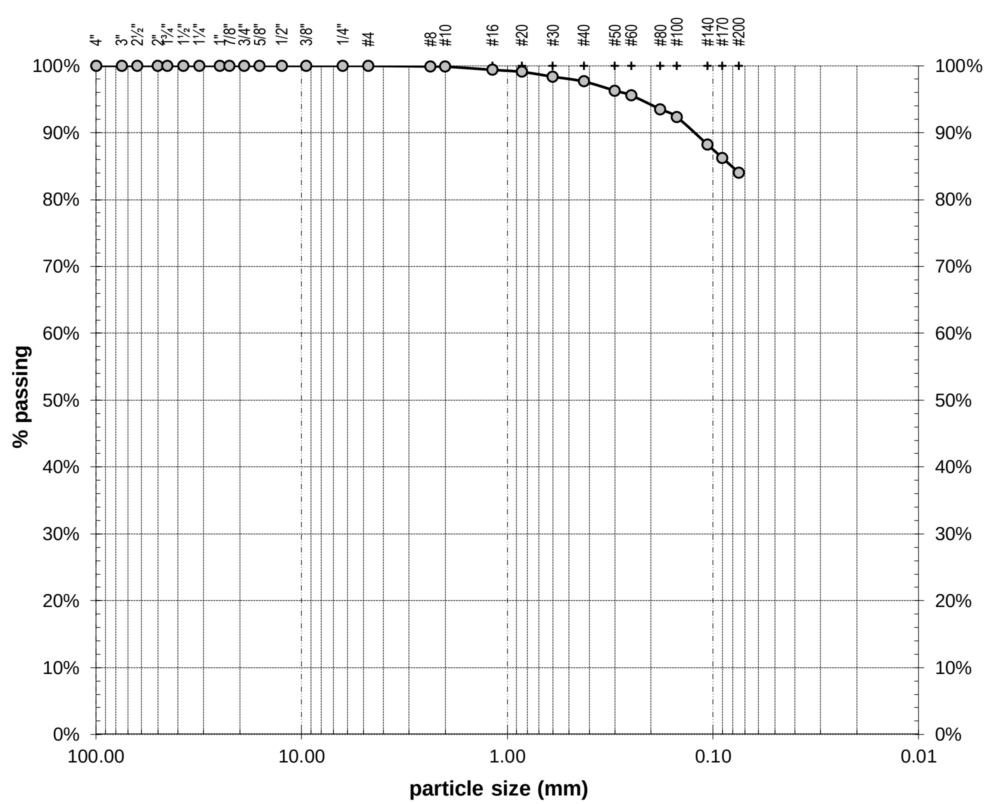
initial dry mass (g) =	155.4		
as-received moisture content =	29.1%	coefficient of curvature, C_C =	n/a
liquid limit =	36	coefficient of uniformity, C_U =	n/a
plastic limit =	23	effective size, $D_{(10)}$ =	n/a
plasticity index =	13	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 15.9%
% silt and clay = 84.1%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		99.9%		
	#10	2.00	99.9%			
	#16	1.18		99.4%		
	#20	0.850	99.1%			
	#30	0.600		98.4%		
	#40	0.425	97.7%			
	#50	0.300		96.3%		
	#60	0.250	95.6%			
	#80	0.180		93.5%		
	#100	0.150	92.4%			
	#140	0.106		88.2%		
	#170	0.090		86.2%		
	#200	0.075	84.1%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
06/16/15	JMR/JJC

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Sunrise Terrace Residential Subdivision La Center, Washington	RK Land Development, LLC c/o Mr. Ed Greer 1520 SW Eaton Blvd Battle Ground, WA 98604	15159	S15-388
		REPORT DATE: 06/18/15	FIELD ID: TP12.1
		DATE SAMPLED: 06/10/15	SAMPLED BY: GLW

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-12 depth = 3.5 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

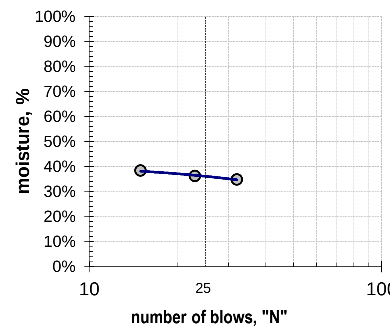
ATTERBERG LIMITS

liquid limit = 36
plastic limit = 23
plasticity index = 13

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	37.85	36.51	37.40	
dry soil + pan weight, g =	33.43	32.34	32.77	
pan weight, g =	20.75	20.82	20.69	
N (blows) =	32	23	15	
moisture, % =	34.9 %	36.2 %	38.3 %	

LIQUID LIMIT



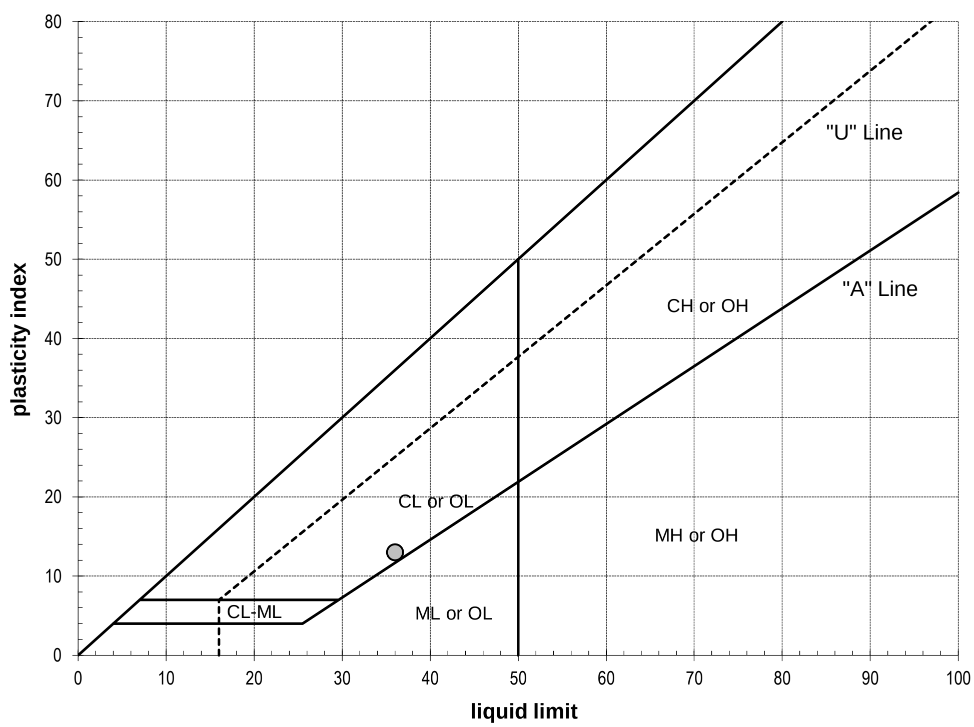
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.09	27.04		
dry soil + pan weight, g =	25.85	25.87		
pan weight, g =	20.57	20.78		
moisture, % =	23.5 %	23.0 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	15.9%
% silt and clay =	84.1%
% silt =	n/a
% clay =	n/a
moisture content =	29.1%

DATE TESTED	TESTED BY
06/17/15	MJR/JMR

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-1
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Contractors	Excavator	GLW	6/10/15
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	186 ft amsl	Seeps below 12.0 ft	0810	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						Approximately 12 inches of grass and topsoil till zone.					
	TP-1.1	Gee silt loam	A-6(12)	CL		Light brown to gray lean CLAY with sand, mottled, moist, stiff, lightly cemented, low plasticity. [Soil Type 1]	32.6	84.8	38	14	
5											
	TP-1.2		A-4(6)	ML		Light brown to orange SILT with sand, mottled, moist to wet, medium stiff, low plasticity. [Soil Type 1]	36.0	77.3	33	8	IT-1 D = 7.0 feet k < 0.1 in/hr
10						Occasional sandy interbeds.					
				▼		Groundwater seeps observed.					
						Increased excavation effort on weathered, orange to black, strongly cemented layer of sand, silt, and clay.					
	TP-1.3		A-6(7)	CL		Blue-gray sandy lean CLAY, wet, medium stiff, low plasticity. [Soil Type 2]	31.9	66.4	32	14	
15						Bottom of test pit at 15.0 feet. Groundwater encountered at 12.0 feet bgs.					

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-2
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**ENGINEER**	**DATE**
La Center, Washington	L&S Contractors	Excavator	GLW	6/10/15
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	158 ft amsl	not encountered	0917	0945

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 to 14 inches of grass and topsoil till zone.					
		Gee silt loam	A-6	CL		Brown to gray lean CLAY with sand, mottled, moist, medium stiff to stiff, low plasticity. [Soil Type 1]					
5						Occasional sandy interbeds throughout profile.					
			A-4	ML		Light brown to orange SILT with sand, mottled, very moist, medium stiff, low plasticity. [Soil Type 1]					
10											
						Bottom of test pit at 13.0 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-3
PROJECT LOCATION La Center, Washington	CONTRACTOR L&S Contractors	EQUIPMENT Excavator	ENGINEER GLW	DATE 6/10/15
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 204 ft amsl	GROUNDWATER DEPTH Seeps below 8.5 ft	START TIME 0954	FINISH TIME 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 12 inches of grass and topsoil till zone.					
		Gee silt loam	A-6	CL		Brown to gray lean CLAY with sand, mottled, moist, stiff, lightly cemented, low plasticity. [Soil Type 1] Fine roots observed to 4 feet.					
5											
			A-4	ML		Light brown SILT with sand, wet, medium stiff, low plasticity. [Soil Type 1] Groundwater seeps observed below 8.5 feet.					
10											
						Bottom of test pit at 13.5 feet. Groundwater encountered at 8.5 feet bgs.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-4
PROJECT LOCATION: La Center, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	ENGINEER: GLW	DATE: 6/10/15
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 196 ft amsl	GROUNDWATER DEPTH: Seeps below 6.0 ft	START TIME: 1027	FINISH TIME: 1055

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						Tilled farmland. Approximately 15 inches of disturbed, organic rich till zone.					
		Gee silt loam	A-6	CL		Brown to gray lean CLAY with sand, mottled, moist to wet, medium stiff to stiff, lightly cemented, low plasticity. [Soil Type 1]					
5											
						Groundwater seeps observed below 6 feet.					
			A-4	ML		Light brown SILT with sand, wet, medium stiff, low plasticity. [Soil Type 1]					
						Occasional sandy interbeds throughout profile.					
10											
						Increased excavation effort on weathered, orange to black, strongly cemented layer of sand, silt, and clay.					
			A-6	CL		Blue-gray sandy lean CLAY, wet, medium stiff, low plasticity. [Soil Type 2]					
						Bottom of test pit at 14.0 feet. Groundwater encountered at 6.0 feet bgs.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-5
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Contractors	Excavator	GLW	6/10/15
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	226 ft amsl	not encountered	1107	1135

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Tilled farmland. Approximately 14 inches of disturbed, organic rich till zone.					
		Gee silt loam	A-6	CL		Brown to gray lean CLAY with sand, mottled, moist, medium stiff to stiff, lightly cemented, low plasticity. [Soil Type 1]					
5											
						Occasional sandy interbeds throughout profile.					
10			A-4	ML		Light brown SILT with sand, very moist, medium stiff to stiff, low plasticity. [Soil Type 1]					
						Bottom of test pit at 11.5 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-6
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Contractors	Excavator	GLW	6/10/15
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	234 ft amsl	Seeps below 10.0 ft	1146	1210

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						Tilled farmland. Approximately 15 inches of disturbed, organic rich till zone.					
		Gee silt loam	A-6	CL		Orange-brown to gray lean CLAY with sand, mottled, moist, medium stiff to stiff, lightly cemented, low plasticity. [Soil Type 1]					
5											
			A-4	ML		Light brown SILT with sand, very moist, medium stiff to stiff, low plasticity. [Soil Type 1]					
10				▼		Groundwater seeps below 10 feet. Becomes wet.					
						Bottom of test pit at 11.0 feet. Groundwater encountered at 10.0 feet bgs.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-7
PROJECT LOCATION: La Center, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	ENGINEER: GLW	DATE: 6/10/15
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 274 ft amsl	GROUNDWATER DEPTH: not encountered	START TIME: 1217	FINISH TIME: 1250

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						Tilled farmland. Approximately 14 inches of disturbed, organic rich till zone.					
		Gee silt loam	A-6	CL		Brown to gray lean CLAY with sand, mottled, moist, medium stiff to stiff, lightly cemented, low plasticity. [Soil Type 1]					
5											
			A-4	ML		Light brown SILT with sand, very moist, medium stiff to stiff, low plasticity. [Soil Type 1]					
10											
						Plasticity increases.					
						Bottom of test pit at 13.0 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-8
PROJECT LOCATION: La Center, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	ENGINEER: GLW	DATE: 6/10/15
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 286 ft amsl	GROUNDWATER DEPTH: not encountered	START TIME: 1303	FINISH TIME: 1350

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						Tilled farmland. Approximately 16 inches of disturbed, organic rich till zone.					
		Gee silt loam	A-6	CL		Brown to gray lean CLAY with sand, mottled, moist, medium stiff to stiff, lightly cemented, low plasticity. [Soil Type 1]					
5											
			A-7	CL		Orange to black lean CLAY with sand, moist, stiff to very stiff, lightly to moderately cemented, medium plasticity. [Soil Type 3] Soils may represent CONGLOMERATE (QTc) of Evarts, 2004; described as rounded to sub angular pebbles and cobbles of igneous and sedimentary composition in semi-consolidated matrix of sand, silt, and clay.					
10											
						Bottom of test pit at 13.0 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-9
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Contractors	Excavator	GLW	6/10/15
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	266 ft amsl	not encountered	1405	1435

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						Recently sown farmland. Approximately 14 inches of disturbed, organic rich till zone.					
		Gee silt loam	A-6	CL		Brown to gray lean CLAY with sand, mottled, moist, medium stiff to stiff, lightly cemented, low plasticity. [Soil Type 1]					
5											
10				CL		Orange to black gravelly lean CLAY with sand, moist, very stiff, lightly to moderately cemented, medium plasticity. [Soil Type 3]					
	TP-9.1		A-7-6(11)			Soils may represent CONGLOMERATE (QTc) of Evarts, 2004; described as rounded to sub angular pebbles and cobbles of igneous and sedimentary composition in semi-consolidated matrix of sand, silt, and clay.	19.4	57.1	44	25	
						Bottom of test pit at 12.0 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-10
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Contractors	Excavator	GLW	6/10/15
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	272 ft amsl	not encountered	1441	1500

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						Tilled farmland. Approximately 12 inches of disturbed, organic rich till zone.					
		Gee silt loam	A-6	CL		Brown to gray lean CLAY with sand, mottled, moist, medium stiff to stiff, lightly cemented, low plasticity. [Soil Type 1]					
5											
						Plasticity increases.					
10											
						Bottom of test pit at 11.0 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-11
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Contractors	Excavator	GLW	6/10/15
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	274 ft amsl	not encountered	1508	1520

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						Tilled farmland. Approximately 14 inches of disturbed, organic rich till zone.					
		Gee silt loam	A-6	CL		Orange-brown to gray lean CLAY with sand, mottled, moist, medium stiff to stiff, lightly cemented, low plasticity. [Soil Type 1]					
5											
10						Bottom of test pit at 9.5 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Sunrise Terrace	RK Land Development, LLC		15159	TP-12
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
La Center, Washington	L&S Contractors	Excavator	GLW	6/10/15
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	292 ft amsl	not encountered	1527	1600

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						Tilled farmland. Approximately 14 inches of disturbed, organic rich till zone.					
	TP-12.1	Gee silt loam	A-6(11)	CL		Brown to gray lean CLAY with sand, mottled, moist, medium stiff to stiff, lightly cemented, low plasticity. [Soil Type 1]	29.1	84.1	36	13	
5											
			A-7	CL		Orange to black gravelly lean CLAY with sand, moist, stiff to very stiff, lightly to moderately cemented, medium plasticity. [Soil Type 3] Soils may represent CONGLOMERATE (QTc) of Evarts, 2004; described as rounded to sub angular pebbles and cobbles of igneous and sedimentary composition in semi-consolidated matrix of sand, silt, and clay.					
10											
						Bottom of test pit at 11.0 feet. Groundwater not encountered.					
15											

APPENDIX C
SOIL CLASSIFICATION INFORMATION

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

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

Consistency for Cohesive Soil

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

Relative Density for Granular Soil

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

Moisture Designations

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

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

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

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

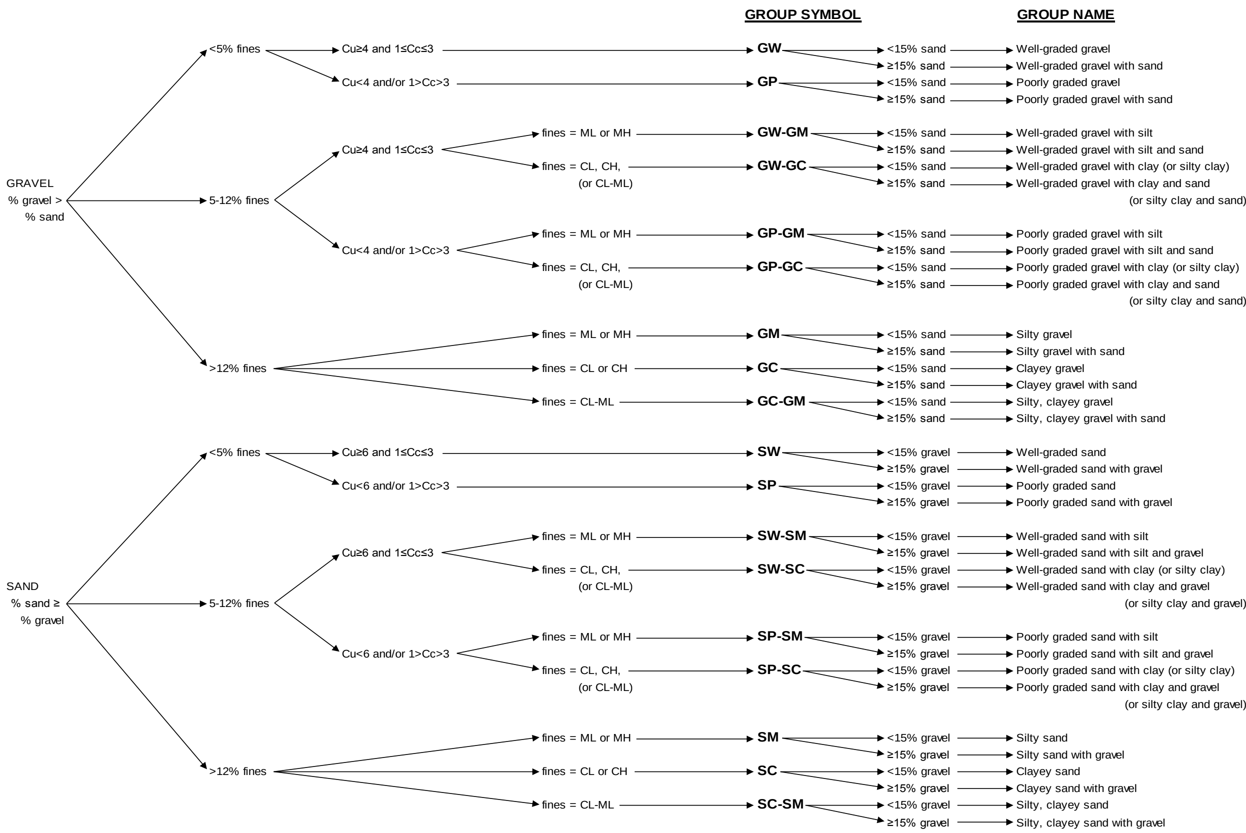
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

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

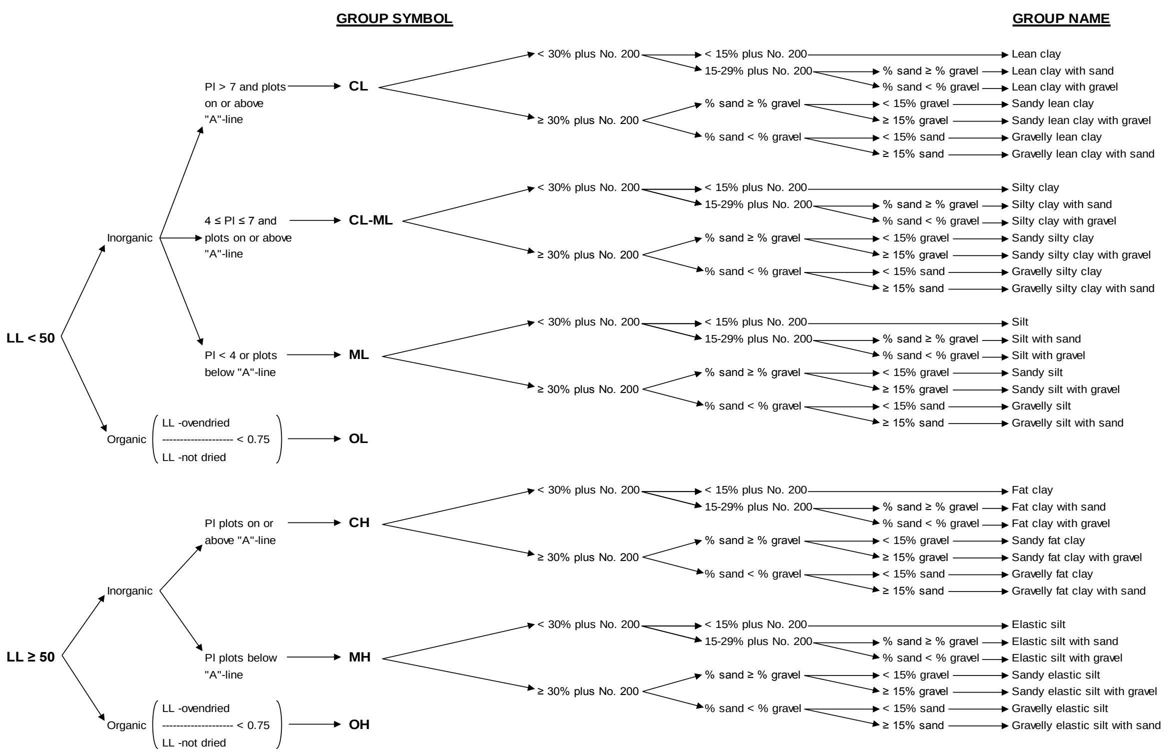
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

USCS SOIL CLASSIFICATION SYSTEM



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



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

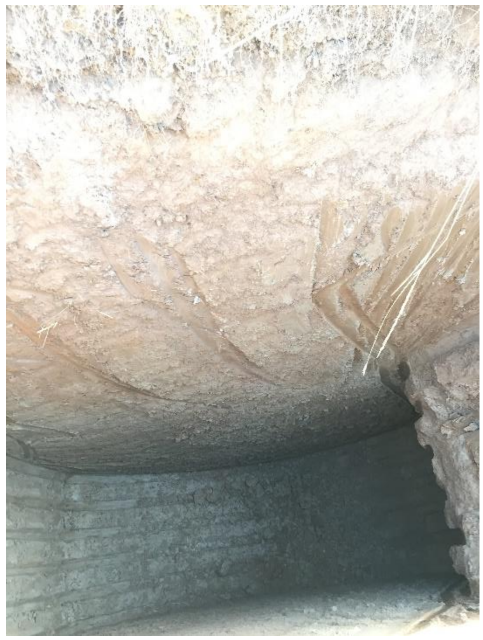
APPENDIX D
PHOTO LOG

SUNRISE TERRACE
LA CENTER, WASHINGTON
PHOTO LOG

Conducting Test Pit Exploration in Southernmost Parcel

Central Area of the Site, Facing Northwest

SUNRISE TERRACE
LA CENTER, WASHINGTON
PHOTO LOG



Fine-Textured Soil Profile Typical of the Site

Shallow Groundwater Observed in Several Test Pits

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Date: June 26, 2015
Project: Sunrise Terrace
La Center, Washington

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

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

Report Conclusions and Preliminary Nature

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

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

Additional Investigation and Construction QA/QC

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

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

Collected Samples

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

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