EXHIBIT 19

Geotechnical Site Investigation

Parklands at Camas Meadows

Camas, Washington

June 23, 2015

Geotechnical ■ Environmental ■ Special Inspections

Columbia West
Engineering, Inc

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Geotechnical ▪ Environmental ▪ Special Inspections
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GEOTECHNICAL SITE INVESTIGATION
PARKLANDS AT CAMAS MEADOWS
CAMAS, WASHINGTON

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Site Location: NE of NW Payne Street and NW Camas Meadows Drive Intersection
Parcels 175948000 and 986031650
Camas, Washington

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1.0 INTRODUCTION

Columbia West Engineering, Inc. was retained by Parklands at Camas Meadows, LLC to conduct a geotechnical site investigation for proposed development on tax parcel numbers 986031650 and 175948000 in Camas, Washington. The purpose of the investigation was to observe and assess subsurface soil conditions at specific locations and provide subsequent appropriate geotechnical engineering analyses to support property development feasibility, planning, and design recommendations. The specific scope of services was outlined in a proposal contract dated May 27, 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 northeast of the intersection of NW Payne Street and NW Camas Meadows Drive in Camas, Washington. The site is comprised of two tax parcels numbered 986031650 and 175948000 totaling approximately 36.4 acres. The regulatory jurisdictional agency is the City of Camas, Washington. The approximate latitude and longitude are N 45° 37' 40" and W 122° 26' 54", and the legal description is a portion of the SW and SE ¼ of Section 28, T2N, R3E, Willamette Meridian.

1.2 Proposed Development

Review of preliminary site plans provided by the client indicates that proposed development will consist of approximately 46 residential lots and 6 commercial buildings, parking areas, loading docks, private roadways and a future extension of NW Camas Meadows Drive. Stormwater facilities and underground utilities may also be constructed as part of proposed development. Columbia West understands that cut and fill areas 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 within the central portion of the Portland/Vancouver Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide.

According to the *Geologic Map of the Lacamas Quadrangle, Clark County, Washington* (US Geological Survey, Science Investigations Map 2924, 2006), the primary geologic unit present at the site is a Pleistocene gravel facies unit deposited by cataclysmic, glacial-outburst floods associated with Lake Missoula in Montana. However, in the areas explored during subsurface excavation, the unconsolidated boulder to cobbly gravel unit was either extremely thin or missing completely. Instead, the subsurface investigation revealed that the bulk of the site is underlain by two similar sedimentary formations. Test pit exploration indicated that the western corner of the site is underlain by an unnamed, Pleistocene to Pliocene, semi-consolidated, pebble to cobble conglomerate (QTc). This geologic unit is lithologically similar to the Pliocene or late Miocene Troutdale Formation, differing primarily in age of emplacement, degree of weathering, and the presence of hyaloclastite interbeds. Previously published geologic mapping has identified this unit as the Troutdale Formation.

The southern and eastern portion of the site is underlain by the Hyaoclastic sandstone member of the Troutdale formation (Ttfh). This Pliocene to Pleistocene formation is comprised of coarse-grained sandstone and pebble conglomerate containing basalt pebbles and cobbles. This geologic unit is lithologically similar to the Pliocene or late Miocene Troutdale Formation, differing primarily in age of emplacement, degree of weathering, and the presence of hyaloclastite interbeds. Previously published geologic mapping has identified this unit as the Troutdale Formation.

The *Web Soil Survey (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2013 Website)* indicates the site is underlain by three soil types. Hesson clay loam soils are mapped on the majority of the site from the northwest corner to the southwest corner of the property, while Cove silty clay loam and Lauren gravelly loam soils are mapped in the northern and northwestern portions of the property, respectively. Soils resembling the Lauren series were not encountered during subsurface excavation.

Although actual on-site soils may vary from the broad USDA descriptions, Lauren soils are generally coarse-textured, well drained soils with rapid permeability. Cove soils are generally fine-textured, poorly drained soils with very slow permeability and high shrink-swell potential. Hesson soils are fine-textured, well drained soils with moderately slow permeability and moderate shrink-swell potential.

3.0 REGIONAL SEISMOLOGY

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

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

Portland Hills Fault Zone

The Portland Hills Fault Zone consists of several northwest-trending faults located along the northeastern margin of the Tualatin Mountains, also known as the Portland Hills, and the southwest margin of the Portland Basin. The fault zone is approximately 25 to 30 miles in length and is located approximately 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 is thought to exist which suggests that fault movement has impacted shallow Holocene deposits and deeper Pleistocene sediments. Seismologists recorded a M3.2 earthquake thought to be associated with the fault zone near Kelly Point Park in November 2012, a M3.9 earthquake thought to be associated with the fault zone near Kelly Point Park in April 2003, and a M3.5 earthquake possibly associated with the fault zone occurred approximately 1.3 miles east of the fault in 1991. Therefore, the Portland Hills Fault Zone is generally thought to be potentially active and capable of producing possible damaging earthquakes.

Gales Creek-Newberg-Mt. Angel Fault Zone

Located approximately 35 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 Creek Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 1 mile east of the site, and form part of the northeastern margin of the Portland basin. According to *Geology and Groundwater Conditions of Clark County Washington (USGS Water Supply Paper 1600, Mundorff, 1964)* and the *Geologic Map of the Lake Oswego Quadrangle (Oregon DOGAMI Series GMS-59, 1989)*, the Lacamas Creek fault zone consists of shear contact between the Troutdale Formation and underlying Oligocene andesite-basalt bedrock. Secondary shear contact associated with the fault zone may have produced a series of prominent northwest-southeast geomorphic lineaments in proximity to the site.

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

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

<u>Cascadia Subduction Zone</u>

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

4.0 GEOTECHNICAL FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance and eight test pits (TP-1 through TP-8) was conducted at the site on June 4, 2015. 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. Laboratory test results are presented in Appendix A. Subsurface exploration locations are indicated on Figure 2. Exploration logs are presented in Appendix B. Soil descriptions and classification information are provided in Appendix C. A photo log is provided in Appendix D.

4.1 Surface Investigation and Site Description

The approximately 36.4-acre subject site occupies two tax parcels to the east of the Camas Meadows Golf Club located at 4105 NW Camas Meadows Drive in Camas, Washington. The site was previously undeveloped and is densely vegetated with large fir and deciduous trees, associated understory vegetation, and a wetland area. The site lies at the toe of a north-facing slope near the western end of Lacamas Lake. Site elevations range from approximately 240 feet elevation in the southern portion of the site to approximately 190 feet elevation along the northern property boundary. Slope grades range from isolated short slopes of approximately 20 percent in the south central portion of the property to generally flat in the wetland area of the northern property boundary. Most slopes are gentle and range from 5 to 10 percent.

4.2 Subsurface Exploration and Investigation

Test pit explorations TP-1 through TP-8 were advanced at the site to a maximum depth of 16.5 feet using a track-mounted excavator on June 4, 2015. Subsurface exploration locations were selected to observe 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 site is generally covered with approximately 10 to 18 inches of topsoil and associated organic-rich root zone material at the locations observed. Underlying the topsoil layer, fine-textured silt and clay soils underlain by weathered conglomerate bedrock and competent conglomerate bedrock were encountered. Subsurface lithology may generally be described by the following soil types for engineering purposes.

<u>Soil Type 1 – *Sandy SILT to Sandy FAT CLAY*</u>

Soil Type 1 was observed to consist primarily of medium brown medium stiff, moist to wet, moderate to high plasticity sandy SILT to sandy FAT CLAY. Soil Type 1 was observed underlying the topsoil layer in test pits TP-3 through TP-6 and TP-8 to a maximum depth of 5 feet.

Analytical laboratory testing conducted upon representative soil samples obtained from test pits TP-3 (sandy SILT) and TP-6 (sandy FAT CLAY) indicate approximately 57 to 63 percent by weight passing the No. 200 sieve and in situ moisture content ranging from 32 to 36 percent. Atterberg test results indicated a liquid limit ranging from 44 to 51 percent and a plasticity index ranging from 16 to 25 percent. Soil Type 1 is classified as ML, sandy SILT, and CH, sandy FAT CLAY according to USCS specifications and A-7-6(7) and A-7-6(7) according to AASHTO specifications.

Soil Type 2 – Clayey SAND to Poorly Graded GRAVEL with silt and sand

Soil Type 2 was observed to consist primarily of light brown to multi-colored, dense to very dense, moist to wet, clayey SAND and poorly graded GRAVEL with silt and sand. Soil Type 2 represents weathered conglomerate bedrock. Soil Type 2 was encountered underlying surficial fine textured soils or topsoil in all test pits.

Analytical laboratory testing conducted upon representative soil samples obtained from test pit TP-8 indicate approximately 8 to 20 percent by weight passing the No. 200 sieve and in situ moisture content ranging from 33 to 43 percent. Atterberg test results indicated a liquid limit ranging from 43 to 46 percent and a plasticity index of 18 percent. Soil Type 2 is classified as SC, clayey SAND and GP-GM, poorly-graded GRAVEL with silt and sand according to USCS specifications and A-2-7(0) according to AASHTO specifications.

Soil Type 3: Weathered and Competent Conglomerate Bedrock

Weathered and competent conglomerate bedrock was encountered in all test pits at various depths. The conglomerate bedrock encountered generally resembled the descriptions of the unnamed Pleistocene to Pliocene, semi-consolidated, pebble to cobble conglomerate (QTc) and the Hyaoclastic sandstone member of the Troutdale formation (Ttfh). The bedrock consisted of angular to sub-rounded clasts of various sizes cemented in a matrix of sand, silt, and clay. The bedrock was very dense and excavator refusal was noted at various depths as indicated in Table 1 in Section 5.7, *Excavation.*

4.2.2 Groundwater

Groundwater was encountered in test pits TP-1 and TP-8 at depths of 2.5 feet and 15 feet below ground surface, respectively. Standing water was observed at the ground surface elevation in the wetland which occupies the central north portion of the site. According to *Clark County Maps Online*, the static aquifer elevation in the vicinity of the subject site ranges from 190 to 210 feet amsl. These elevations correspond to an approximate depth to groundwater between 0 and 20 feet below ground surface.

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. Piezometer installation and long-term monitoring, beyond the scope of this investigation, would be necessary to provide more detailed groundwater information.

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. 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 from 10 to 18 inches. The required stripping depth may increase in areas of heavy organics, large tree root balls, or disturbed soil. 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, unsuitable, or undocumented fill encountered during grading or construction activities should be removed completely and thoroughly from structural areas. This includes old foundations, basement walls, utilities, associated soft soils, and debris. Excavation areas should be backfilled with engineered structural fill.

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, grading activities, and demolition debris removal verification should be observed and documented by an experienced geotechnical engineer or designated representative.

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

Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D6938. Field compaction testing should be performed for each vertical foot of engineered fill placed. Engineered fill placement should be observed by an experienced geotechnical engineer or designated representative.

Engineered structural fill placement activities should be performed during dry summer months if possible. If fill placement occurs during dry weather conditions, clean, fine-textured native soils are anticipated be suitable for use as structural fill if adequately moisture-conditioned to achieve recommended compaction specifications. Areas of sandy FAT CLAY that may be encountered may not be suitable for building foundation subgrade or road subgrade embankments. The use of clay soils for structural fill should be analyzed by Columbia West during site grading activities.

Because they are moisture-sensitive, fine-textured soils such as Soil Type 1 are often difficult to excavate and compact during wet weather conditions. If adequate compaction is not achievable with clean, fine-textured soils, import 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 for structural fill.

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

5.3 Cut and Fill Slopes

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

Final cut or fill slopes at the site should not exceed 2H:1V or 15 feet in total height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 4.

Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Fill slopes should be constructed by placing fill material in maximum 12-inch level lifts, compacting as described in Section 5.2, *Engineered Structural Fill* and horizontally benching where appropriate. Fill slopes should be overbuilt, compacted, and trimmed at least two feet horizontally to provide adequate compaction of the outer slope face. Proper cut and fill slope construction is critical to overall project stability and should be observed by an experienced geotechnical engineer.

5.4 Foundations

Review of preliminary site plans indicates that both residential and commercial/light industrial buildings are proposed. 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 3 to 4 kips per foot for perimeter footings or 80 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 a 12-inch-thick layer of crushed aggregate base compacted to at least 95 percent of modified Proctor maximum dry density (ASTM D1557) placed on firm competent in situ soil or engineered structural fill. Disturbed surface soils and unsuitable fill should be removed from foundation alignments and replaced with structural fill.

Footings should have a minimum width of 18 inches and 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 a qualified geotechnical engineer 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.

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 and bearing on Soil Type 1 is 1,500 psf. The estimated allowable bearing capacity for well-drained foundations bearing upon Soil Types 2 and 3 is 2,000 psf. Bearing capacity may be increased by one-third for transient lateral forces such as seismic or wind. The estimated coefficient of friction between in situ compacted native soil or engineered structural fill and in-place poured concrete is 0.35. Lateral forces may also be resisted by an assumed passive soil equivalent fluid pressure of 250 psf/f against embedded footings. The upper six inches of soil should be neglected in passive pressure calculations.

Foundations should not be permitted to bear upon existing fill, soft soil, or disturbed soil. Because soil is often heterogeneous and anisotropic, it is recommended that an experienced geotechnical engineer or designated representative observe foundation excavations prior to placing forms or reinforcing bar to verify subgrade support conditions are as anticipated in this report.

5.5 Slabs on Grade

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

Preparation and compaction beneath slabs should be performed in accordance with the recommendations presented in Section 5.1, *Site Preparation and Grading* and Section 5.2, *Engineered Structural Fill*. Slabs should be underlain by at least 6 inches of free-draining 1

¼”-0 crushed aggregate meeting WSDOT 9-03.9(3). Geotextile filter fabric conforming to *WSDOT 2010 Standard Specification M 41-10, 9-33.2(1), Geotextile Properties, Table 3: Geotextile for Separation or Soil Stabilization* may be used below the crushed aggregate to increase subgrade support. If desired, a moisture barrier may be constructed beneath the slabs. Slabs should be appropriately waterproofed in accordance with the desired type of finished flooring. Slab thickness and reinforcement should be designed by an experienced structural engineer in accordance with anticipated loads.

5.6 Settlement

Total long-term static footing displacement for shallow to medium-depth foundations constructed as described in this report is not anticipated to exceed approximately 1 inch. Differential settlement between comparably loaded footing elements is not expected to exceed approximately ½ inch over a span of 50 feet. The resulting vertical displacement after loading may be due to elastic distortion, dissipation of excess pore pressure, or soil creep.

5.7 Excavation

Soils at the site were explored to a maximum depth of 16.5 feet using a track-mounted excavator. As mentioned previously, weathered and competent conglomerate bedrock was encountered in all test pits at various depths ranging from 3 to 16.5 feet below ground surface. Table 1 presents a summary of depths to bedrock and groundwater.

Table 1. *Depth to bedrock and groundwater*.

Test Pit	Depth to Bedrock Refusal (feet below ground surface)	Depth of Seep or Groundwater (feet below ground surface)
TP-1	3	2.5
TP-2	3.5	not encountered
TP-3	4.5	not encountered
TP-4	3	not encountered
TP-5	4.5	not encountered
TP-6	6.5	not encountered
TP-7	3.5	not encountered
TP-8	16.5	15

The conglomerate was generally weathered in the top few feet, but became dense and massive with depth. If significant utilities or other excavations are designed at elevations that encounter bedrock, specialized rock-excavation techniques or blasting may be necessary. As mentioned previously, groundwater seeps were also observed during the site investigation, often at a depth coincident with the soil-to-bedrock interface.

Near-surface soils are likely classified as 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.

5.8 Dewatering

Groundwater elevation and hydrostatic pressure should be carefully considered during design of utilities, retaining walls, or other structures that require below-grade excavation. As described previously, groundwater may be encountered in the vicinity of proposed development areas. 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.

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

5.9 Lateral Earth Pressure

If retaining walls are proposed, lateral earth pressures should be carefully considered for design. Hydrostatic pressure and additional surcharge loading should also be considered. Retained material may include engineered structural backfill or undisturbed 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 undisturbed native soils and engineered structural fill consisting of imported granular fill meeting WSDOT specifications for *Gravel Backfill for Walls 9-03.12(2)* are presented in Table 1. Soil Type 1 is excluded due to the relative thin profile observed on the site.

The design parameters presented in Table 2 are valid for static loading cases. The recommended earth pressures do not include surcharge loads, dynamic loading, hydrostatic pressure, or seismic design.

Table 2. Lateral Earth Pressure Parameters for Level Backfill

Backfill Material	Equivalent Fluid Pressure for Level Backfill			Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive		
WSDOT 9-03.12(2) compacted aggregate backfill	54 pcf	33 pcf	589 pcf	135 pcf	38°
In situ undisturbed clayey SAND and Poorly Graded GRAVEL with silt and sand (Soil Type 2)	64 pcf	43 pcf	360 pcf	125 pcf	29°

** 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, 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 5.12, *Drainage*. If walls cannot be gravity drained, saturated base conditions and/or applicable hydrostatic pressures should be assumed.

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

5.10 Seismic Design Considerations

According to the *United States Geologic Survey (USGS) 2010 ASCE 7 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 3.

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 site soils may be represented by Site Class C. Based upon observed subsurface soil conditions at the site, and review of well logs and local geologic maps, site soils may be considered to be Site Class C as defined in 2012 IBC Section 1613.3.5. This site class designation indicates that some amplification of seismic energy may occur during a seismic event because of subsurface conditions. This assessment is preliminary and is based upon limited field exploration and research of existing published literature. Additional exploration would be necessary to provide soil site class information at greater depths.

Table 3. 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	0.38 g
0.2 sec Spectral Acceleration	0.89 g
1.0 sec Spectral Acceleration	0.38 g

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

Identification of specific seismic response spectra for the site 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.

5.11 Soil Liquefaction and Dynamic Settlement

According to the *Alternative Liquefaction Susceptibility Map of Clark County Washington (Washington State Department of Natural Resources, 2004)*, the site is mapped as very low susceptibility for liquefaction. Liquefaction, defined as the transformation of the behavior of a granular material from a solid to a liquid due to increased pore-water pressure and reduced effective stress, may occur when granular or non-plastic silt materials quickly compact under cyclic stresses caused by a seismic event. The effects of liquefaction may include immediate ground settlement and lateral spreading.

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

Based upon the results of the geotechnical investigation, the potential for liquefaction of shallow soils at the site is considered to be low.

5.12 Drainage

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. Drainage design in general should conform to City of Camas regulations. Finished site grading should be conducted with positive drainage away from structures. Depressions or shallow areas that may retain ponding water should be avoided. Roof drains, low-point drains, and perimeter foundation drains are recommended for structures. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from foundations into the stormwater system or approved discharge location. Concentrated discharge of water should be prohibited across slopes and water should not be diverted, routed, or allowed to flow over or across slope faces.

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. The geotechnical engineer should be consulted to provide appropriate recommendations.

5.13 Bituminous Asphalt and Portland Cement Concrete

Preliminary site plans indicate that proposed development includes private asphalt concrete driveways and parking areas. Additionally, an extension of the City of Camas' NW Camas Meadows Drive may be constructed as part of the development. Pavement section thickness should be carefully considered to provide adequate lifespan and serviceability. Pavement section design is outside the scope of this investigation; however, Columbia West can provide section design services in the future if requested. Columbia West recommends adherence to the City of Camas standards for public works construction if improvements to public roads are proposed.

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 5.14, *Wet Weather Construction Methods and Techniques*. Areas proposed for asphalt pavement construction should be prepared as described in Section 5.1, *Site Preparation and Grading*. Subgrade conditions should be evaluated and tested by a licensed geotechnical engineer or designated representative 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 250-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 D698. Areas of observed deflection or rutting during proof-roll evaluation should be excavated to a firm surface and replaced with compacted crushed aggregate.

Crushed aggregate base should be compacted and tested in accordance with the specifications outlined above. Asphalt concrete pavement should be compacted to at least 91 percent of maximum Rice density. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with Washington Department of Transportation and City of Camas specifications.

Portland cement concrete curbs should be installed in accordance with the City of Camas specifications. Aggregate base should be observed and proof-rolled in the presence of an experienced geotechnical engineer or designated representative. 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.

5.14 Wet Weather Construction Methods and Techniques

Wet weather construction often results in significant shear strength reduction and soft areas that may rut or deflect. Installation of granular working layers may be necessary to provide a firm support base and sustain construction equipment. Granular layers should consist of all-weather gravel, 4-inch by 6-inch gabion, or other similar material (6-inch maximum size with less than 5 percent passing the No. 200 sieve).

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

Construction during wet weather conditions may require increased base thickness. Road base should consist of 3”-0 or 1¼”-0 crushed aggregate and should be placed on previously stripped and structurally competent subgrade. Over-excavation may be necessary to provide a firm base upon which to place crushed aggregate. Geotextile filter fabric such as Mirafi 500X or an approved equivalent is also recommended. Crushed aggregate base should be installed in a single lift with trucks end-dumping from an advancing layer of granular fill. During extended wet periods, stripping activities may also need to be conducted from an advancing layer 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 subgrade performance.

It should be understood that wet weather construction is risky and costly. An experienced geotechnical engineer or designated representative should observe and document wet weather construction activities. Proper construction methods and techniques are critical to overall project integrity.

5.15 Erosion Control Measures

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

Site-specific erosion control measures should be implemented to address the maintenance of exposed areas. This may include silt fence, biofilter bags, straw wattles, or other suitable methods. During construction activities, exposed areas should be well-compacted and protected from erosion with visqueen, surface 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 species. Jute mesh or straw may be applied to enhance vegetation. Once established, vegetation should be properly maintained. Disturbance to existing native vegetation and surrounding organic soil should also be minimized during construction activities.

5.16 Soil Shrink/Swell Potential

Based upon laboratory analysis, subsurface soils contain as much as 63 percent by weight passing the No. 200 sieve and exhibit a plasticity index ranging from 16 to 25 percent. This indicates low to moderate potential for soil shrinking or swelling.

5.17 Utility Installation

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

Utilities should be installed in general accordance with manufacturer's recommendations. Utility trench backfill should consist of crushed aggregate or other coarse-textured, free-draining material acceptable to the client, City of Camas, and the site geotechnical engineer. 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. Even slight variations may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent QA/QC construction observation and testing to verify soil conditions are as anticipated in this report.

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

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

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

Sincerely,

COLUMBIA WEST ENGINEERING, Inc.

Daniel E. Lehto, PE, GE
Principal

DANIEL E. LEHTO
STATE OF WASHINGTON
44702
REGISTERED
PROFESSIONAL ENGINEER

6-23-15

REFERENCES

Annual Book of ASTM Standards, Soil and Rock (I), v04.08, American Society for Testing and Materials, 1999.

Beeson, M.H., Tolan, T.L., Madin, I.P., *Geologic Map of the Lake Oswego Quadrangle, Clackamas, Multnomah, and Washington Counties, Oregon*; Oregon Department of Geology and Mineral Industries; Geological Map Series GMS-59, 1989.

Clark County Maps Online (http://gis.clark.wa.gov/ccgis/mol/property.htm)

Evarts, Russel C., *Geologic Map of the Lacamas Creek Quadrangle, Clark County, Washington.* US Geological Survey, Science Investigations Map 2924, 2006.

Geomatrix Consultants, *Seismic Design Mapping, State of Oregon*, January 1995.

International Building Code: *2012 International Building Code, 2012 edition*, International Code Council, 2012.

Mundorff, M.J., *Geology and Groundwater Conditions of Clark County*, USGS Water Supply Paper 1600, 1964.

Palmer, Stephen P., Magsino, Sammantha L., Poelstra, James L., and Niggemann, Rebecca A., *Site Class Map of Clark County, Washington; Liquefaction Susceptibility Map of Clark County Washington;* Washington State Department of Natural Resources, September 2004.

Phillips, William M., *Geological Map of the Vancouver Quadrangle, Washington and Oregon,* Open File Report 87-10, Washington State Department of Natural Resources, Division of Geology and Earth Resources, 1987.

Safety and Health Regulations for Construction, 29 CFR Part 1926, Occupational Safety and Health Administration (OSHA), revised July 1, 2001.

Safety Standards for Construction Work, Part N, Excavation, Trenching and Shoring, Washington Administrative Code, Chapter 296-155, Division of Industrial Safety and Health, Washington Department of Labor and Industries, February, 1993.

United States Geologic Survey (USGS)*, 2008 NSHMP PSHA Interactive Deaggregation,* Web Application, Accessed February 2014

Web Soil Survey, Natural Resources Conservation Service, United States Department of Agriculture *2013 website* (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm.).

Wong, Ivan, et al, *Earthquake Scenario and Probabilistic Earthquake Ground Shaking Maps for the Portland, Oregon, Metropolitan Area*, IMS-16, Oregon Department of Geology and Mineral Industries, 2000.

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	Drawn: ASR		**SITE LOCATION MAP**	**FIGURE 1**
	Checked: DEL	Date: 6/12/15			
	Client: PACM, LLC	Rev	By	Date	PARKLANDS AT CAMAS MEADOWS CAMAS, WASHINGTON
	Job No.: 15153				
	CAD File: FIGURE 1				
	Scale: ~1:50,000				

NOTES:
1. SITE LOCATION: 2150 NE iONE STREET.
2. DRAWING IS NOT TO SCALE.
3. BASE MAP OBTAINED FROM BING MAPS.
4. TEST PIT LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
5. SOIL BOTING BACKFILLED WITH BENTONITE ON 6-26-2013.

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Design:	Drawn: ASR			
Checked: DEL	Date: 6/12/15			
Client: PACM, LLC	Rev	By	Date	
Job No: 15153				
CAD File: FIGURE 2				
Scale: NONE				

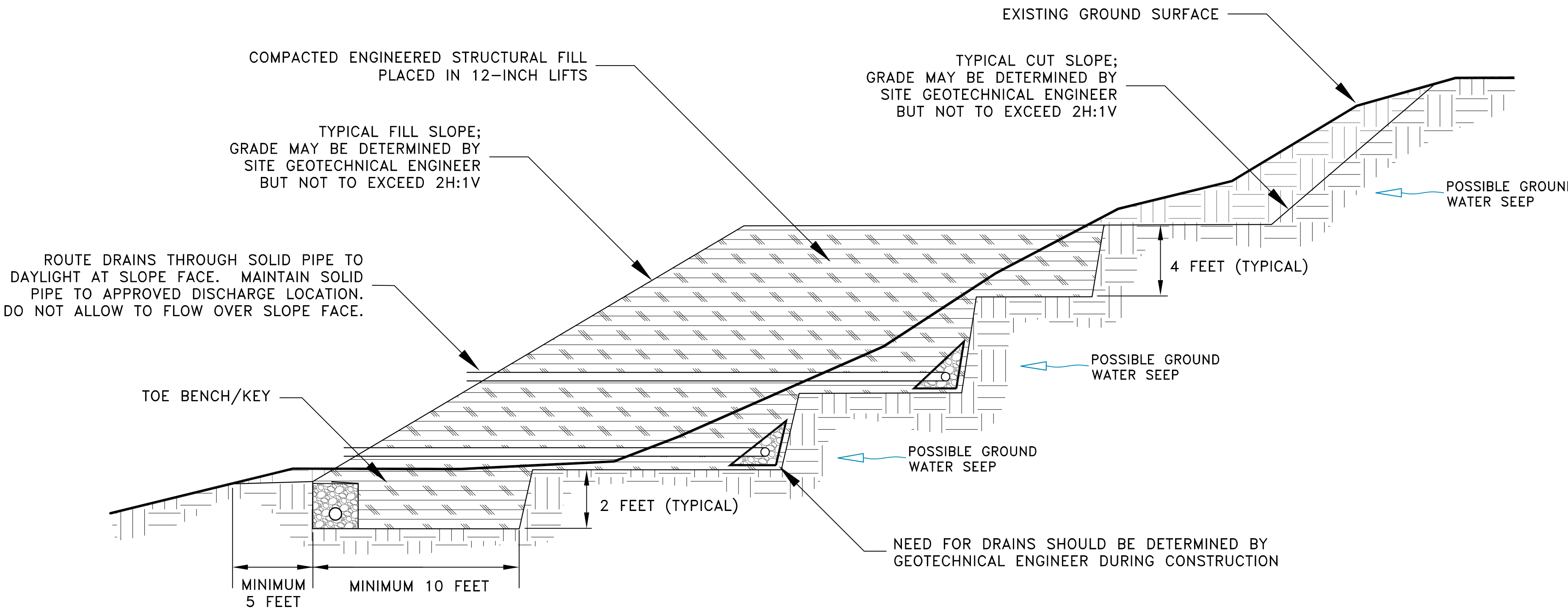
SUBSURFACE EXPLORATION LOCATION MAP

PARKLANDS AT CAMAS MEADOWS

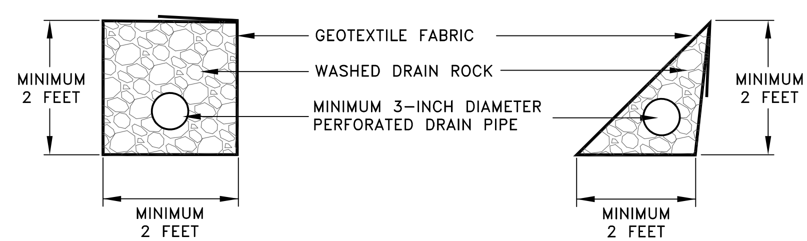
CAMAS, WASHINGTON

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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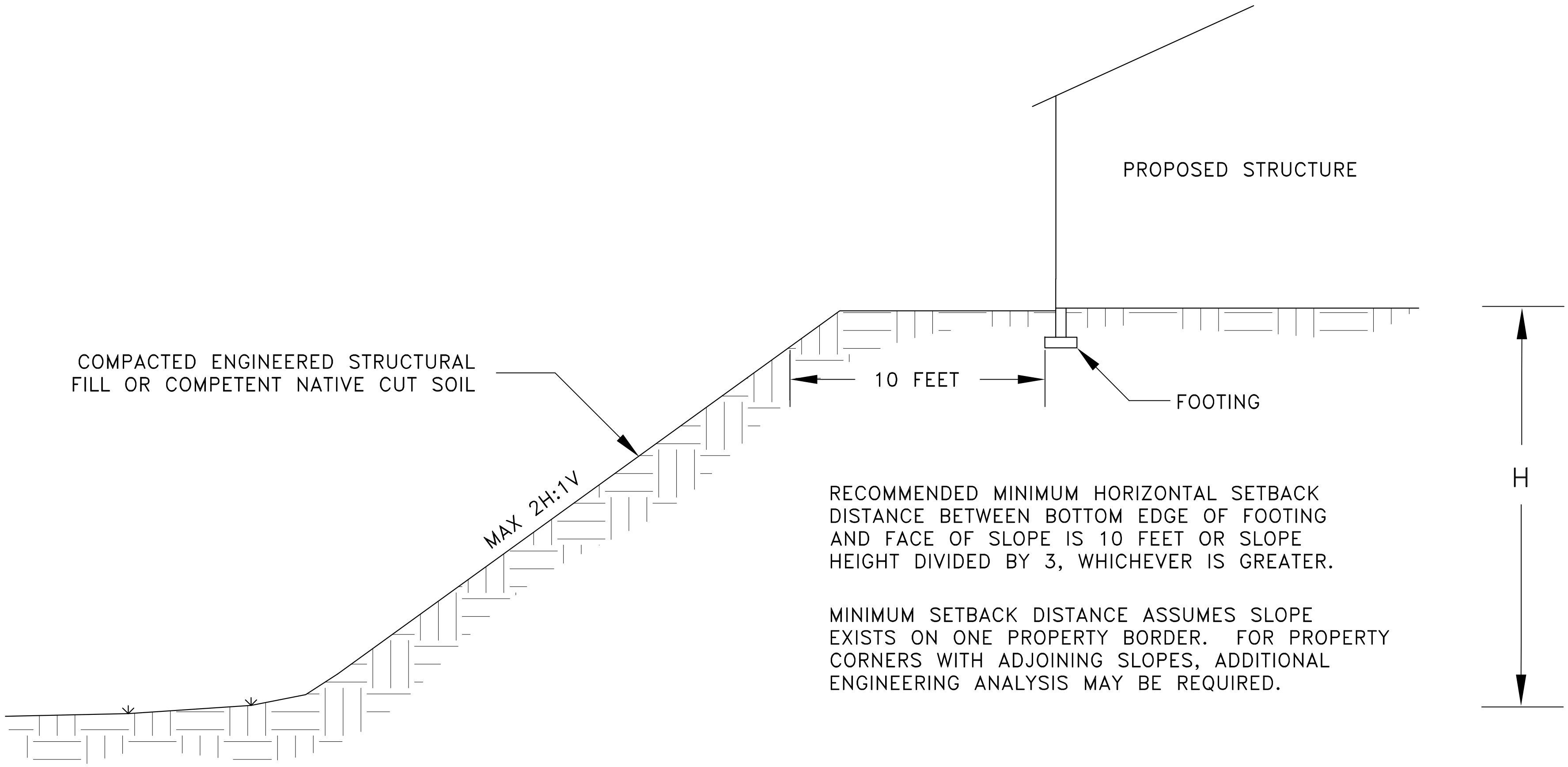
Design:	Drawn: ASR	TYPICAL CUT AND FILL SLOPE CROSS-SECTION
Checked: DEL	Date: 6/12/15	
Client: PACM, LLC	Rev / By / Date	PARKLANDS AT CAMAS MEADOWS
Job No: 15153		CAMAS, WASHINGTON
CAD File: FIGURE 3		
Scale: NONE		

FIGURE 3

TYPICAL CUT AND FILL SLOPE CROSS-SECTION



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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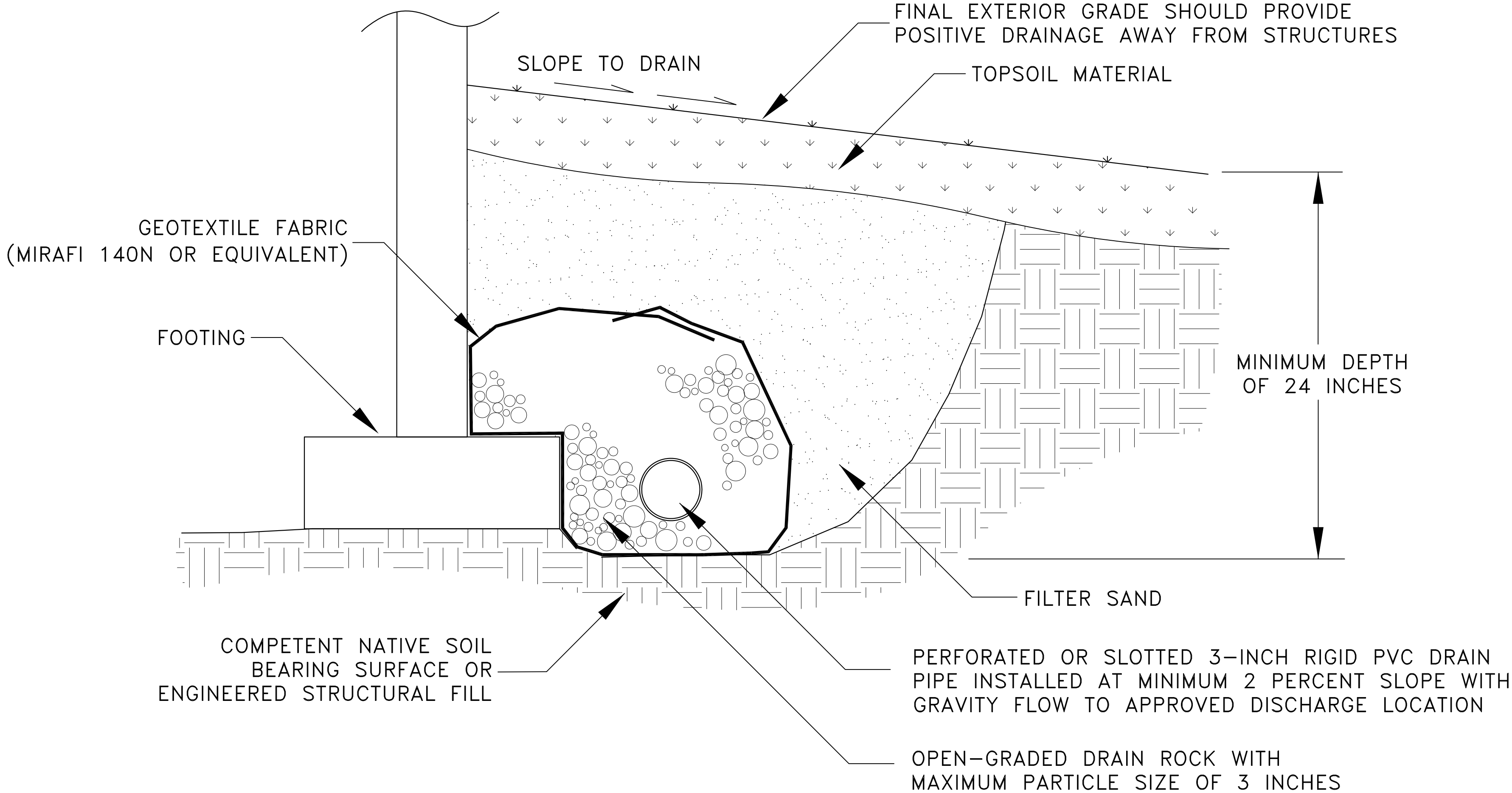
Design:	Drawn: ASR			MINIMUM FOUNDATION SLOPE SETBACK DETAIL	FIGURE 4
Checked: DEL	Date: 06/12/15				
Client: PACM, LLC	Rev	By	Date	PARKLANDS AT CAMAS MEADOWS CAMAS, WASHINGTON	
Job No: 15153					
CAD File: FIGURE 4					
Scale: NONE					

TYPICAL PERIMETER FOOTING DRAIN DETAIL



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

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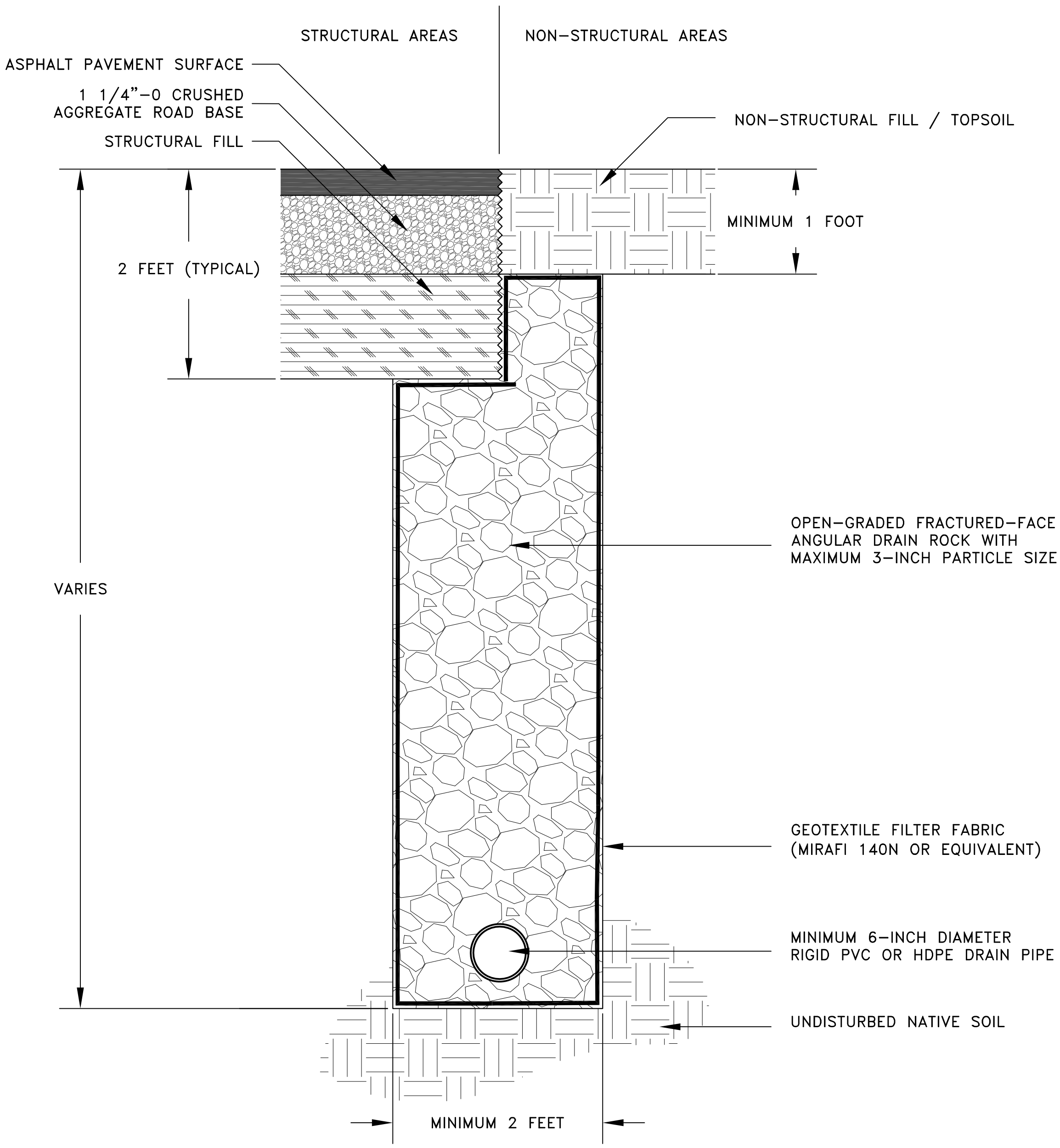
Design:	Drawn: ASR			
Checked: DEL	Date: 6/12/15			
Client: PACM, LLC	Rev	By	Date	
Job No: 15153				
CAD File: FIGURE 5				
Scale: NONE				

TYPICAL PERIMETER FOOTING DRAIN DETAIL

PARKLANDS AT CAMAS MEADOWS
CAMAS, WASHINGTON

FIGURE 5

TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL



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

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Design:	Drawn: ASR			
Checked: DEL	Date: 06/12/15			
Client: PACM, LLC	Rev	By	Date	
Job No: 15153				
CAD File: FIGURE 6				
Scale: NONE				

TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL

PARKLANDS AT CAMAS MEADOWS
CAMAS, WASHINGTON

FIGURE 6

APPENDIX A
ANALYTICAL LABORATORY TEST RESULTS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Parklands at Camas Meadows Camas, Washington	Mr. Aaron Barr & Mr. Kevin Deford Parklands at Camas Meadows, LLC PO Box 61962 Vancouver, WA 98666	15153	S15-359
		REPORT DATE: 06/16/15	FIELD ID: TP3.1
		DATE SAMPLED: 06/04/15	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy SILT	Test Pit TP-03 depth = 2 feet	ML, Sandy Silt

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

LABORATORY TEST DATA

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

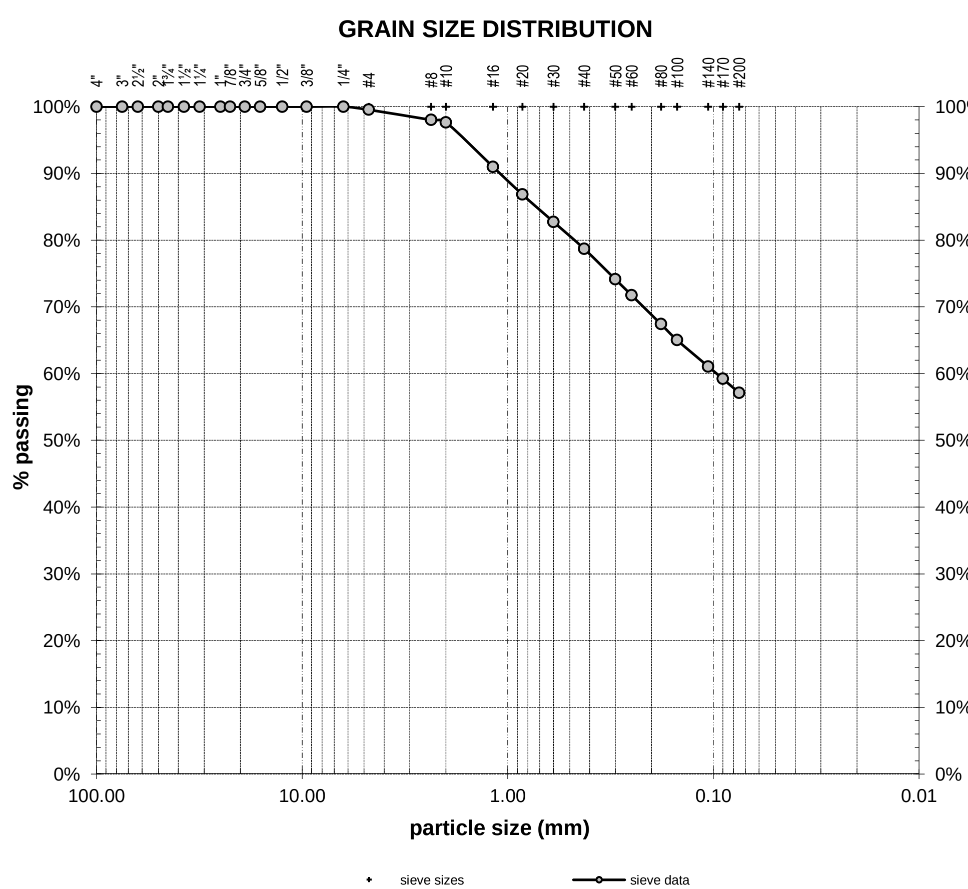
ADDITIONAL DATA

initial dry mass (g) =	158.3		
as-received moisture content =	36.4%	coefficient of curvature, C_C =	n/a
liquid limit =	44	coefficient of uniformity, C_U =	n/a
plastic limit =	28	effective size, $D_{(10)}$ =	n/a
plasticity index =	16	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	0.097 mm

SIEVE DATA

% gravel = 0.5%
% sand = 42.4%
% 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		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	99.5%			
SAND	#8	2.36		98.0%		
	#10	2.00	97.6%			
	#16	1.18		91.0%		
	#20	0.850	86.8%			
	#30	0.600		82.7%		
	#40	0.425	78.7%			
	#50	0.300		74.1%		
	#60	0.250	71.7%			
	#80	0.180		67.4%		
	#100	0.150	65.0%			
	#140	0.106		61.1%		
	#170	0.090		59.2%		
	#200	0.075	57.1%			



DATE TESTED	TESTED BY
06/08/15	BTT/MJR

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: 06/16/15	FIELD ID: TP3.1
		DATE SAMPLED: 06/04/15	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy SILT	Test Pit TP-03 depth = 2 feet	ML, Sandy Silt

LABORATORY TEST DATA

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

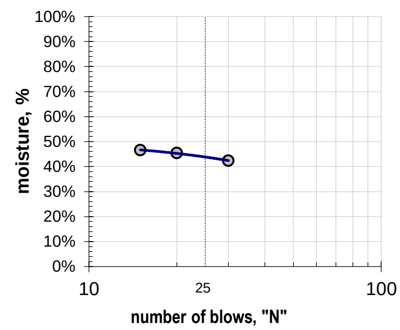
ATTERBERG LIMITS

liquid limit = 44
plastic limit = 28
plasticity index = 16

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	33.30	33.31	33.93	
dry soil + pan weight, g =	29.59	29.37	29.72	
pan weight, g =	20.83	20.70	20.68	
N (blows) =	30	20	15	
moisture, % =	42.4 %	45.4 %	46.6 %	

LIQUID LIMIT



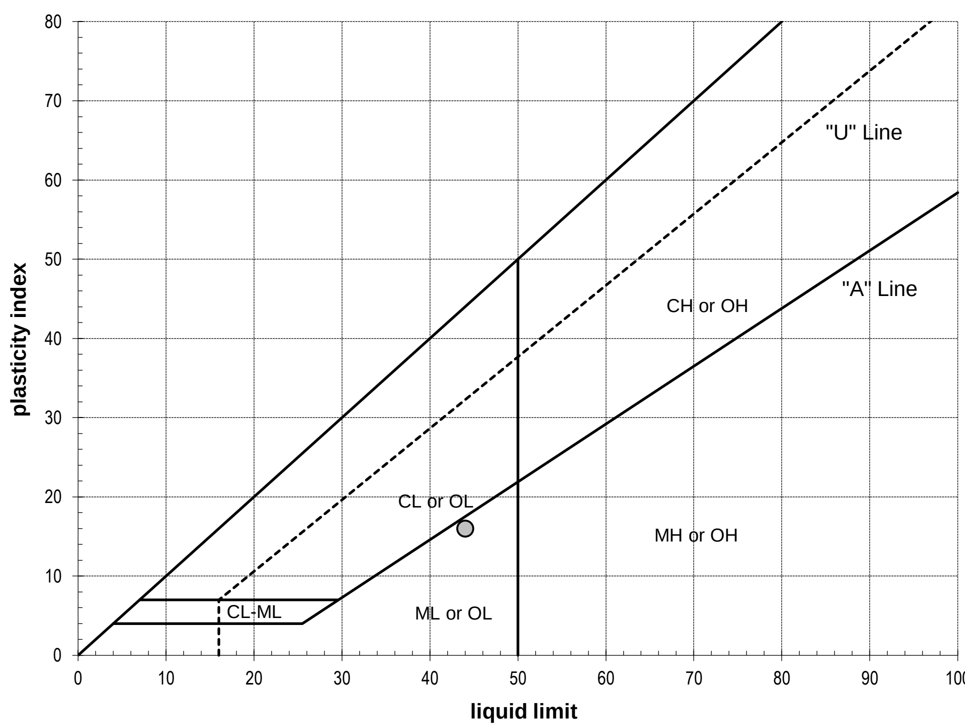
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.85	27.97		
dry soil + pan weight, g =	26.28	26.35		
pan weight, g =	20.69	20.50		
moisture, % =	28.1 %	27.7 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.5%
% sand = 42.4%
% silt and clay = 57.1%
% silt = n/a
% clay = n/a
moisture content = 36.4%

DATE TESTED	TESTED BY
06/15/15	MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Parklands at Camas Meadows Camas, Washington	Mr. Aaron Barr & Mr. Kevin Deford Parklands at Camas Meadows, LLC PO Box 61962 Vancouver, WA 98666	15153	S15-360
		REPORT DATE: 06/16/15	FIELD ID: TP6.1
		DATE SAMPLED: 06/04/15	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Fat CLAY	Test Pit TP-06 depth = 2.5 feet	CH, Sandy Fat Clay

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(14)

LABORATORY TEST DATA

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

ADDITIONAL DATA

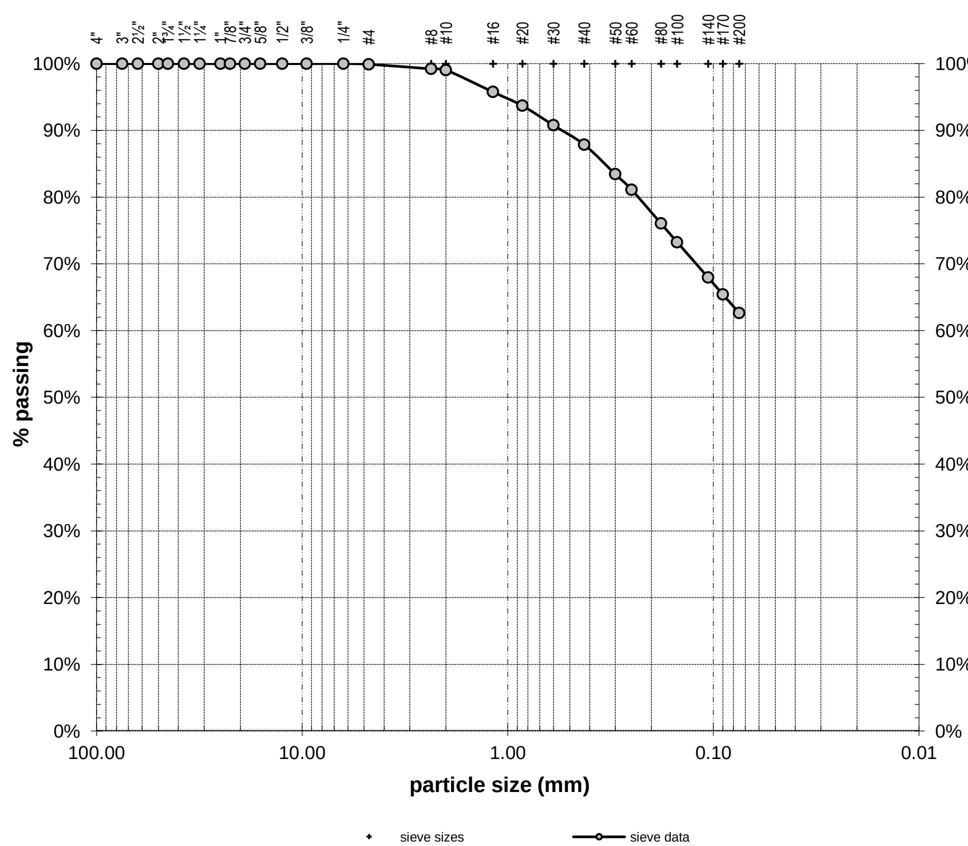
initial dry mass (g) =	173.4		
as-received moisture content =	32.3%	coefficient of curvature, C_C =	n/a
liquid limit =	51	coefficient of uniformity, C_U =	n/a
plastic limit =	26	effective size, $D_{(10)}$ =	n/a
plasticity index =	25	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.1%
% sand = 37.3%
% silt and clay = 62.6%

	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	99.9%			
SAND	#8	2.36		99.2%		
	#10	2.00	99.0%			
	#16	1.18		95.7%		
	#20	0.850	93.7%			
	#30	0.600		90.8%		
	#40	0.425	87.9%			
	#50	0.300		83.4%		
	#60	0.250	81.1%			
	#80	0.180		76.1%		
	#100	0.150	73.2%			
	#140	0.106		67.9%		
	#170	0.090		65.4%		
	#200	0.075	62.6%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
06/08/15	BTT/MJR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Parklands at Camas Meadows Camas, Washington	Mr. Aaron Barr & Mr. Kevin Deford Parklands at Camas Meadows, LLC PO Box 61962 Vancouver, WA 98666	15153	S15-360
		REPORT DATE: 06/16/15	FIELD ID: TP6.1
		DATE SAMPLED: 06/04/15	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Fat CLAY	Test Pit TP-06 depth = 2.5 feet	CH, Sandy Fat Clay

LABORATORY TEST DATA

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

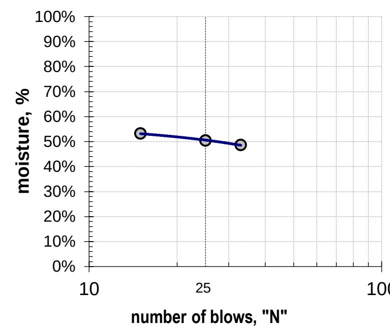
ATTERBERG LIMITS

liquid limit =	51
plastic limit =	26
plasticity index =	25

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.63	34.09	37.32	
dry soil + pan weight, g =	30.07	29.61	31.60	
pan weight, g =	20.69	20.73	20.85	
N (blows) =	33	25	15	
moisture, % =	48.6 %	50.5 %	53.2 %	

LIQUID LIMIT



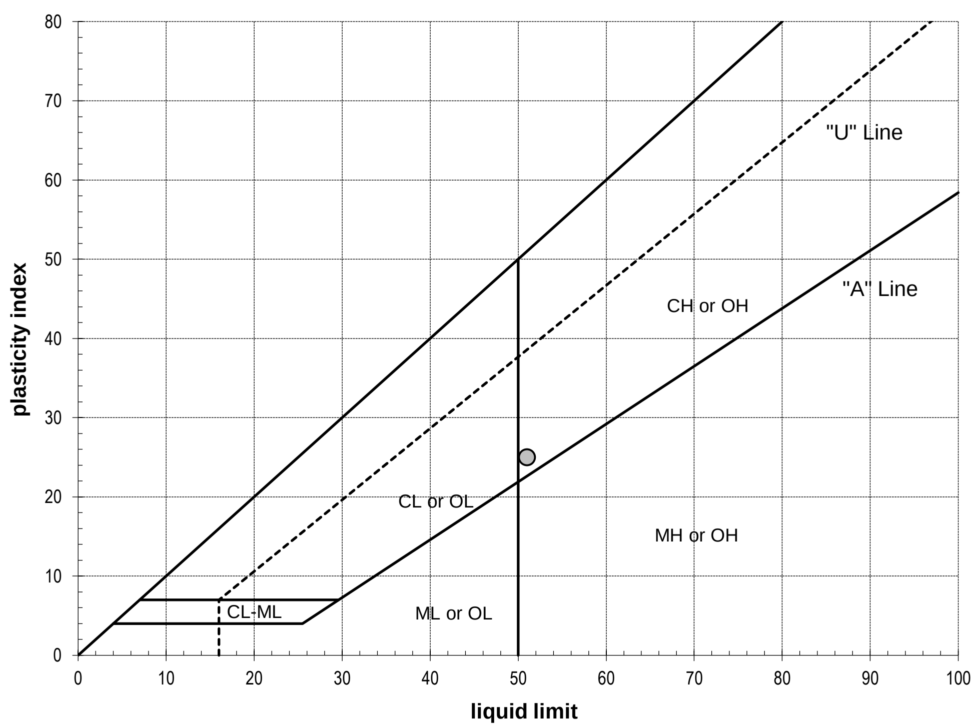
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.64	27.63		
dry soil + pan weight, g =	26.18	26.21		
pan weight, g =	20.69	20.82		
moisture, % =	26.6 %	26.4 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.1%
% sand =	37.3%
% silt and clay =	62.6%
% silt =	n/a
% clay =	n/a
moisture content =	32.3%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Parklands at Camas Meadows Camas, Washington	Mr. Aaron Barr & Mr. Kevin Deford Parklands at Camas Meadows, LLC PO Box 61962 Vancouver, WA 98666	15153	S15-361
		REPORT DATE: 06/16/15	FIELD ID: TP8.2
		DATE SAMPLED: 06/04/15	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey SAND	Test Pit TP-08 depth = 8 feet	SC, Clayey Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-2-7(0)

LABORATORY TEST DATA

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

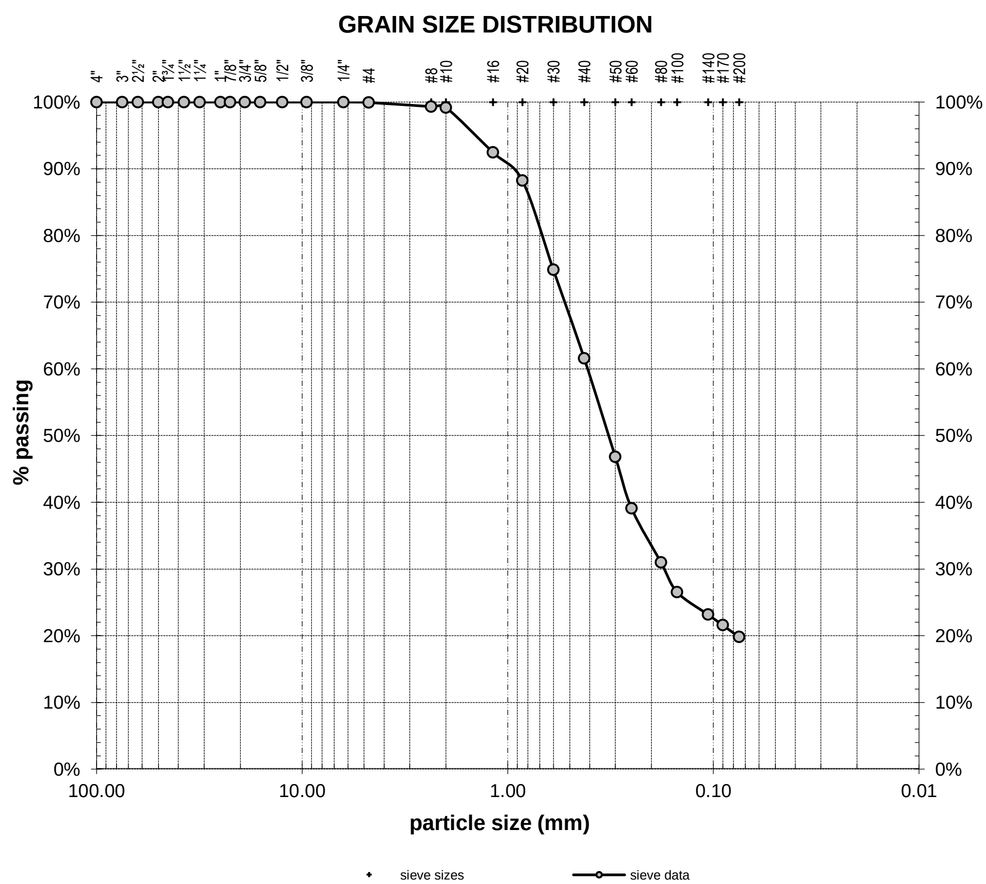
ADDITIONAL DATA

initial dry mass (g) =	151.9		
as-received moisture content =	42.6%	coefficient of curvature, C_C =	n/a
liquid limit =	43	coefficient of uniformity, C_U =	n/a
plastic limit =	25	effective size, $D_{(10)}$ =	n/a
plasticity index =	18	$D_{(30)}$ =	0.173 mm
fineness modulus =	n/a	$D_{(60)}$ =	0.410 mm

SIEVE DATA

% gravel = 0.1%
% sand = 80.1%
% silt and clay = 19.8%

	SIEVE SIZE		PERCENT PASSING			
			SIEVE		SPECS	
	US	mm	act.	interp.	max	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	99.9%			
SAND	#8	2.36		99.3%		
	#10	2.00	99.2%			
	#16	1.18		92.4%		
	#20	0.850	88.3%			
	#30	0.600		74.8%		
	#40	0.425	61.6%			
	#50	0.300		46.8%		
	#60	0.250	39.1%			
	#80	0.180		31.0%		
	#100	0.150	26.5%			
	#140	0.106		23.2%		
	#170	0.090		21.6%		
	#200	0.075	19.8%			



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Parklands at Camas Meadows Camas, Washington	Mr. Aaron Barr & Mr. Kevin Deford Parklands at Camas Meadows, LLC PO Box 61962 Vancouver, WA 98666	15153	S15-361
		REPORT DATE 06/16/15	**FIELD ID** TP8.2
		DATE SAMPLED 06/04/15	**SAMPLED BY** HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey SAND	Test Pit TP-08 depth = 8 feet	SC, Clayey Sand

LABORATORY TEST DATA

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

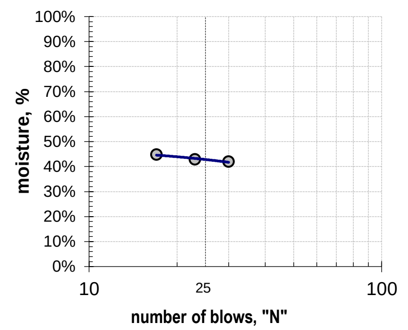
ATTERBERG LIMITS

liquid limit = 43
plastic limit = 25
plasticity index = 18

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	39.55	37.83	35.98	
dry soil + pan weight, g =	33.98	32.70	31.23	
pan weight, g =	20.69	20.73	20.63	
N (blows) =	30	23	17	
moisture, % =	41.9 %	42.9 %	44.8 %	

LIQUID LIMIT



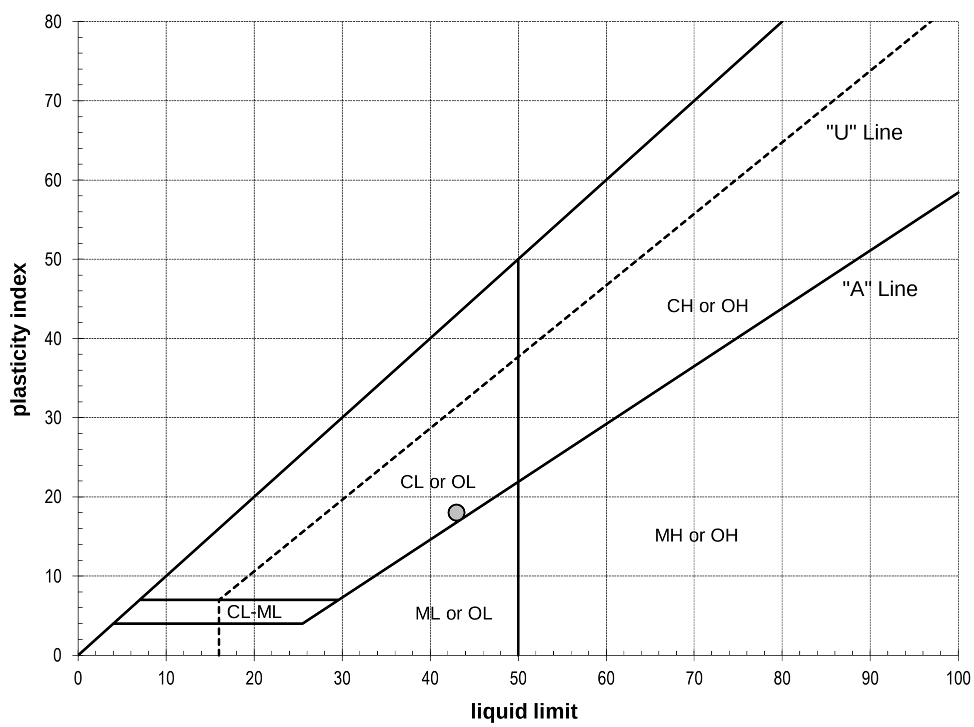
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.97	28.28		
dry soil + pan weight, g =	26.54	26.79		
pan weight, g =	20.74	20.77		
moisture, % =	24.7 %	24.8 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.1%
% sand = 80.1%
% silt and clay = 19.8%
% silt = n/a
% clay = n/a
moisture content = 42.6%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Parklands at Camas Meadows Camas, Washington	Mr. Aaron Barr & Mr. Kevin Deford Parklands at Camas Meadows, LLC PO Box 61962 Vancouver, WA 98666	15153	S15-362
		REPORT DATE	**FIELD ID**
		06/16/15	TP8.3
		DATE SAMPLED	**SAMPLED BY**
		06/04/15	HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Poorly graded GRAVEL with Silt and Sand	Test Pit TP-08 depth = 12 feet	GP-GM, Poorly graded gravel with silt and sand
SPECIFICATIONS		**AASHTO SOIL TYPE**
none		A-2-7(0)

LABORATORY TEST DATA

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

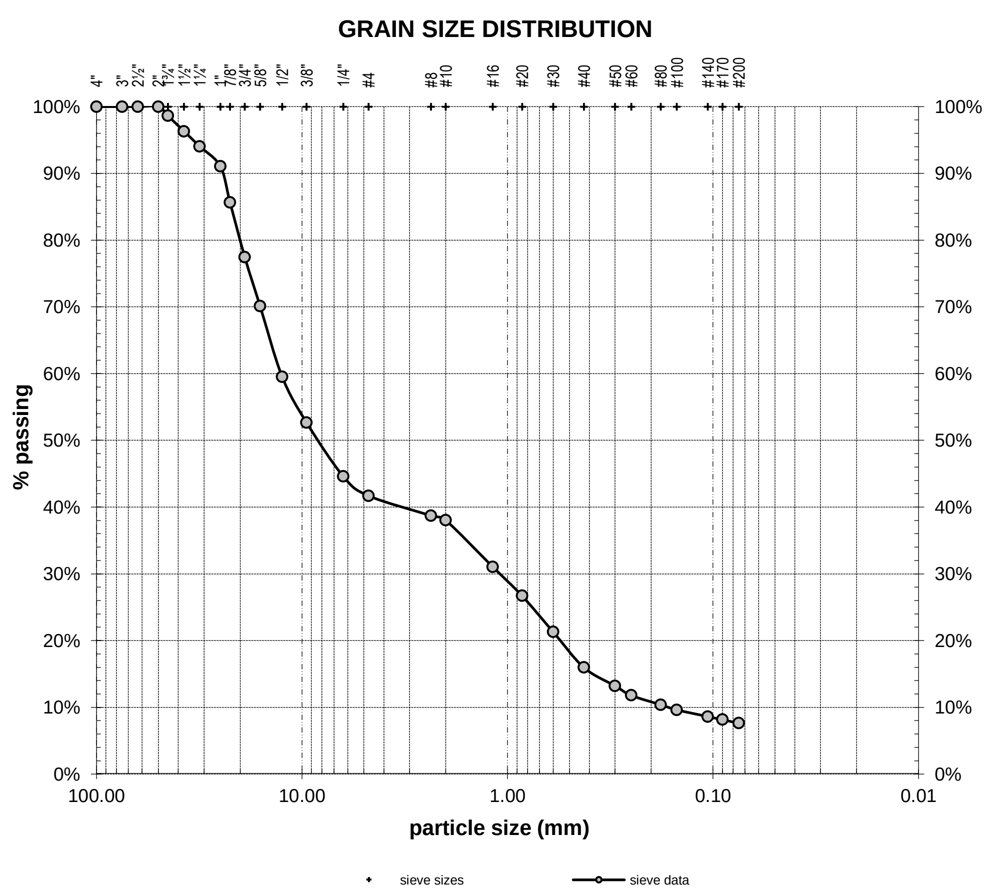
ADDITIONAL DATA

initial dry mass (g) =	3089.1		
as-received moisture content =	33.2%	coefficient of curvature, C_C =	0.57
liquid limit =	46	coefficient of uniformity, C_U =	76.93
plastic limit =	28	effective size, $D_{(10)}$ =	0.164 mm
plasticity index =	18	$D_{(30)}$ =	1.091 mm
fineness modulus =	n/a	$D_{(60)}$ =	12.637 mm

SIEVE DATA

% gravel = 58.3%
% sand = 34.1%
% silt and clay = 7.6%

	SIEVE SIZE		PERCENT PASSING			
			SIEVE		SPECS	
	US	mm	act.	interp.	max	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		98.6%		
	1.50"	37.5		96.3%		
	1.25"	31.5		94.0%		
	1.00"	25.0	91.1%			
	7/8"	22.4		85.6%		
	3/4"	19.0	77.5%			
	5/8"	16.0		70.1%		
	1/2"	12.5	59.5%			
	3/8"	9.50	52.6%			
	1/4"	6.30	44.6%			
	#4	4.75	41.7%			
SAND	#8	2.36		38.7%		
	#10	2.00	38.0%			
	#16	1.18		31.0%		
	#20	0.850	26.7%			
	#30	0.600		21.3%		
	#40	0.425	16.0%			
	#50	0.300		13.2%		
	#60	0.250	11.8%			
	#80	0.180		10.4%		
	#100	0.150	9.6%			
	#140	0.106		8.6%		
	#170	0.090		8.1%		
	#200	0.075	7.6%			



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Parklands at Camas Meadows Camas, Washington	Mr. Aaron Barr & Mr. Kevin Deford Parklands at Camas Meadows, LLC PO Box 61962 Vancouver, WA 98666	15153	S15-362
		REPORT DATE: 06/16/15	FIELD ID: TP8.3
		DATE SAMPLED: 06/04/15	SAMPLED BY: HDG

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Poorly graded GRAVEL with Silt and Sand	Test Pit TP-08 depth = 12 feet	GP-GM, Poorly graded gravel with silt and sand

LABORATORY TEST DATA

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

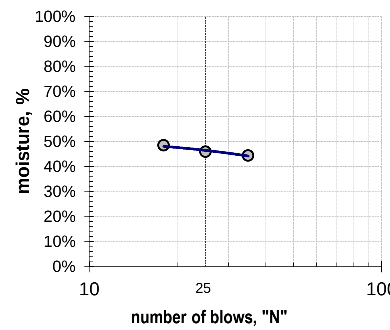
ATTERBERG LIMITS

liquid limit =	46
plastic limit =	28
plasticity index =	18

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	39.57	37.55	37.51	
dry soil + pan weight, g =	33.76	32.29	32.08	
pan weight, g =	20.69	20.82	20.87	
N (blows) =	35	25	18	
moisture, % =	44.5 %	45.9 %	48.4 %	

LIQUID LIMIT



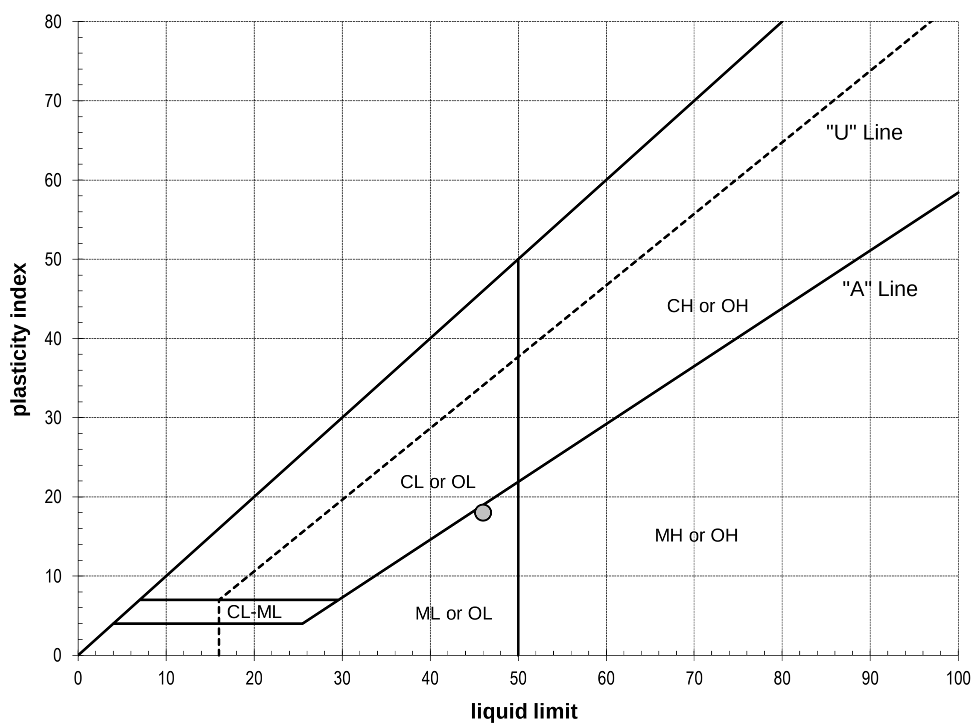
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.57	27.51		
dry soil + pan weight, g =	26.11	26.06		
pan weight, g =	20.82	20.80		
moisture, % =	27.6 %	27.6 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	58.3%
% sand =	34.1%
% silt and clay =	7.6%
% silt =	n/a
% clay =	n/a
moisture content =	33.2%

DATE TESTED	TESTED BY
06/15/15	MJR

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
The Parklands at Camas Meadows	Parklands, LLC		15153	TP-1
PROJECT LOCATION: Camas, Washington	CONTRACTOR:	EQUIPMENT: Excavator	ENGINEER: HDG	DATE: 6/4/15
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 186	GROUNDWATER DEPTH: 2.5 ft bgs	START TIME: 1030	FINISH TIME: 1035

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 10 to 12 inches of topsoil and root zone material.					
				GP-GM		Poorly graded GRAVEL with silt and sand, dense, wet to saturated. Represents weathered conglomerate bedrock. Difficult excavation. 6" to 8" weathered cobbles observed.					
						Refusal at 3.0 feet, competent conglomerate bedrock encountered. Bottom of test pit at 3.0 feet. Groundwater encountered at 2.5 feet.					
5											
10											
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
The Parklands at Camas Meadows	Parklands, LLC		15153	TP-2
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**ENGINEER**	**DATE**
Camas, Washington		Excavator	HDG	6/4/15
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	190	Not encountered.	1000	1015

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 topsoil and root zone material.					
				GP-GM		Poorly graded GRAVEL with silt and sand, dense, wet to saturated. Represents weathered conglomerate bedrock. Difficult excavation. 6" to 8" weathered cobbles observed.					
						Refusal at 3.5 feet, competent conglomerate bedrock encountered. Bottom of test pit at 3.5 feet. Groundwater not encoutered.					
5											
10											
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
The Parklands at Camas Meadows	Parklands, LLC		15153	TP-3
PROJECT LOCATION: Camas, Washington	CONTRACTOR:	EQUIPMENT: Excavator	ENGINEER: HDG	DATE: 6/4/15
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 208	GROUNDWATER DEPTH: Not encountered.	START TIME: 1115	FINISH TIME: 1130

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 16 to 18 inches of topsoil and root zone material.					
	TP3.1		A-7-6(7)	CL		Brown sandy CLAY, moist, medium stiff, fines are medium plasticity.	36.4	57.1	44	16	
				GP-GM		Poorly graded GRAVEL with silt and sand, dense, wet to saturated, gravels are rounded to subrounded. Represents weathered conglomerate bedrock. Difficult excavation.					
5						Refusal at 4.5 feet, competent conglomerate bedrock encountered. Bottom of test pit at 4.5 feet. Groundwater not encoutered.					
10											
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
The Parklands at Camas Meadows	Parklands, LLC		15153	TP-4
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington		Excavator	HDG	6/4/15
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	214	Not encountered.	1150	1200

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 16 inches of topsoil and root zone material.					
				CL		Brown sandy CLAY, moist, medium stiff, fines are medium plasticity.					
				GP-GM		Poorly graded GRAVEL with silt and sand, dense, wet to saturated, gravels are rounded to subrounded. Represents weathered conglomerate bedrock.					
						Refusal at 3.0 feet, competent conglomerate bedrock encountered. Bottom of test pit at 3.0 feet. Groundwater not encountered.					
5											
10											
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
The Parklands at Camas Meadows	Parklands, LLC		15153	TP-5
PROJECT LOCATION: Camas, Washington	CONTRACTOR:	EQUIPMENT: Excavator	ENGINEER: HDG	DATE: 6/4/15
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 228	GROUNDWATER DEPTH: Not encountered.	START TIME: 1245	FINISH TIME: 1255

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 topsoil and root zone material.					
				CL		Brown sandy CLAY, moist, medium stiff, fines are medium plasticity.					
				GP-GM		Poorly graded GRAVEL with silt and sand, dense, wet to saturated, gravels are rounded to subrounded. Represents weathered conglomerate bedrock. Difficult excavation.					
5						Refusal at 4.5 feet, competent conglomerate bedrock encountered. Bottom of test pit at 4.5 feet. Groundwater not encoutered.					
10											
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
The Parklands at Camas Meadows	Parklands, LLC		15153	TP-6
PROJECT LOCATION: Camas, Washington	CONTRACTOR:	EQUIPMENT: Excavator	ENGINEER: HDG	DATE: 6/4/15
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 210	GROUNDWATER DEPTH: Not encountered.	START TIME: 1215	FINISH TIME: 1230

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 to 14 inches of topsoil and root zone material.					
	TP6.1		A-7-6(14)	CH		Brown sandy FAT CLAY, moist, medium stiff, fines are moderate to high plasticity.	32.3	62.6	51	25	
5				GP-GM		Poorly graded GRAVEL with silt and sand, dense, wet to saturated, gravels are rounded to subrounded. Represents weathered conglomerate bedrock. Difficult excavation. 6" to 8" weathered cobbles observed.					
						Refusal at 6.5 feet, competent conglomerate bedrock encountered. Bottom of test pit at 6.5 feet. Groundwater not encountered.					
10											
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
The Parklands at Camas Meadows	Parklands, LLC		15153	TP-7
PROJECT LOCATION: Camas, Washington	CONTRACTOR:	EQUIPMENT: Excavator	ENGINEER: HDG	DATE: 6/4/15
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 218	GROUNDWATER DEPTH: Not encountered.	START TIME: 810	FINISH TIME: 815

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 topsoil and root zone material.					
				GP-GM		Poorly graded GRAVEL with silt and sand, dense, wet to saturated, gravels are rounded to subrounded. Represents weathered conglomerate bedrock. Difficult excavation. 6" to 8" weathered cobbles observed.					
5						Refusal at 3.5 feet, competent conglomerate bedrock encountered. Bottom of test pit at 3.5 feet. Groundwater not encoutered.					
10											
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
The Parklands at Camas Meadows	Parklands, LLC		15153	TP-8
PROJECT LOCATION: Camas, Washington	CONTRACTOR:	EQUIPMENT: Excavator	ENGINEER: HDG	DATE: 6/4/15
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 210	GROUNDWATER DEPTH: Not encountered	START TIME: 830	FINISH TIME: 900

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 16 to 18 inches of topsoil and root zone material.					
				CL		Brown sandy CLAY, moist, medium stiff, fines are medium plasticity.					
5				SC		Light brown clayey SAND, moist, dense, lightly cemented, fines are medium plasticity.					
	TP8.2		A-2-7(0)				42.6	19.8	43	18	
						Difficult excavation.					
10				GP-GM		Poorly graded GRAVEL with silt and sand, dense, saturated, gravels are subrounded to rounded.					
	TP8.3		A-2-7(0)				33.2	7.6	46	18	
15						Groundwater seep.					
						Refusal at 16.5 feet, competent conglomerate bedrock encountered.. Bottom of test pit at 16.5 feet. Groundwater encoutered at 15.5 feet.					
20											

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			

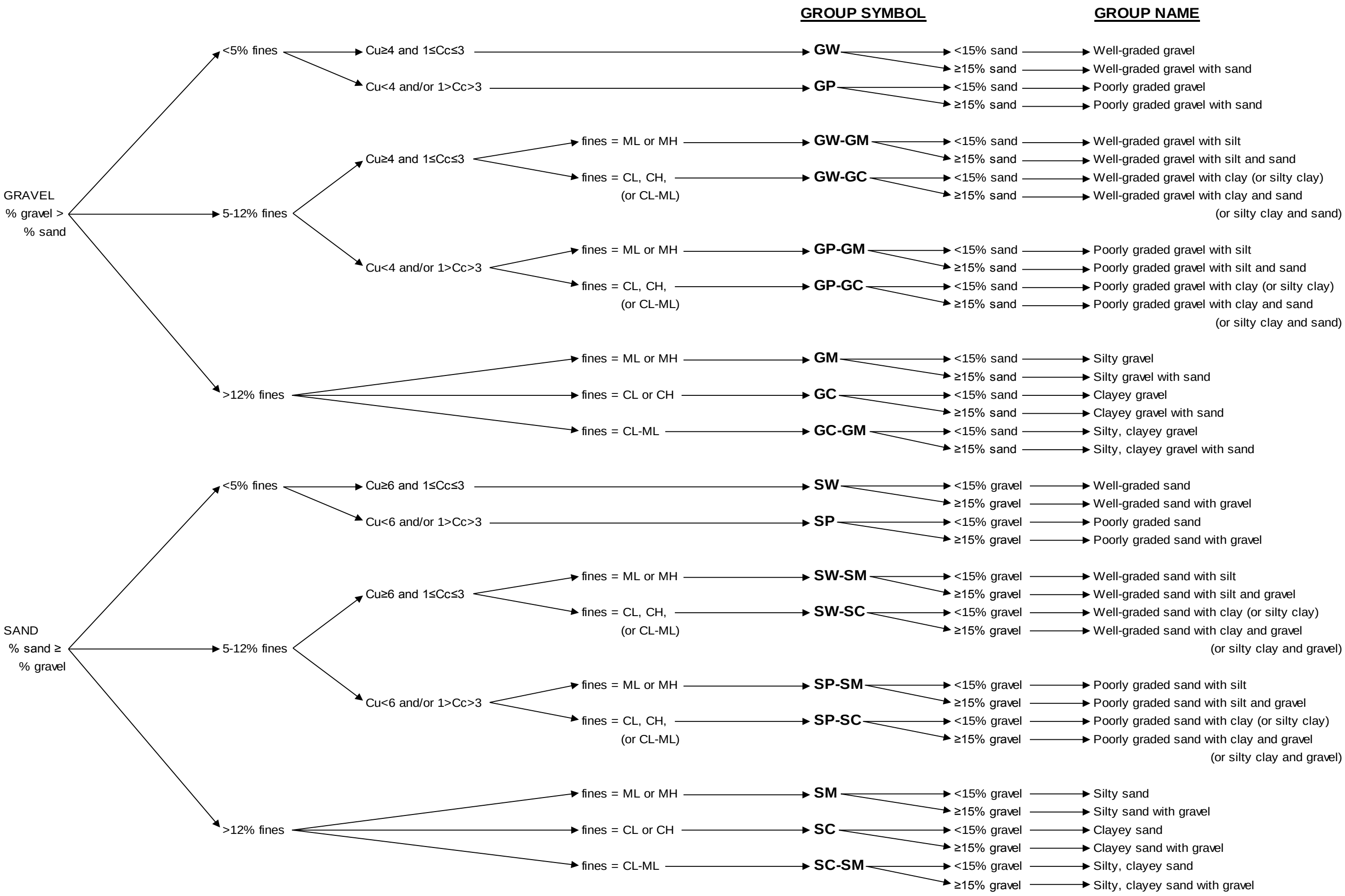
Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

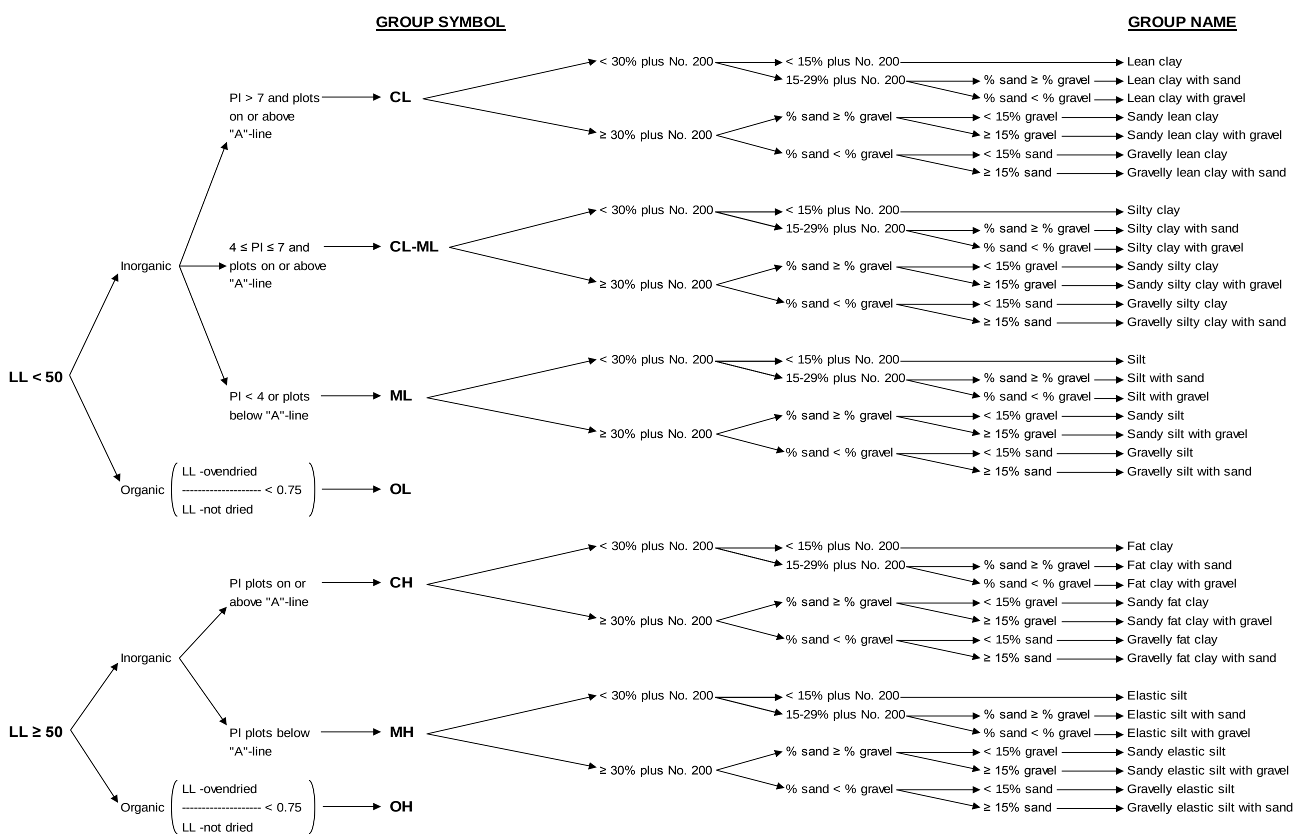
General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 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



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

PHOTO LOG
Parklands at Camas Meadows
Camas, Washington

Shallow conglomerate bedrock in Test Pit TP-1

Shallow hyaoclastic sandstone bedrock in Test Pit TP-3

Shallow groundwater near Test Pit TP-1

Deeper conglomerate bedrock encountered in Test Pit TP-8

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

Date: June 23, 2015
Project: Parklands at Camas Meadows
Camas, Washington

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

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

Report Conclusions and Preliminary Nature

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

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

Additional Investigation and Construction QA/QC

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

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

Collected Samples

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

Report Contents

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

Report Limitations for Contractors

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

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Consultant Responsibility

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