EXHIBIT 29

Critical Areas Report

Green Mountain – Pods B3 & E1

Camas, Washington

September 27, 2016

Geotechnical ■ Environmental ■ Special Inspections

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CRITICAL AREAS REPORT
GREEN MOUNTAIN – PODS B3 & E1
CAMAS, WASHINGTON

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Site Location:	**2817 NE Ingle Road** **Parcel No. 172557000** **Camas, Washington**
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Date Prepared:	**September 27, 2016**

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1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by CLB Washington Option Solutions, LLC to conduct a critical areas report for proposed development within residential Pods B3 and E1 of the Green Mountain Mixed Use development located at 2817 NE Ingle Road. The purpose of the report was to observe and assess critical areas and accompanying 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 March 23, 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 7.0, *Conclusion and Limitations*, and Appendix E.

1.1 General Site Information

As indicated on Figures 1 and 2, the Green Mountain site is located at 2817 NE Ingle Road, north of the intersection with NE Goodwin Road in Camas, Washington on what was previously the Green Mountain Golf Course. The study area for this investigation comprises a portion of the proposed development, identified as Pods B3 and E1, that includes a portion of parcels 172557000 and 986037308. Additional test pit exploration, conducted for construction estimation purposes, was also conducted in portions of the Phases 1G-I area. The regulatory jurisdictional agency is the City of Camas, Washington. The approximate latitude and longitude are N 45° 39’ 00” and W 122° 27’ 25”, and the legal description is a portion of the SE ¼ of Section 17, T2N, R3E, a portion of the NE ¼ of Section 20, T2N, R3E, and a portion of the NW ¼ of Section 21, T2N, R3E, Willamette Meridian.

1.2 Proposed Development

Review of preliminary civil site plans provided by Olson Engineering indicates that proposed development will consist of approximately 13 residential lots in Pod B3 and approximately 13 residential lots in Pod E1. Columbia West understands that these residential pods, as well as Phases 1G-I, and Phase 2 are planned to be constructed in a phased approach coincident with construction of Phase 1A through 1F which is currently under construction as of June, 2016.

This report provides geotechnical analysis and recommendations for site development of Pods B3 and E1 which are located within and adjacent to a mapped geologic hazard area (slopes greater than 15 percent). Subsurface information from test pits excavated in areas of Phases 1G-1I is also included. Figure 2 depicts the approximate extent and location of Pods B3 and E1. 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-eastern portion of the Portland/Vancouver Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide. Specifically, the site is located at the foot of the south-facing slope of a basalt cinder cone known as Green Mountain.

According to the *Geologic Map of the Lacamas Creek Quadrangle, Clark County, Washington, (US Geological Survey, Science Investigations Map 2924, 2006)*, the study area lies near the contact of three geologic units: an unnamed, Pleistocene to Pliocene unconsolidated to semi-consolidated, pebble to cobble conglomerate (QTc); the Conglomerate member of the Troutdale Formation (Ttfc), a Pliocene to Miocene semi-consolidated to well consolidated pebble to cobble conglomerate with lenses of arkosic to basaltic sandstone; and the Basaltic Andesite of Green Mountain (Qbgm), a Pleistocene-aged cinder cone comprised of olivine-phyric, nonscoriaceous, platy lava.

Both conglomerate formations are lithologically similar to one another, differing primarily in age of emplacement, degree of weathering, and the presence of hyaloclastite interbeds in the Troutdale Formation. Previously published geologic mapping has identified the unnamed conglomerate unit as the Troutdale Formation. The cinder cone that formed Green Mountain represents the northern portion of the Quaternary Boring Volcanic Field which intruded vertically through the conglomeratic members, solidified, and weathered in place. Although the cinder cone is mapped within the boundaries of the subject parcel, subsurface investigation indicates that the proposed developed portions of the study area are underlain by the two conglomerate formations and may not intersect with intact bedrock of the cinder cone.

The *Web Soil Survey (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2013 Website)* indicates the site is underlain by two soil types. Dollar loam soils are mapped on the southern portion of the parcel and generally overlies the conglomerate bedrock, while Olympic stony clay loam soils are mapped in the northern areas and are underlain by the basaltic andesite bedrock. Although actual on-site soils may vary from the broad USDA descriptions, Olympic series soils generally consist of fine- to medium-textured, generally well drained, slowly permeable silts and clays with varying plasticity. They are generally moisture sensitive, have a high water capacity, moderate shrink-swell potential, and slight erosion hazard based primarily upon slope grade. Dollar loam soils are generally fine-textured soils that form in old alluvium on terraces. These soils are moderately well drained, exhibit moderate to very slow permeability, and have low shear strength.

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 16 miles southwest of the site. According to *Seismic Design Mapping, State of Oregon (Geomatrix Consultants, 1995),* there is no definitive consensus among geologists as to the zone fault type. Several alternate interpretations have been suggested.

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

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

Gales Creek-Newberg-Mt. Angel Fault Zone

Located approximately 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 of the fault zone forms the southwest margin of the Tualatin basin. Possible late-Quaternary geomorphic surface deformation may exist along the structural zone *(Geomatrix Consultants, 1995)*.

According to the *USGS Earthquake Hazards Program*, the Mount Angel fault is mapped as a high-angle, reverse-oblique fault, which offsets Miocene rocks of the Columbia River Basalts, and Miocene and Pliocene sedimentary rocks. The fault appears to have controlled emplacement of the Frenchman Spring Member of the Wanapum Basalts, and thus must have a history that predates the Miocene age of these rocks. No unequivocal evidence of deformation of Quaternary deposits has been described, but a thick sequence of sediments deposited by the Missoula floods covers much of the southern part of the fault trace.

Although no definitive evidence of impacts to Holocene sediments have clearly been identified, the Mount Angel fault appears to have been the location of minor earthquake swarms in 1990 near Woodburn, Oregon, and a M5.6 earthquake in March 1993 near Scotts Mills, approximately four miles south of the mapped extent of the Mt. Angel fault. It is unclear if the earthquake occurred along the fault zone or a parallel structure. Therefore, the Gales Creek-Newberg-Mt. Angel Structural Zone is considered potentially active.

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

The northwest-trending Lacamas Creek Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 4 miles east of the site, and form part of the northeastern margin of the Portland basin. According to *Geology and Groundwater Conditions of Clark County Washington (USGS Water Supply Paper 1600, Mundorff, 1964)* and the *Geologic Map of the Lake Oswego Quadrangle (Oregon DOGAMI Series GMS-59, 1989)*, the Lacamas 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 eleven test pits (TP-1 through TP-11) was conducted at the site on April 14, 2016. Additional subsurface exploration was conducted on August 15, 2016 and consisted of an additional seven test pits (TP-12 through TP-18). 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 development portion of the project lies at the foot of the south-facing aspect of Green Mountain in the vicinity of the previous Green Mountain Golf Course. Pods B3 and E1, as well as the bulk of Phase 1H are generally densely vegetated with mature fir and deciduous trees and associated understory vegetation. The remainder of the site had been previously been cleared during construction of the golf course.

Preliminary civil site plans indicate that Pod B3 will be located in the vicinity of the previous 18th golf green and terminate in a cul-de-sac approximately 400 feet to the northwest. Slopes in Pod B3 are generally 5 to 15 percent, with a portion of the northern-most lots encroaching slopes measuring 15 to 20 percent. Steeper slopes approaching approximately 40 percent begin north of the boundary of Pod B3. This area of steeper slopes represents the portion of the property underlain by a layer of basalt gravels weathered from the intrusive cinder cone to the north.

Pod E1 will be located in the vicinity of the previous 16th golf green. Slopes in Pod E1 range from 10 to 15 percent, with steep slopes approaching 40 percent generally occupying the area north of the boundary of Pod E1. Additional reconnaissance information is provided in Section 5.1.2.

4.2 Subsurface Exploration and Investigation

Test pit explorations TP-1 through TP-11 were advanced at the site to a maximum depth of 14 feet using a track-mounted excavator on April 14, 2016. Test pit explorations TP-12 through TP-18 were advanced to a maximum depth of 20 feet with a larger (320) excavator on August 15, 2016. Test pit explorations were loosely backfilled with onsite soil and should be identified and repaired at the time of site construction. Subsurface exploration locations were selected to observe soil characteristics in proximity to proposed development and sloped areas. Subsurface exploration locations are indicated on Figure 2.

4.2.1 Soil Type Description

The field investigation indicated the proposed development area is generally covered with 12 to 18 inches of topsoil and associated organic-rich root zone material at the locations observed. Underlying the topsoil layer, sandy clay soils underlain by weathered conglomerate were encountered in Pod B3, while weathered basalt gravel soils were identified above weathered conglomerate in Pod E1. Due to the variability of the conglomerate parent material, weathered soils at the site were found to consist of variable grain-size distributions and plasticity values. For engineering purposes, subsurface lithology may generally be described by the following soil types.

Soil Type 1 –*Sandy Lean Clay*

Soil Type 1 was observed to consist of brown to orange-brown medium stiff, moist, moderate to high plasticity sandy SILT and sandy LEAN CLAY. Soil Type 1 was observed underlying the topsoil layer in test pits TP-1, TP-2, TP-4, TP-5, TP-12, TP-14, TP-17, and TP-18 to a maximum depth of 9 feet.

Analytical laboratory testing conducted upon a representative soil sample of the clay indicate approximately 63 percent by weight passing the No. 200 sieve and in situ moisture content of 25 percent. Atterberg test results indicated a liquid limit of 38 percent and a plasticity index of 15 percent. Soil Type 1 is classified as CL, Sandy Lean CLAY according to USCS specifications and A-6-8 according to AASHTO specifications.

Soil Type 2 – Weathered Basalt Gravel

Soil Type 2 was observed to consist of reddish-brown, medium-dense, moist, clayey SAND with gravel to silty GRAVEL with sand. Soil Type 2 represents weathered basalt gravel eroded from the Green Mountain cinder cone north of the Pods. Soil Type 2 was encountered at the surface in explorations TP-3 and TP-6 through TP-11, TP-15, and TP-16 to a maximum depth of 6 feet below ground surface.

Analytical laboratory testing conducted upon representative soil samples indicate approximately 34 percent by weight passing the No. 200 sieve and in situ moisture content ranging from 25 to 30 percent. Atterberg test results indicated a liquid limit ranging from 33 to 38 percent and a plasticity index of 10 percent. Soil Type 2 is classified as SC, clayey SAND with gravel and GM, silty GRAVEL with sand according to USCS specifications and A-2-4(0) according to AASHTO specifications.

Soil Type 3: Weathered Conglomerate Bedrock

Semi-consolidated to unconsolidated conglomerate was encountered beneath Soil Types 1 and 2 at depths below 6 to 9 feet in all test pits. The conglomerate was completely weathered to soil in many locations, possibly accelerated due to proximity to the cinder cone intrusion. Soil Type 3 was observed to consist of orange-brown to gold-brown, moist, dense or stiff clayey GRAVEL with sand or Lean CLAY with sand to silty SAND. Gravels, where present, were observed to be rounded to sub-rounded and ½-inch to 6 inches in size.

Analytical laboratory testing conducted upon representative soil samples indicate approximately 13 to 71 percent by weight passing the No. 200 sieve and in situ moisture content ranging from 18 to 35 percent. Atterberg test results indicated a liquid limit ranging from 0 to 52 percent and a plasticity index of 0 to 28 percent. Soil Type 3 is classified as GC, clayey GRAVEL with sand, CL, Lean CLAY with sand, ML, sandy SILT, and SM, silty SAND according to USCS specifications.

4.2.2 Groundwater

Groundwater was not encountered in test pits TP-1 through TP-11 at depths up to 14 feet below ground surface and test pits TP-12 through TP-18 at depths up to 20 feet. Groundwater seeps were encountered in test pit TP-5 at approximately 2.5 feet. Significant runoff from the south-facing Green Mountain slopes was apparent during visits to the site during periods of wet weather. Cut-off trenches constructed by the previous golf course were observed in several areas in the study area that served to divert runoff from saturating golf course fairways. Review of well logs available through the Washington Department of Ecology indicate wells on the golf course grounds could exhibit artesian pressure.

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 GEOLOGIC HAZARDS

According to the Geologically Hazardous Areas provision of the Camas Municipal Code (*CMC Section 16.59*), designated or potential geologic hazard areas include areas susceptible to erosion, landslide, seismic, and other hazards including mass wasting, debris flow, rock fall, and differential settlement hazards. Hazard mapping provided by *Clark County Maps Online* indicates the presence of erosion hazards mapped along the south-facing slopes of Green Mountain, and areas of potential instability or slopes greater than 15 percent associated with the same sloped areas. The hazard map identifies the site as having very low to low susceptibility to soil liquefaction in the event of an earthquake.

Columbia West reviewed the preliminary lot layout (identified in Figure 2) provided by Olson Engineering for proposed residential Pods B3 and E1. A geologic hazard assessment was conducted for the subject site to determine the presence of potential geologic hazards and if applicable, the effect proposed site development may have on identified hazards. Results of the geologic hazard assessment are discussed in the following sections.

5.1 Slope and Landslide Hazards

To assess whether slope or landslide hazards are present at the site, Columbia West conducted a geologic hazard assessment consisting of literature review, site reconnaissance, and slope stability analysis. Existing conditions were analyzed and mitigative measures for identified hazards pertaining to proposed grading are recommended below.

5.1.1 Literature Review

According to the *Geologic Map of the Lacamas Creek Quadrangle, Clark County, Washington,* (US Geological Survey, Science Investigations Map 2924, 2006), near-surface geology underlying the area of Pods B3 and E1 is expected to consist of unconsolidated to semi-consolidated, pebble to cobble conglomerate (QTc) and semi-consolidated to well consolidated pebble to cobble conglomerate with lenses of arkosic to basaltic sandstone (Ttfc). A cinder cone comprised of the basaltic andesite of Green Mountain (Qbgm) has intruded the conglomerate just north of the Pods, creating the conical mountain adjacent to the site. No landslide deposits are mapped in the vicinity.

Columbia West also reviewed *Slope Stability, Clark County, Washington (Fiksdal, 1975)* to assess site slope characteristics. The report identifies four levels of potential slope instability within Clark County: (1) stable areas – no slides or unstable slopes, (2) areas of potential instability because of underlying geologic conditions and physical characteristics associated with steepness, (3) areas of historical or still active landslides, and (4) older landslide debris. The site is generally mapped as (1) stable areas – no slides or unstable slopes with portions of the steeper south-facing Green Mountain slopes mapped as (2), areas of potential instability.

Clark County MapsOnline further provides slope mapping used to designate landslide hazard areas. Based upon map information and site reconnaissance, areas of potential instability or slopes greater than 15 percent are mapped in localized areas associated with the south-facing slopes on the property.

5.1.2 Subsurface Reconnaissance

To observe geomorphic and geologic conditions, Columbia West conducted visual and physical reconnaissance of the area proposed for development. Subsurface investigation indicates the two Pods are underlain by soils consisting of weathered conglomerate and associated fine-textured surficial soils. Minor gravel deposits consisting of angular basaltic andesite boulders and cobbles exist along the northern fringes of the explored area near the base of the steep slopes associated with the south-face of Green Mountain. Partially buried 10- to 15-foot basalt boulders are also present at the base of the steep slopes in the northern-most boundaries of Pod E1.

Intact basalt bedrock associated with Green Mountain was not encountered during subsurface test pit exploration within the two Pods, but was observed uphill, outside of the proposed development area. Near-vertical cliff faces of approximately 30 feet in height are located approximately 200 feet north of the northern boundary of Pod B3 and approximately 200 feet northwest of Pod E1.

5.1.3 Slope Reconnaissance

Slopes are gentle in the central and southern portion of Pod B3 and field measurements indicate slopes range from 0 to 15 percent with a maximum height of approximately 10 to

12 feet. Slopes steepen quickly at the northern boundary of Pod B3. Field measurements indicate slopes ranging from 15 to 40 percent in the area. Elevations within Pod B3 range from approximately 230 feet in the southern area to approximately 254 feet in the northern area.

Pod E1 is situated on a minor terrace above the valley floor at the intersection of the southern slopes of Green Mountain. Slopes are gentle in the southeastern portion of the Pod and field measurements indicate slopes range from 5 to 15 percent. The terrain gradually steepens to the north, east, and west with slopes approaching 40 percent north of Pod E1. Elevations within Pod E1 range from approximately 280 feet in the southeastern area to approximately 315 in the north central area.

Field observations indicate that slopes above both areas are generally planar with little or no evidence of recent instability. Slopes currently support heavy vegetation consisting of established coniferous and deciduous trees with mixed understory vegetation. Evidence of surface water runoff coming from the upper slopes of Green Mountain was observed in several places. Scoured channels were not observed, but areas of matted vegetation were apparent.

5.1.4 Rock-fall

No marked trees, disturbed ground, or other evidence of recent rock-fall activity was observed within the pods. Rock-fall that may have occurred in the past appeared heavily weathered and buried in surrounding soil or tree roots or appeared to be the result of long-term soil creep and ancient physical weathering of the cinder cone slopes.

Rock-fall hazard is typically associated with blasted rock faces and is largely evaluated based on frequency of falls. Because no evidence of recent rock-fall activity was observed and the slopes are highly weathered and heavily vegetated, the current risk for rock-fall is estimated to be low. However, to further reduce potential risk, Columbia West recommends that proposed structures are set beyond (i.e., south of) the partially buried 10- to 15-foot boulders.

If a more definitive or qualitative assessment of rock-fall risk is needed, a qualified rock-fall expert should be contacted to conduct a detailed analysis. In addition, if future development, blasting, removal of existing vegetation, or grading are proposed on the top or side of Green Mountain, detailed rock-fall analysis may be required at that time.

5.1.5 Geologic Slope Hazard Areas and Hazard Mitigation

As previously discussed, areas of steep slopes are identified on the southern slopes of Green Mountain. However, these slopes are underlain by and, in some areas consist of, competent basaltic andesite bedrock and show no indications of instability. Although portions of the proposed development are in proximity to this mapped hazard area, development is feasible provided mitigation measures are put in place to limit the hazard risk.

To reduce the potential risk of adverse impacts to slope stability near the mapped geologic hazard, Columbia West recommends that earthwork-grading consisting of significant soil removal or cut areas should be limited at the base of the south-facing slopes of Green Mountain where both proposed pods B3 and E1 are located. Earthwork-grading consisting of fill placement is generally acceptable in these areas as fill placed near the base of the slope and may improve overall global stability. Some soil cut and fill associated with development is acceptable. Columbia West should be contacted to review the final grading plan for general compliance with geotechnical hazard mitigation recommendations.

5.1.6 Hazard Area Grading and Drainage Recommendations

Columbia West has reviewed preliminary grading design and lot layouts for Pods B3 and E1. The preliminary grading plan provided by Olson Engineering depicts significant fill material placed at the base of the Green Mountain slopes in Lots 6 through 11 in Pod B3 and Lots 9 through 13 in Pod E1. Fill material which is benched and keyed at the toe of the Green Mountain slopes according to the recommendations in Sections 6.2 and 6.3 is acceptable and may increase overall slope stability.

The preliminary grading plans also depict cuts along the northern border of Lots 3/4 and 7/8 in Pod E1 with a proposed retaining wall to make up the grade change. Other cuts and fills are depicted in the central portion of Lots 2 through 8 (Pod E1). Minor cuts, as depicted, are acceptable in these areas provided retaining walls are engineered and properly constructed. Columbia West should be contacted to review the final retaining wall design for compliance with geotechnical recommendations.

Stormwater runoff and groundwater seeps and springs associated with Green Mountain are anticipated to impact Pods B3 and E1. Columbia West recommends that adequate drainage is designed to intercept and control both surface flow and groundwater seeps and springs. Subsurface drainage trenches can be constructed at the time of retaining wall, road, and utility installation or on an as-needed basis during construction. Drains should discharge to an approved location. Adequate stormwater run-on management should also be taken into consideration in the civil design. Additional drainage recommendations are provided in Section 6.11.

5.1.7 Potential Encroachment within the Geologic Hazard Area

The geologic hazard area is not intended to be a do-not-disturb conservation area. Earthwork and grading associated with development are acceptable provided it is consistent with geotechnical recommendations. As such, encroachment of some site improvements or structural facilities within the geologic hazard area may be possible if evaluated in detail on a case-by-case basis. Retaining wall design and proposed construction within mapped hazard areas should be reviewed by Columbia West. Feasibility of encroachment will depend upon dimensions, locations, and specific design features of the proposed improvements. The investigation may include additional exploratory activities and data analysis to develop appropriate design recommendations.

For individual residential lots, disturbances such as minor landscaping, or fence building are acceptable. The text herein pertains only to the geotechnical aspect of construction within the hazard areas.

5.1.8 Geohazard Limitations and Risk

Columbia West's geologic hazard analysis as described in this report indicates some inherent risk associated with slope instability due to proposed residential development in proximity to the slopes adjacent to Green Mountain. This is typical for development near any sloped areas. Reduction of slope instability risk may be partially obtained by applying proper site planning and engineering principles as described in this report.

Due to multiple unknowns inherent in geologic hazard analysis it is often difficult or impossible to definitively predict stability. This geologic hazard analysis is based upon information gathered from research of existing data, subsurface soil explorations, and visual site observations as described in the text herein. This geologic hazard analysis may not be valid if building locations, site grading, or other site plans are altered.

5.2 Seismic Hazards

Seismic hazard areas include areas subject to severe risk of earthquake-induced damage. Damage may occur due to soil liquefaction, dynamic settlement, ground shaking amplification, or surface faulting rupture. These seismic hazards are discussed below.

<u>Soil Liquefaction or Dynamic Settlement</u>

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

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

Based upon results of laboratory analysis and the lack of groundwater within the observed soil profile, observed site soils do not meet the criteria described above for liquefiable soils. Therefore, the potential for liquefaction of site soils is considered to be low.

<u>Ground Shaking Amplification</u>

Based upon observed subsurface soil properties and review of the *Site Class Map of Clark County, Washington (Washington State Department of Natural Resources, 2004),* site soils may be represented by Site Class C as defined in *2015 IBC Table 1613.3.2*. A designation of Site Class C indicates that minor amplification of seismic energy may occur during a seismic event due to subsurface conditions. However, this is typical for many areas within

Clark County and will not prohibit development if properly accounted for during the design process. Additional seismic information is presented in Section 6.9, *Seismic Design Considerations.*

Fault Rupture

According to review of published geologic literature by the Washington Department of Natural Resources, USGS, DOGAMI, and others, no known geologic seismic faults are within the site boundaries. Therefore, the site is not identified as a Potential Fault Rupture Hazard Area.

5.3 Erosion Hazard Areas

According to *Clark County Maps Online (http://gis.clark.wa.gov)*, the *Web Soil Survey (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2014 Website)*, and field observations, the erosion hazard for site soils ranges from slight to severe. The severe erosion hazard areas are associated with steeper mountain slopes as described above. In order to minimize impacts within erosion hazard areas, development impacts should be designed to mitigate or improve erosion potential.

For flat to shallow-gradient portions of the property, the erosion hazard is low. Erosion potential generally increases in sloped areas. Soil is prone to erosion if unprotected and unvegetated during periods of increased precipitation. Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required.

A site-specific erosion control plan and BMPs should be utilized to reduce potential impacts on site soils during construction. Properly implemented erosion control measures as proposed by the site civil engineer should adequately mitigate impacts due to site development. Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Erosion potential is discussed further in Section 6.14, *Erosion Control Measures*. Provided the measures identified above are followed, in Columbia West's opinion, the erosion hazard can be sufficiently mitigated.

6.0 DESIGN RECOMMENDATIONS

The geotechnical site investigation suggests the proposed development is generally compatible with surface and subsurface soils, provided recommendations presented in this report are utilized and incorporated into the design and construction processes. The primary geotechnical issues at the site are the presence of steep slopes above the proposed pod development and groundwater and stormwater seepage and runoff.

6.1 Site Preparation and Grading

Vegetation, organic material, unsuitable fill, and deleterious material that may be encountered should be cleared from areas identified for structures and site grading. Vegetation, other organic material, and debris should be removed from the site. Stripped topsoil should also be removed, or used only as landscape fill in nonstructural areas with slopes less than 25 percent. The anticipated stripping depth for sod and highly organic topsoil is anticipated to vary from 12 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.

Because a portion of the site has been previously developed into a golf course, 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 following the guidelines presented in Section 6.2. Previously backfilled utility trenches may not contain structural fill which may need to be reconditioned so that it can support designed infrastructure.

Site grading activities should be performed in accordance with requirements specified in the 2015 *International Building Code* (IBC), Chapter 18 and Appendix J, with exceptions noted in the text herein. Site preparation, soil stripping, grading activities, and demolition debris removal verification should be observed and documented by an experienced geotechnical engineer or designated representative.

6.2 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. A field density at least equal to 95 percent of the maximum dry density, obtained from the modified Proctor moisture-density relationship test (AASHTO T-180), is recommended for structural fill placement in roadways. A field density of at least 90 percent (AASHTO T-180) is recommended for lot fills. The soil moisture content should be within three percentage points of the optimum moisture content as determined by laboratory Proctor test results. 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. 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 native soils, import structural fill consisting of *WSDOT 9-03.14(1) Gravel Borrow* is recommended. 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.

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.

6.3 Cut and Fill Slopes

Fill placed on existing grades steeper than 5H:1V should be horizontally benched at least 10 feet into the slope. Fill slopes greater than six feet in height should be vertically keyed into existing subsurface soil. A typical fill slope cross-section is shown in Figure 3. Drainage implementations, including subdrains or perforated 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 20 feet in total height without geotechnical review. Values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 4.

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

6.4 Foundations

Foundations are anticipated to consist of shallow continuous perimeter or column spread footings. Footings should be designed by a licensed structural engineer and conform to the recommendations below. Typical building loads are not expected to exceed approximately 2 to 3 kips per foot for perimeter footings or 10 to 20 kips per column.

The existing ground surface should be prepared as described in Section 6.1, *Site Preparation and Grading*, and Section 6.2, *Engineered Structural Fill*. Foundations should bear upon firm undisturbed native soil or engineered structural fill. Disturbed surface soils and unsuitable fill should be removed from foundation alignments and replaced with structural fill. Footings should be designed by a qualified structural engineer, have a minimum width of 12 inches, and extend to a depth at least 18 inches below lowest adjacent grade. 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. 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.

6.5 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 6.1, *Site Preparation and Grading* and Section 6.2, *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.

6.6 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.

6.7 Excavation

Soils at the site were explored to a maximum depth of 20 feet using a track-mounted excavator. As mentioned previously, weathered conglomerate bedrock was encountered at depth underlying fine-textured and gravelly soils. The weathered conglomerate was generally dense or stiff and medium-sized boulders were encountered both on the surface and at depth. Specialized rock-excavation techniques or blasting are not anticipated within 15 feet of existing surface in the locations observed. Areas of pinnacled bedrock may hinder utility installation in areas not explored. Groundwater seeps may also be encountered.

As mentioned previously, 10- to 15-foot basalt boulders are present in the northern-most boundaries of Pod E1 and smaller boulders were observed north of Pod B3. If encountered below ground surface during construction, these boulders may be very difficult to excavate due to their size. Specialized equipment to break up the basalt boulders may be necessary for grading or utility installation in these areas.

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.

6.8 Lateral Earth Pressure

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 90 percent of maximum dry density as determined by the modified Proctor test (ASTM D1557 or AASHTO T-180). 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.

The design parameters presented in Table 1 are valid for static loading cases. The recommended earth pressures do not include surcharge loads, dynamic loading, hydrostatic pressure, or seismic design.

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

Table 1. 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 sandy lean CLAY Soil Type 1	63 pcf	43 pcf	306 pcf	115 pcf	27°
In situ undisturbed weathered basalt gravel and CONGLOMERATE (clayey gravel) Soil Type 2	62 pcf	42 pcf	346 pcf	120 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.*

Final retaining wall design should be reviewed 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.

6.9 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 2.

The listed probabilistic ground motion values are based upon “firm rock” sites with an assumed shear wave velocity of 2,500 ft/s in the upper 100 feet of soil profile. These values should be adjusted for site class effects by applying site coefficients Fa and Fv as defined in 2015 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.

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

	2% Probability of Exceedance in 50 yrs
Peak Ground Acceleration	0.37 g
0.2 sec Spectral Acceleration	0.88 g
1.0 sec Spectral Acceleration	0.37 g

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 2015 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.

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 2015 IBC, the potential for peak ground accelerations in excess of the adjusted and amplified values should be understood.

6.10 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.

6.11 Drainage

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. Drainage design in general should conform to City of Camas regulations. Finished site grading should be conducted with positive drainage away from structures. Depressions or shallow areas that may retain ponding water should be avoided. Roof drains, low-point drains, and perimeter foundation drains are recommended for structures. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from foundations into 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.

Columbia West also recommends incorporation of intercept drains into the stormwater system design to capture potential runoff and variable groundwater flow originating from Green Mountain and directed toward development areas in Pods B3 and E1. The drains may be incorporated into other facilities design or be constructed as independent systems, as design layout allows.

Perimeter foundation drains for residential structures are recommended. 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.

6.12 Bituminous Asphalt and Portland Cement Concrete

For dry weather construction, asphalt pavement surface sections should bear upon subgrade materials constructed per the recommendations provided in Section 6.1, *Site Preparation and Grading*, and Section 6.2, *Engineered Structural Fill*. Wet weather construction is discussed in Section 6.13, *Wet Weather Construction Methods and Techniques*. 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 AASHTO T-180. 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 consist of well-graded crushed surface base course materials meeting specifications provided in the WSDOT Standard Specifications Division 9.03.9(3) and be compacted to at least 95 percent of the modified Proctor dry density, as determined by AASHTO T-180 and tested at intervals determined by the onsite geotechnical engineer. Asphalt concrete pavement should consist of WSDOT HMA Class ½" PG 64-22, or equivalent, and be compacted to at least 92 percent of maximum Rice density. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with Washington Department of Transportation and City of 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.

6.13 Wet Weather Construction Methods and Techniques

Wet weather construction often results in significant shear strength reduction and soft areas that may rut or deflect. Installation of granular working layers may be necessary to provide a firm support base and sustain construction equipment. Granular layers should consist of all-weather gravel, 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 (AASHTO T-180). 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.

6.14 Erosion Control Measures

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

Site-specific erosion control measures should be implemented to address the maintenance of exposed areas. This may include silt fence, biofilter bags, straw wattles, or other suitable methods. During construction activities, exposed areas should be well-compacted and protected from erosion with visqueen, surface tactifier, or other means, as appropriate. Temporary slopes or exposed areas may be covered with straw, crushed aggregate, or riprap in localized areas to minimize erosion. Erosion and water runoff during wet weather conditions may be controlled by application of strategically placed channels and small detention depressions with overflow pipes.

After grading, exposed surfaces should be vegetated as soon as possible with erosion-resistant native 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.

6.15 Soil Shrink/Swell Potential

Based upon laboratory analysis, subsurface soils contain as much as 71 percent by weight passing the No. 200 sieve and exhibit a plasticity index ranging from 0 to 28 percent. This indicates minor potential for soil shrinking or swelling. To minimize potential risks, foundation embedment depth may be increased. Columbia West should closely monitor placement and compaction activities if onsite soils are proposed for use as engineered structural fill. The potential for soil swelling can be minimized by properly controlling moisture content during fill placement.

6.16 Utility Installation

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

Utilities should be installed in general accordance with manufacturer's recommendations. Utility trench backfill should consist of crushed aggregate or other coarse-textured, free-draining material acceptable to the client, City of 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 modified Proctor moisture-density test (AASHTO T-180). 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.

7.0 CONCLUSION AND LIMITATIONS

This geotechnical site investigation report was prepared in accordance with accepted standard conventional principles and practices of geotechnical engineering. This investigation pertains only to material tested and observed as of the date of this report, and is based upon proposed site development as described in the text herein. This report is a 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

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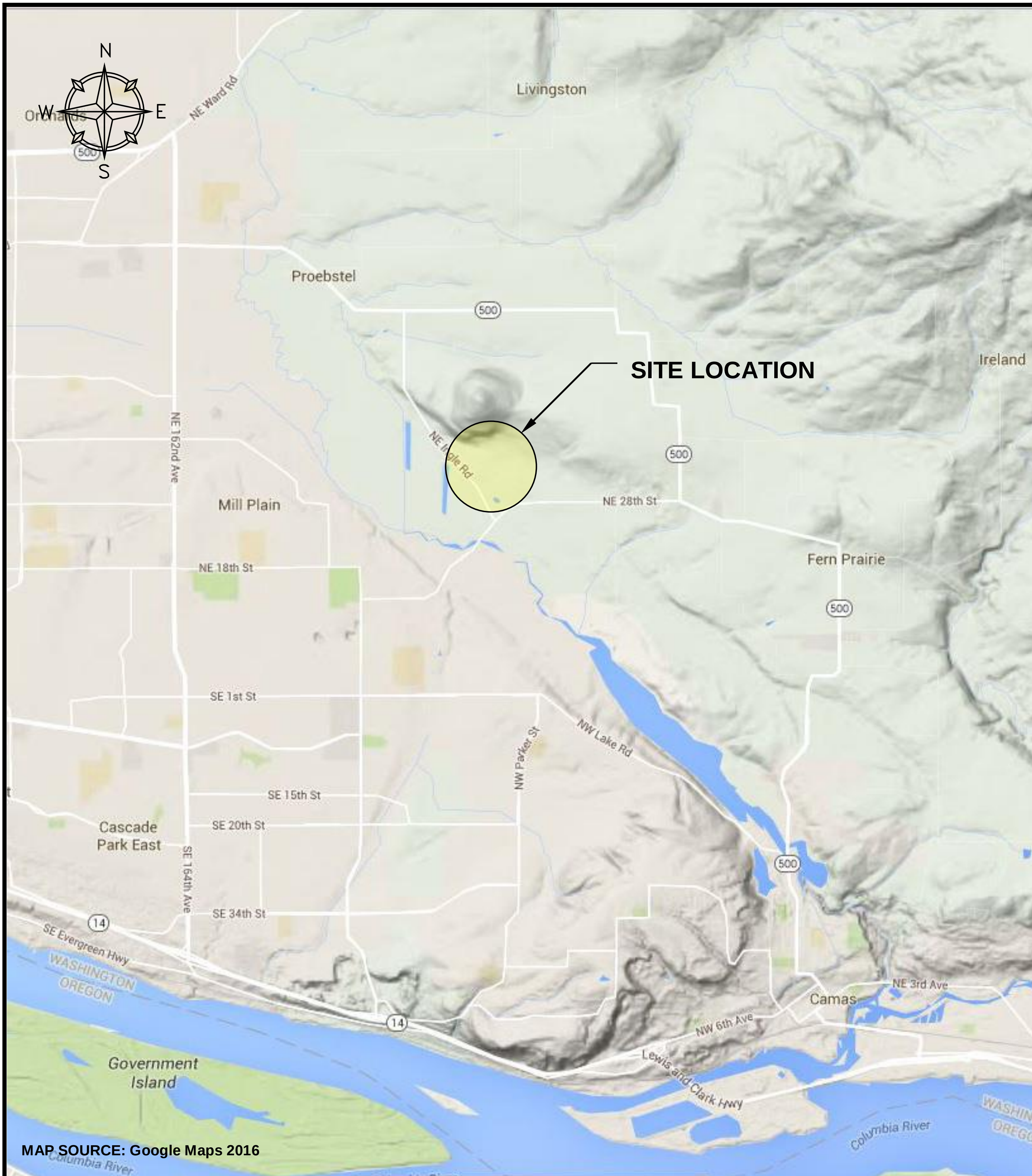
FIGURES



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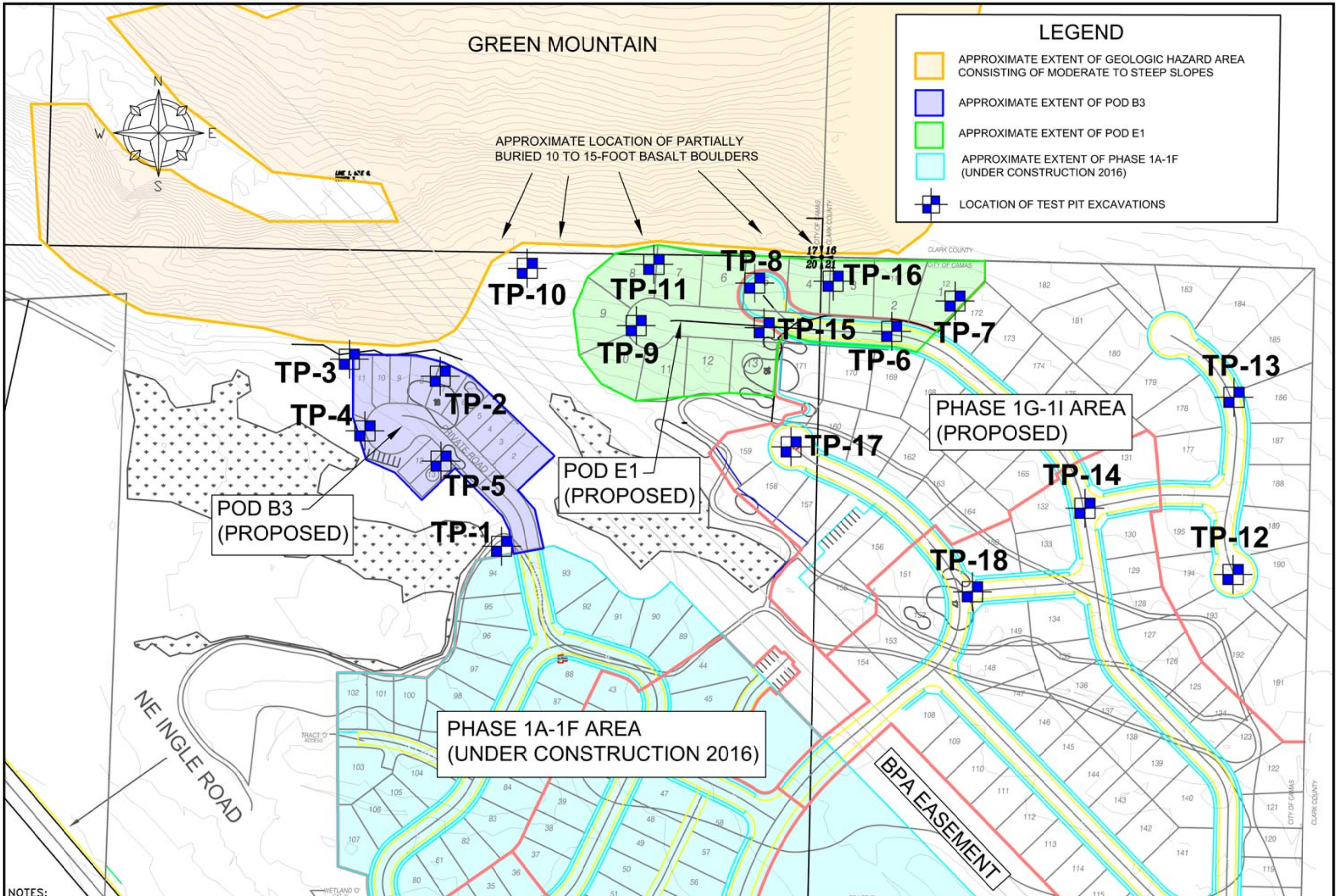
Design	Drawn: ASR		
Checked: DEL	Date: 05/10/16		
Client: CLB WOS, LLC	Rev	By	Date
Job No.: 16081			
CAD File: FIGURE 1			
Scale: ~1:50,000			

SITE LOCATION MAP

GREEN MOUNTAIN PODS B3 & E1
CAMAS, WASHINGTON

FIGURE
1

PRELIMINARY



NOTES:
1. SITE LOCATION: 2817 NE INGLE ROAD [PARCEL 172557000], CAMAS, WASHINGTON.
2. DRAWING IS NOT TO SCALE.
3. BASE MAP PROVIDED BY OLSON ENGINEERING.
4. TEST PIT LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
5. ALL TEST PITS BACKFILLED LOOSELY WITH ON-SITE SOIL ON 4/14/2016 (TP-1 TO TP-11) AND 8/15/16 (TP-12 TO TP-18).

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Design:	Drawn: ASR			
Checked: DEL	Date: 05/02/16			
Client: CLB WOS, LLC	Rev	By	Date	
Job No: 16081				
CAD File: FIGURE 2				
Scale: NONE				

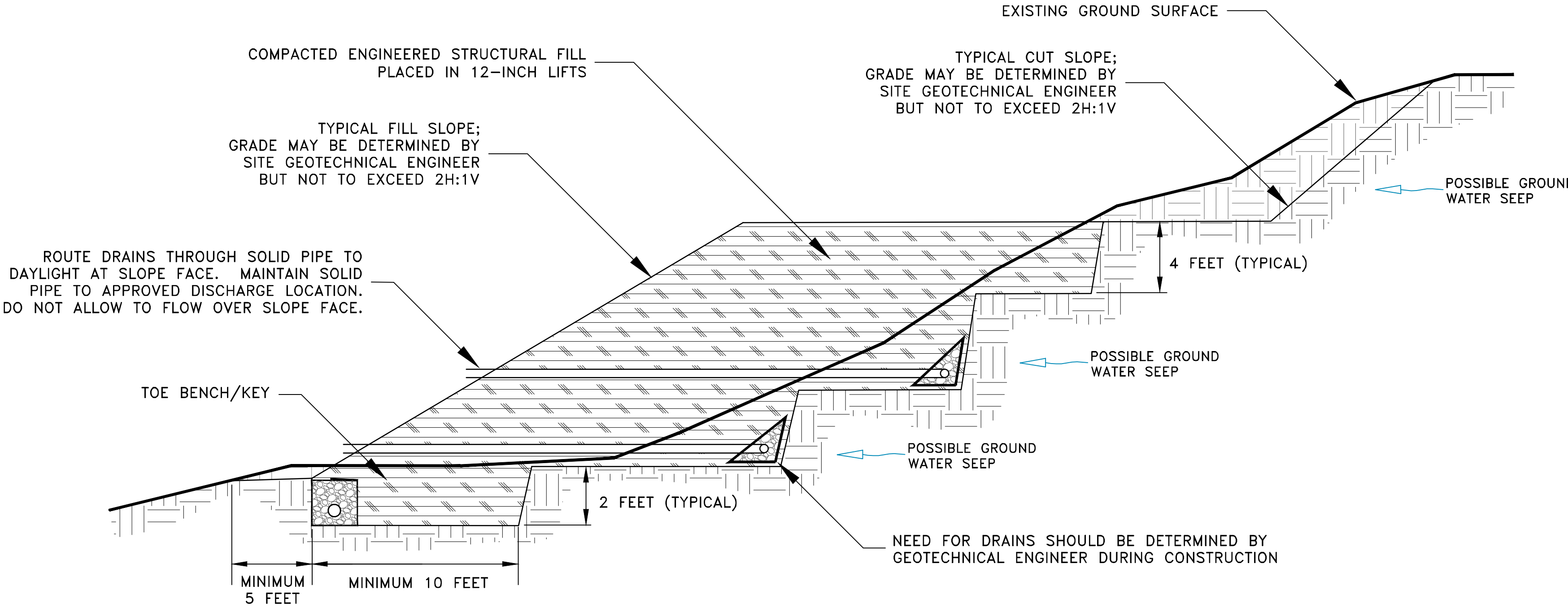
SUBSURFACE EXPLORATION LOCATION MAP

GREEN MOUNTAIN PODS B3 AND E1

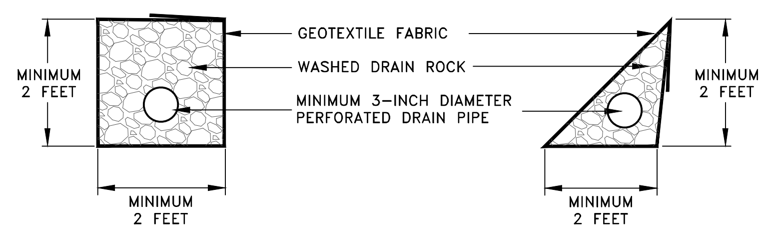
CAMAS, WASHINGTON

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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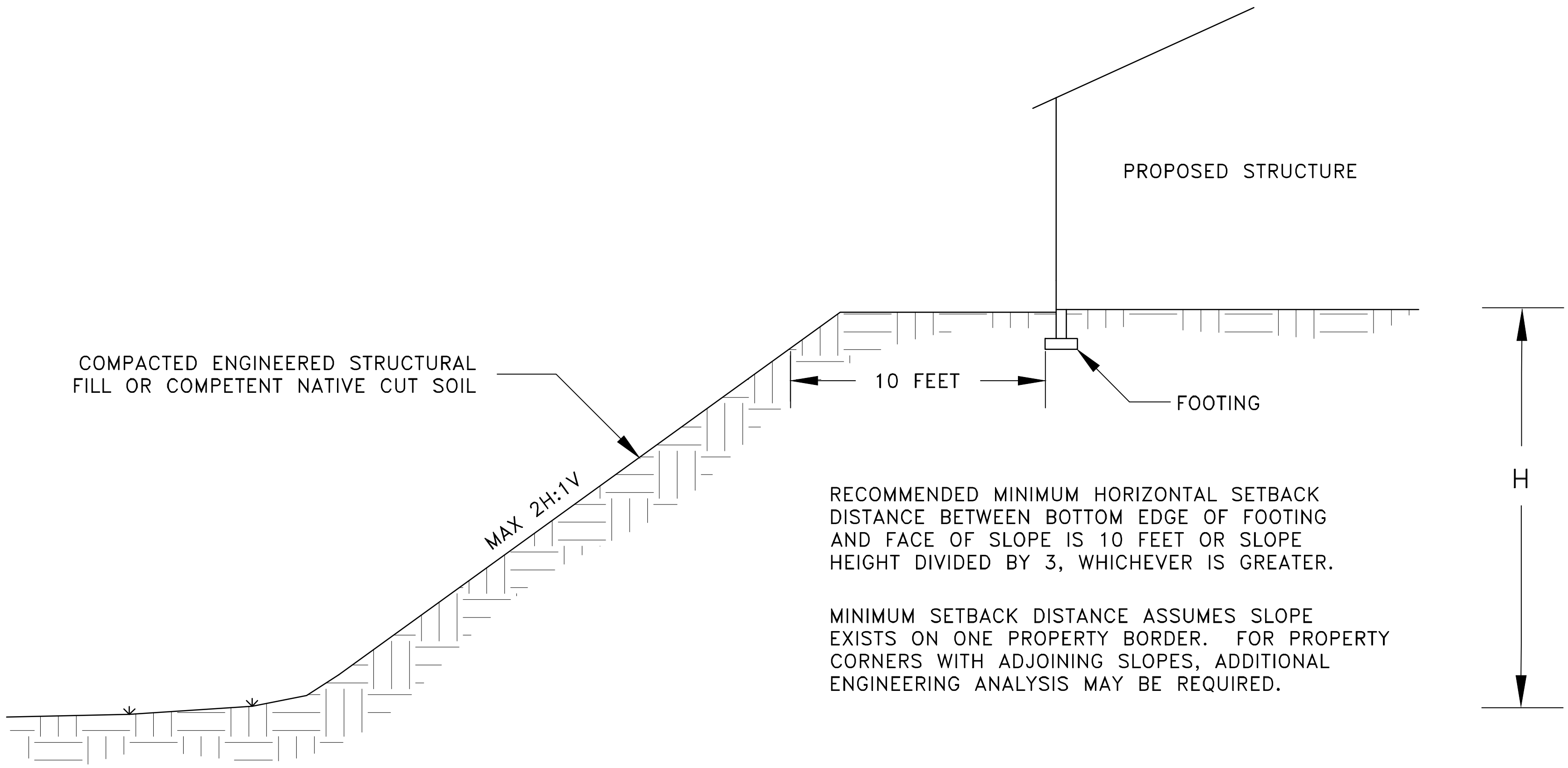
Design:	Drawn: ASR			TYPICAL CUT AND FILL SLOPE CROSS-SECTION
Checked: DEL	Date: 05/10/16			
Client: CLB WOS, LLC	Rev	By	Date	GREEN MOUNTAIN PODS B3 & E1
Job No: 16081				CAMAS, WASHINGTON
CAD File: FIGURE 3				
Scale: NONE				

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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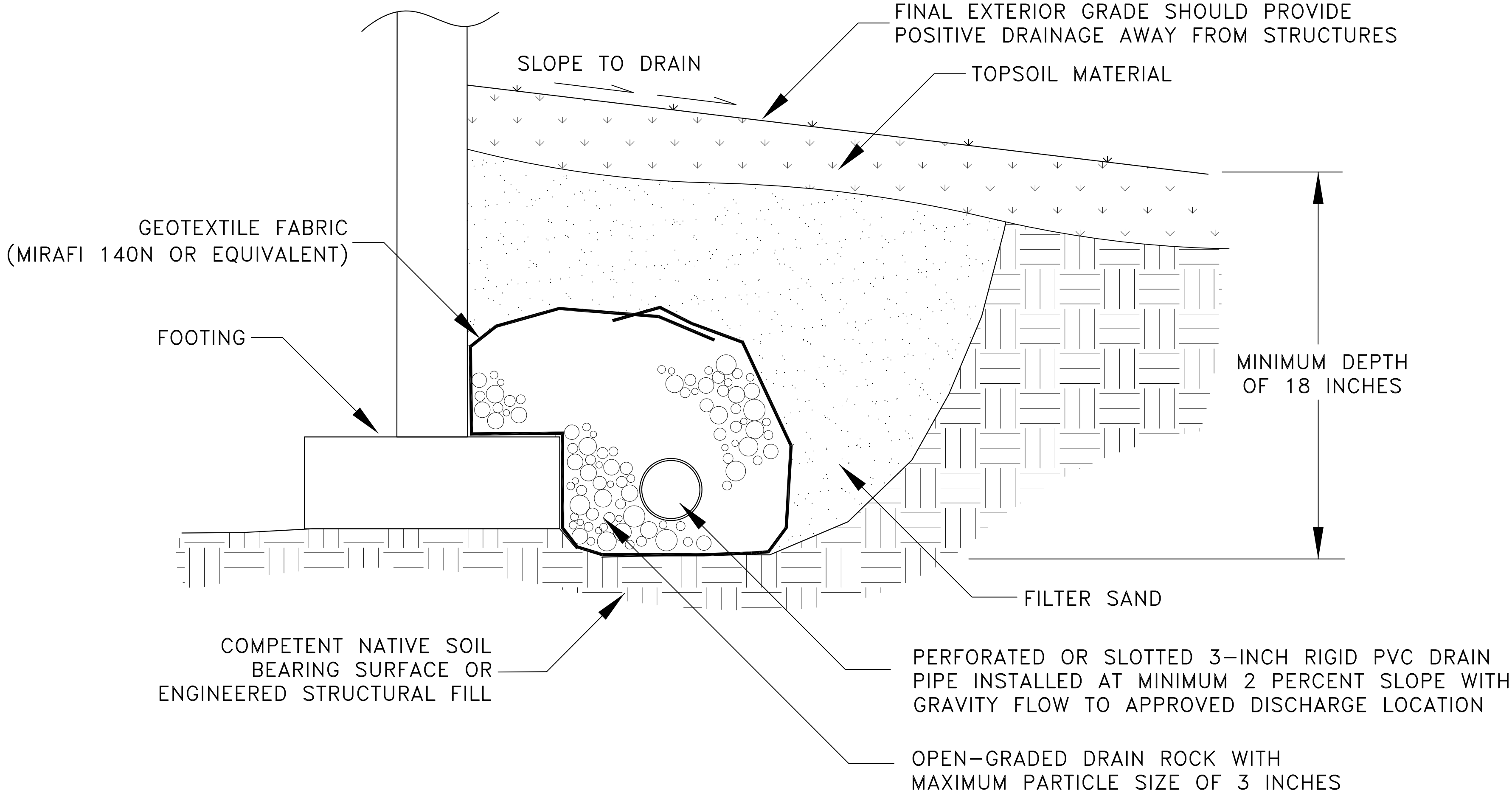
Design:	Drawn: ASR	MINIMUM FOUNDATION SLOPE SETBACK DETAIL	FIGURE
Checked: DEL	Date: 05/10/16		
Client: CLB WOS, LLC	Rev / By / Date	GREEN MOUNTAIN PODS B3 & E1	4
Job No: 16081		CAMAS, WASHINGTON	
CAD File: FIGURE 4			
Scale: NONE			

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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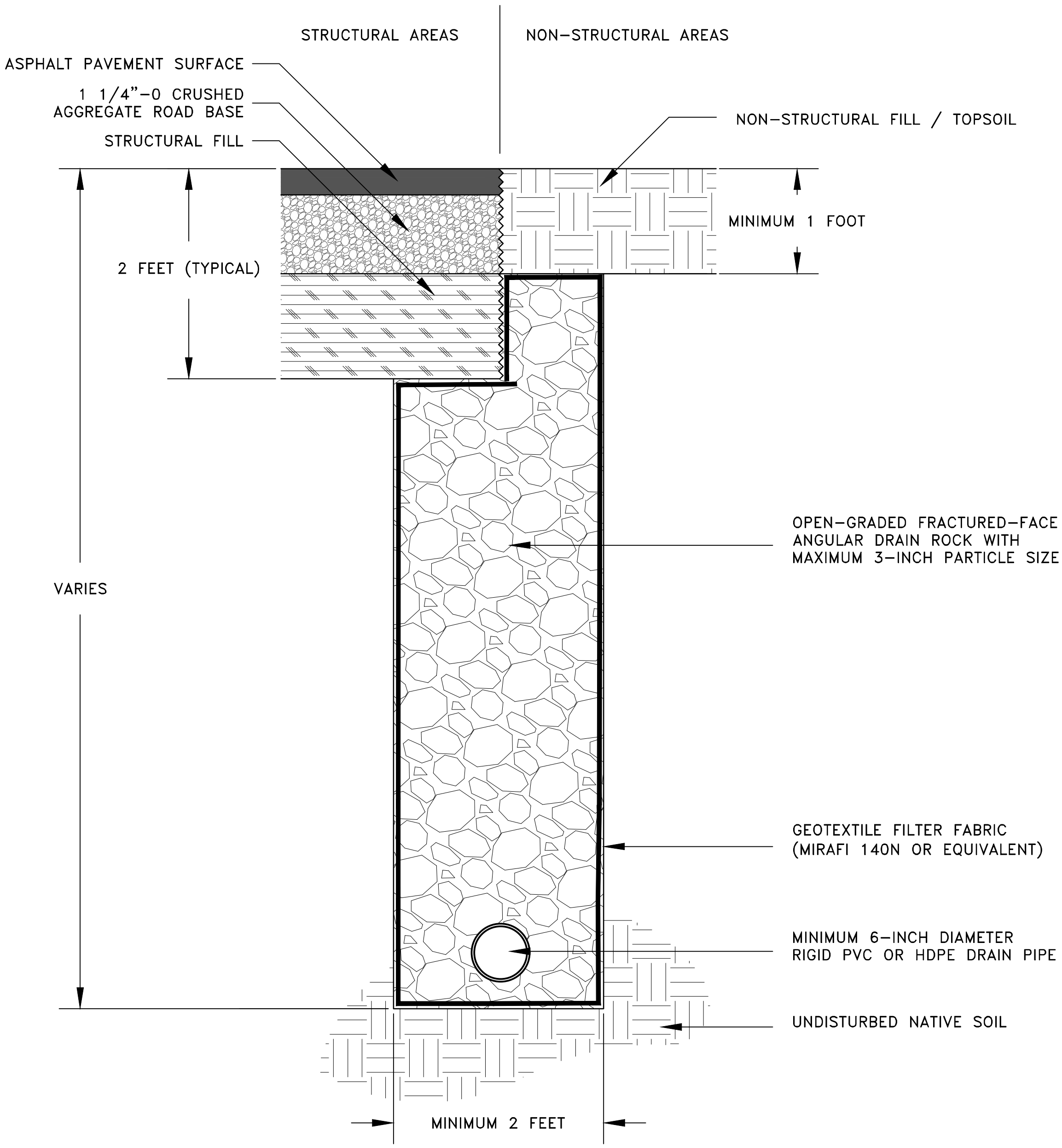
Design:	Drawn: ASR	TYPICAL PERIMETER FOOTING DRAIN DETAIL	FIGURE 5
Checked: DEL	Date: 05/10/16		
Client: CLB WOS, LLC	Rev / By / Date	GREEN MOUNTAIN PODS B3 & E1	
Job No: 16081		CAMAS, WASHINGTON	
CAD File: FIGURE 3			
Scale: NONE			

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: 05/10/16		
Client: CLB WOS, LLC	Rev	By	Date
Job No: 16081			
CAD File: FIGURE 6			
Scale: NONE			

TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL

GREEN MOUNTAIN PODS B3 & E1

CAMAS, WASHINGTON

FIGURE 6

APPENDIX A
LABORATORY TEST RESULTS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-224
		REPORT DATE: 04/25/16	FIELD ID: TP2.1
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

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

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-6(8)

LABORATORY TEST DATA

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

ADDITIONAL DATA

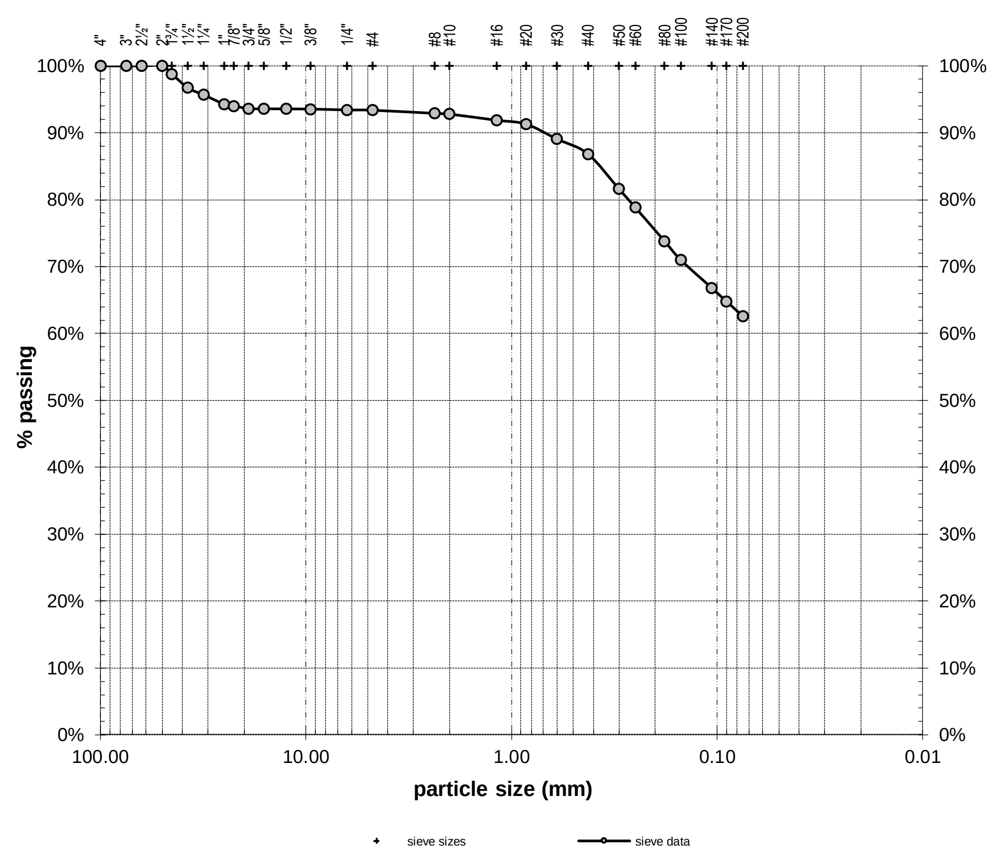
initial dry mass (g) =	2556.2		
as-received moisture content =	25.4%	coefficient of curvature, C_C =	n/a
liquid limit =	38	coefficient of uniformity, C_U =	n/a
plastic limit =	23	effective size, $D_{(10)}$ =	n/a
plasticity index =	15	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 6.6%
% sand = 30.8%
% 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%		
GRAVEL	4.00"	100.0		100.0%		
GRAVEL	3.00"	75.0		100.0%		
GRAVEL	2.50"	63.0		100.0%		
GRAVEL	2.00"	50.0	100.0%			
GRAVEL	1.75"	45.0		98.8%		
GRAVEL	1.50"	37.5	96.7%			
GRAVEL	1.25"	31.5		95.7%		
GRAVEL	1.00"	25.0	94.3%			
GRAVEL	7/8"	22.4		94.0%		
GRAVEL	3/4"	19.0	93.6%			
GRAVEL	5/8"	16.0		93.6%		
GRAVEL	1/2"	12.5		93.6%		
GRAVEL	3/8"	9.50	93.5%			
GRAVEL	1/4"	6.30	93.4%			
GRAVEL	#4	4.75	93.4%			
SAND	#8	2.36		92.9%		
SAND	#10	2.00	92.8%			
SAND	#16	1.18		91.9%		
SAND	#20	0.850	91.3%			
SAND	#30	0.600		89.1%		
SAND	#40	0.425	86.8%			
SAND	#50	0.300		81.6%		
SAND	#60	0.250	78.9%			
SAND	#80	0.180		73.8%		
SAND	#100	0.150	71.0%			
SAND	#140	0.106		66.8%		
SAND	#170	0.090		64.8%		
SAND	#200	0.075	62.6%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
04/20/16	JMR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-224
		REPORT DATE: 04/25/16	FIELD ID: TP2.1
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

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

LABORATORY TEST DATA

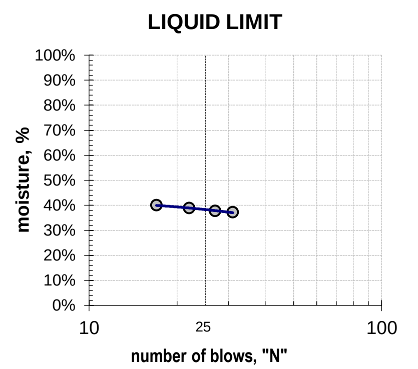
LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

ATTERBERG LIMITS

liquid limit =	38
plastic limit =	23
plasticity index =	15

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.44	35.06	33.23	32.38
dry soil + pan weight, g =	30.67	31.17	29.76	29.07
pan weight, g =	20.53	20.86	20.83	20.81
N (blows) =	31	27	22	17
moisture, % =	37.2 %	37.7 %	38.9 %	40.1 %

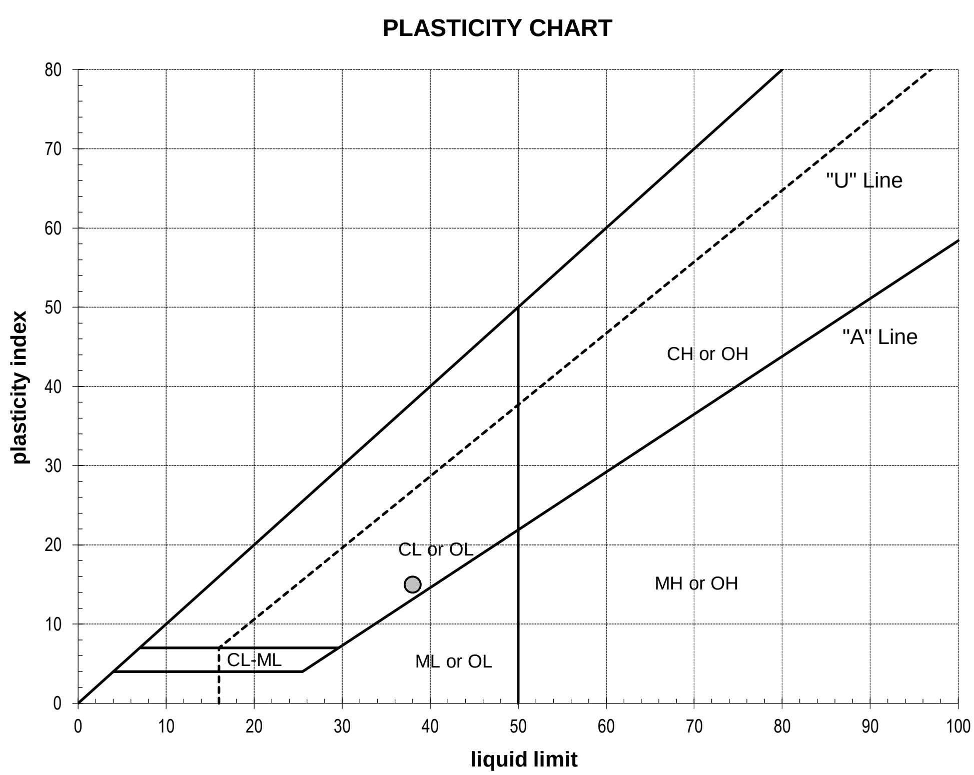


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.79	28.58		
dry soil + pan weight, g =	27.29	27.10		
pan weight, g =	20.83	20.72		
moisture, % =	23.2 %	23.2 %		



ADDITIONAL DATA

% gravel =	6.6%
% sand =	30.8%
% silt and clay =	62.6%
% silt =	n/a
% clay =	n/a
moisture content =	25.4%

DATE TESTED	TESTED BY
04/21/16	MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-225
		REPORT DATE: 04/25/16	FIELD ID: TP2.2
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey GRAVEL with Sand	Test Pit TP-02 depth = 8 feet	GC, Clayey Gravel with 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

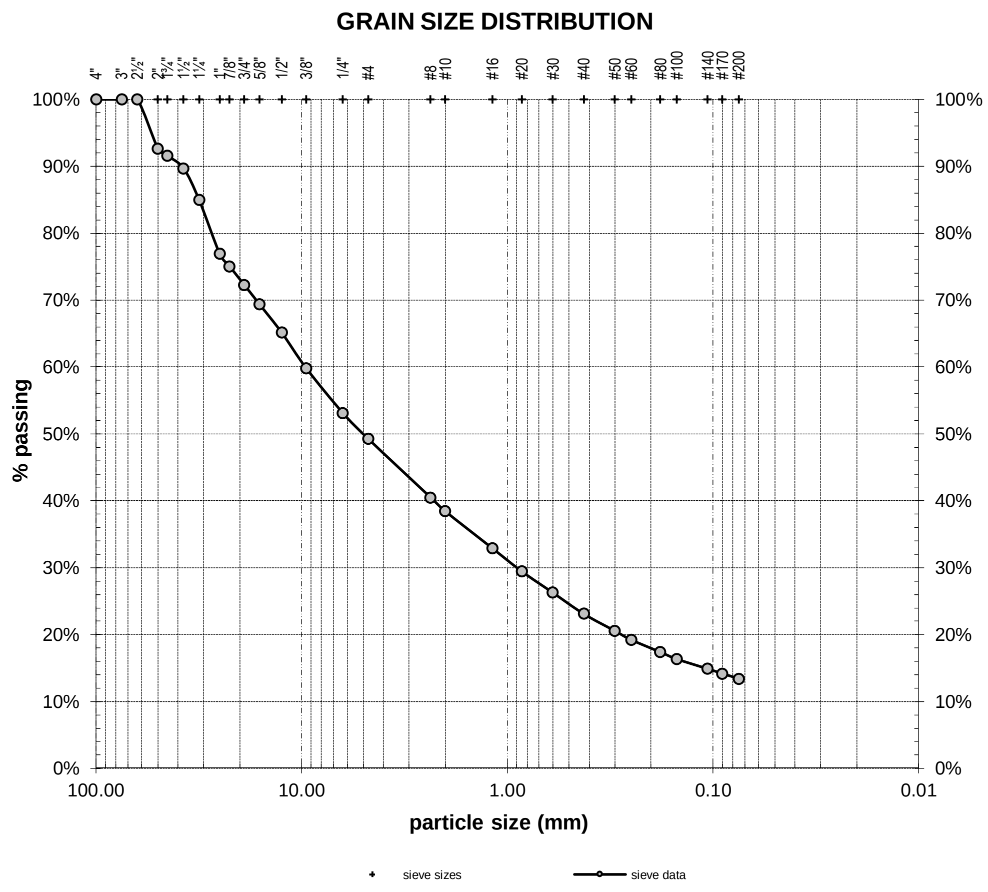
ADDITIONAL DATA

initial dry mass (g) =	3428.8		
as-received moisture content =	17.7%	coefficient of curvature, C_C =	n/a
liquid limit =	52	coefficient of uniformity, C_U =	n/a
plastic limit =	24	effective size, $D_{(10)}$ =	n/a
plasticity index =	28	$D_{(30)}$ =	0.896 mm
fineness modulus =	n/a	$D_{(60)}$ =	9.583 mm

SIEVE DATA

% gravel = 50.7%
% sand = 35.9%
% silt and clay = 13.4%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0	100.0%			
	2.00"	50.0	92.7%			
	1.75"	45.0		91.6%		
	1.50"	37.5	89.7%			
	1.25"	31.5	85.0%			
	1.00"	25.0	76.9%			
	7/8"	22.4		75.0%		
	3/4"	19.0	72.3%			
	5/8"	16.0		69.4%		
	1/2"	12.5	65.2%			
	3/8"	9.50	59.8%			
	1/4"	6.30	53.1%			
	#4	4.75	49.3%			
SAND	#8	2.36		40.5%		
	#10	2.00	38.4%			
	#16	1.18		32.9%		
	#20	0.850	29.4%			
	#30	0.600		26.3%		
	#40	0.425	23.1%			
	#50	0.300		20.6%		
	#60	0.250	19.2%			
	#80	0.180		17.4%		
	#100	0.150	16.4%			
	#140	0.106		14.9%		
	#170	0.090		14.2%		
	#200	0.075	13.4%			



DATE TESTED	TESTED BY
04/20/16	JMR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-225
		REPORT DATE: 04/25/16	FIELD ID: TP2.2
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey GRAVEL with Sand	Test Pit TP-02 depth = 8 feet	GC, Clayey Gravel with Sand

LABORATORY TEST DATA

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

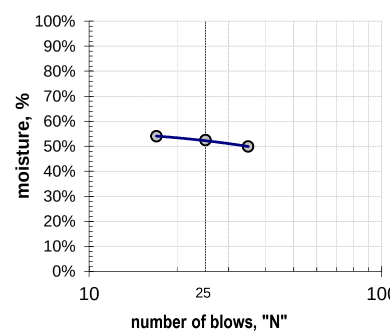
ATTERBERG LIMITS

liquid limit =	52
plastic limit =	24
plasticity index =	28

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.52	32.88	32.40	
dry soil + pan weight, g =	29.91	28.75	28.34	
pan weight, g =	20.66	20.87	20.82	
N (blows) =	35	25	17	
moisture, % =	49.8 %	52.4 %	54.0 %	

LIQUID LIMIT



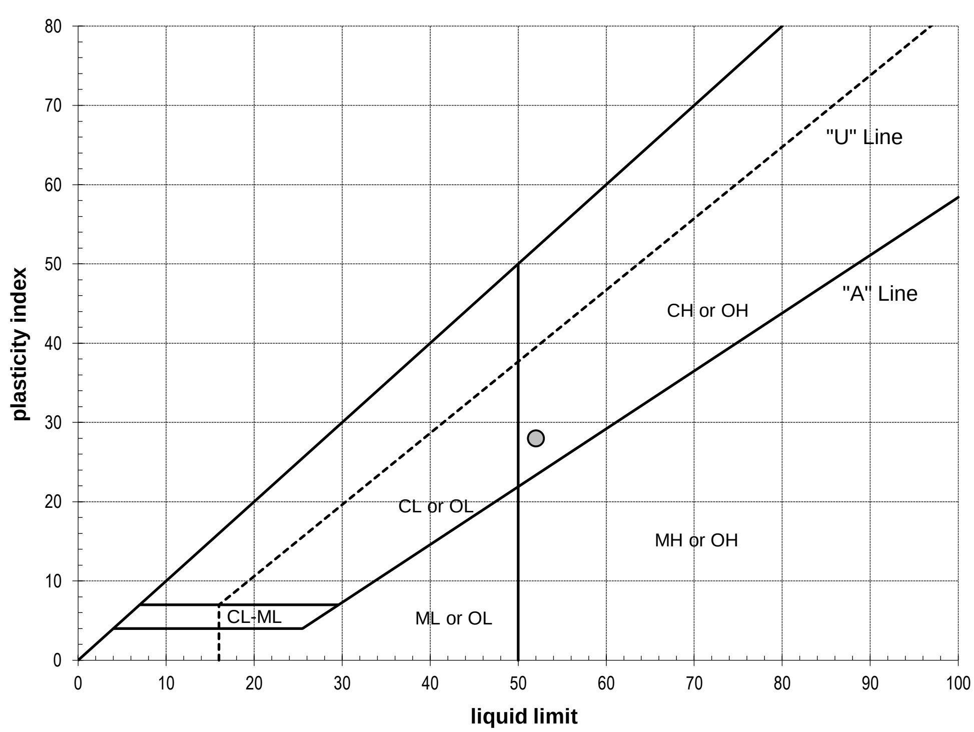
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.25	28.15		
dry soil + pan weight, g =	26.74	26.71		
pan weight, g =	20.50	20.77		
moisture, % =	24.2 %	24.2 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	50.7%
% sand =	35.9%
% silt and clay =	13.4%
% silt =	n/a
% clay =	n/a
moisture content =	17.7%

DATE TESTED	TESTED BY
04/21/16	MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-226
		REPORT DATE: 04/25/16	FIELD ID: TP6.1
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey SAND with Gravel	Test Pit TP-06 depth = 3 feet	SC, Clayey Sand with Gravel

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-2-4(0)

LABORATORY TEST DATA

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

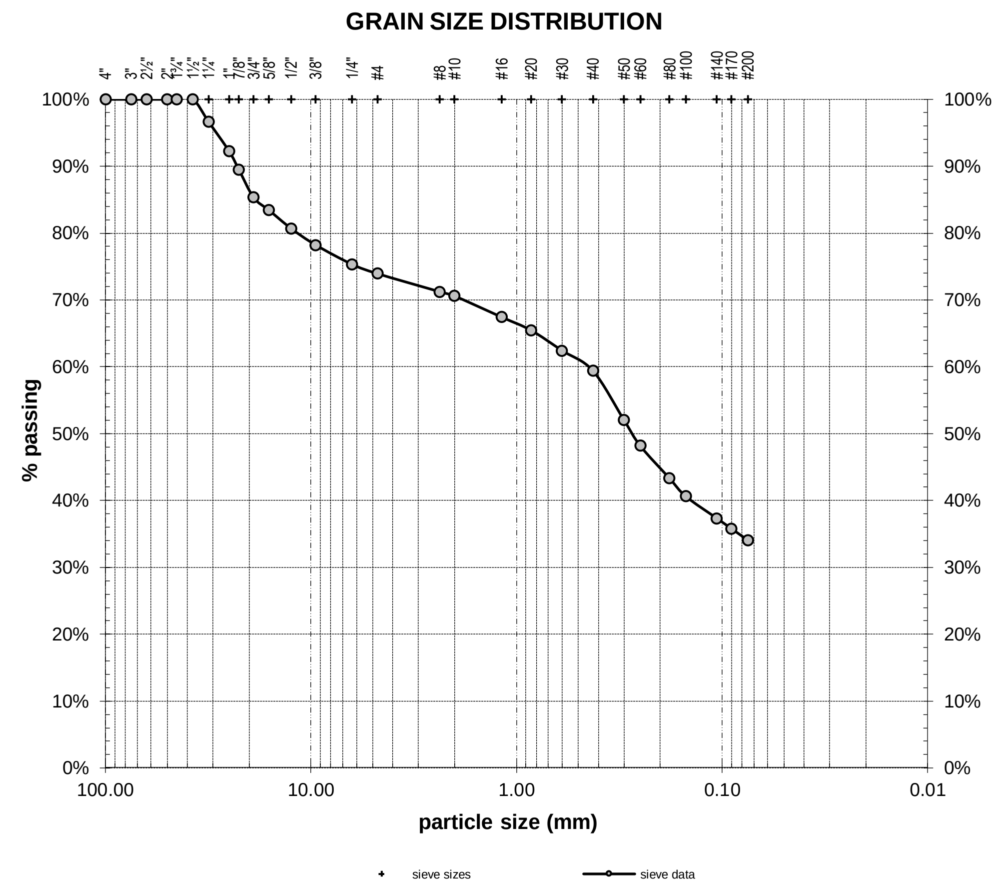
ADDITIONAL DATA

initial dry mass (g) =	2612.7		
as-received moisture content =	25.0%	coefficient of curvature, C_C =	n/a
liquid limit =	33	coefficient of uniformity, C_U =	n/a
plastic limit =	23	effective size, $D_{(10)}$ =	n/a
plasticity index =	10	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	0.454 mm

SIEVE DATA

% gravel = 26.1%
% sand = 39.9%
% silt and clay = 34.1%

	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		96.7%		
	1.00"	25.0	92.2%			
	7/8"	22.4		89.5%		
	3/4"	19.0	85.3%			
	5/8"	16.0		83.4%		
	1/2"	12.5	80.7%			
	3/8"	9.50	78.2%			
	1/4"	6.30	75.3%			
	#4	4.75	73.9%			
SAND	#8	2.36		71.2%		
	#10	2.00	70.6%			
	#16	1.18		67.4%		
	#20	0.850	65.5%			
	#30	0.600		62.4%		
	#40	0.425	59.4%			
	#50	0.300		52.0%		
	#60	0.250	48.2%			
	#80	0.180		43.3%		
	#100	0.150	40.6%			
	#140	0.106		37.3%		
	#170	0.090		35.8%		
	#200	0.075	34.1%			



DATE TESTED	TESTED BY
04/20/16	JMR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-226
		REPORT DATE: 04/25/16	FIELD ID: TP6.1
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey SAND with Gravel	Test Pit TP-06 depth = 3 feet	SC, Clayey Sand with Gravel

LABORATORY TEST DATA

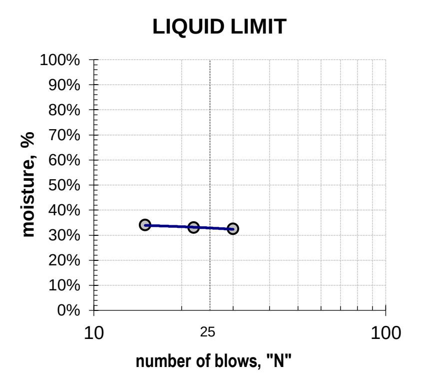
LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

ATTERBERG LIMITS

liquid limit = 33
plastic limit = 23
plasticity index = 10

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	36.89	35.68	34.92	
dry soil + pan weight, g =	32.88	31.96	31.30	
pan weight, g =	20.52	20.69	20.65	
N (blows) =	30	22	15	
moisture, % =	32.4 %	33.0 %	34.0 %	

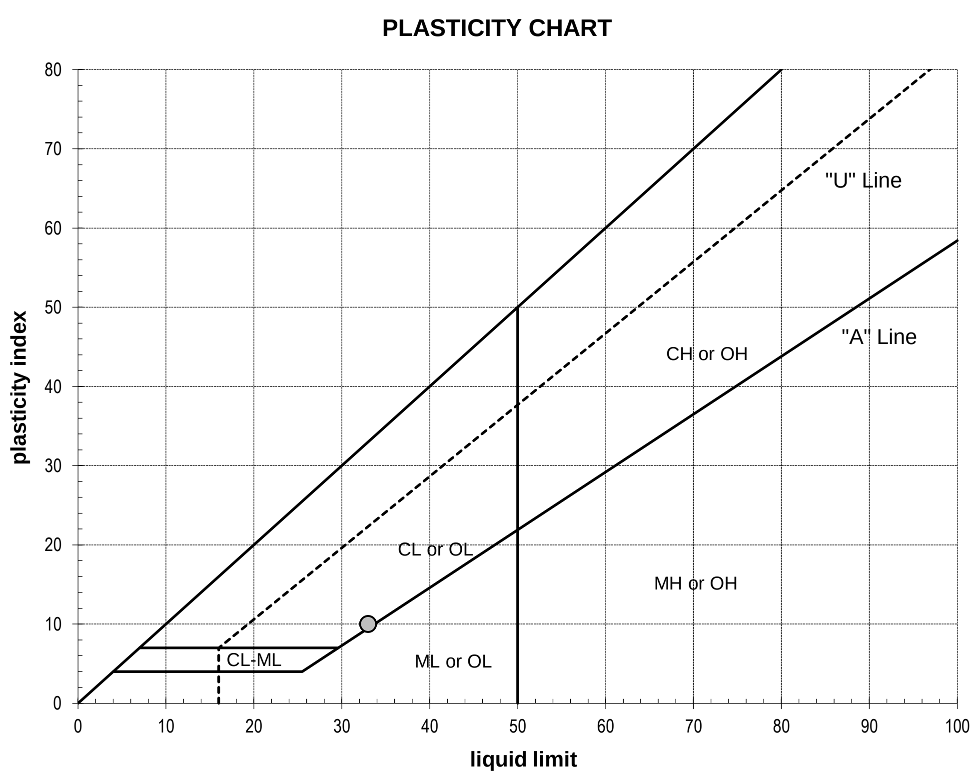


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.01	28.48		
dry soil + pan weight, g =	26.66	27.00		
pan weight, g =	20.77	20.59		
moisture, % =	22.9 %	23.1 %		



ADDITIONAL DATA

% gravel = 26.1%
% sand = 39.9%
% silt and clay = 34.1%
% silt = n/a
% clay = n/a
moisture content = 25.0%

DATE TESTED	TESTED BY
04/22/16	MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-227
		REPORT DATE: 04/25/16	FIELD ID: TP6.2
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-06 depth = 7 feet	CL, Lean Clay with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-6(7)

LABORATORY TEST DATA

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

ADDITIONAL DATA

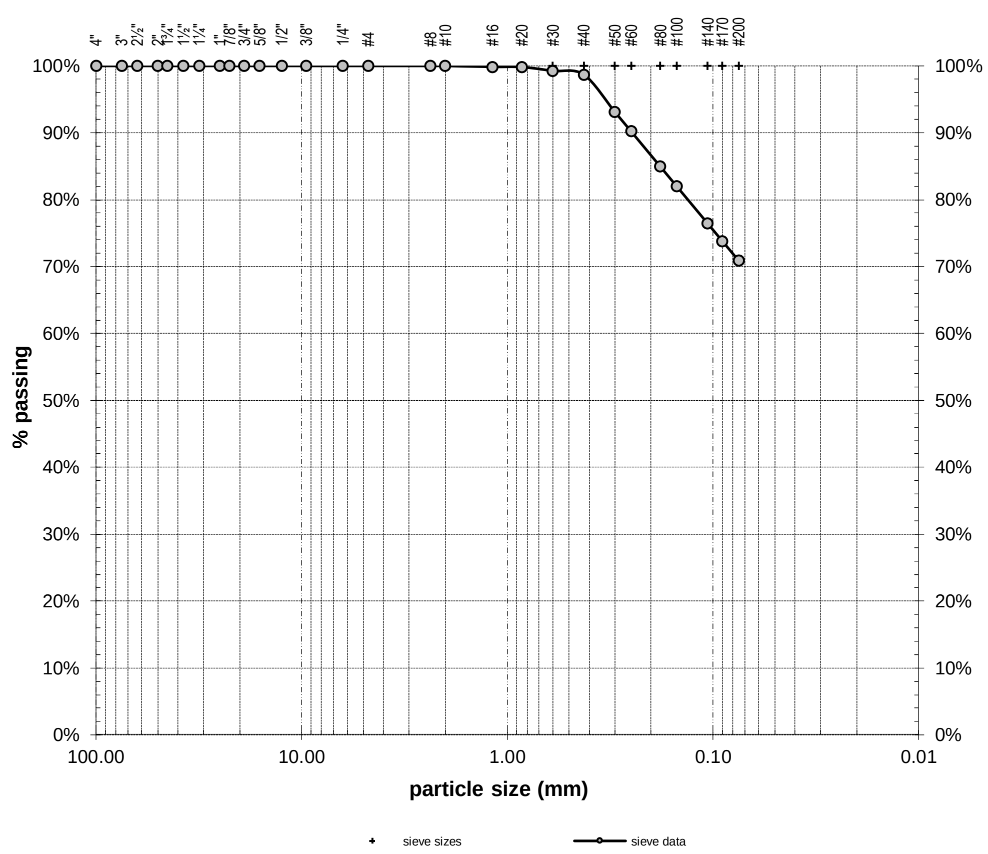
initial dry mass (g) =	196.1		
as-received moisture content =	35.1%	coefficient of curvature, C_C =	n/a
liquid limit =	31	coefficient of uniformity, C_U =	n/a
plastic limit =	19	effective size, $D_{(10)}$ =	n/a
plasticity index =	12	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 29.1%
% silt and clay = 70.9%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		100.0%		
	#10	2.00	100.0%			
	#16	1.18		99.9%		
	#20	0.850	99.8%			
	#30	0.600		99.3%		
	#40	0.425	98.7%			
	#50	0.300		93.1%		
	#60	0.250	90.2%			
	#80	0.180		84.9%		
	#100	0.150	82.0%			
	#140	0.106		76.4%		
	#170	0.090		73.8%		
	#200	0.075	70.9%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
04/20/16	JMR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-227
		REPORT DATE 04/25/16	FIELD ID TP6.2
		DATE SAMPLED 04/14/16	SAMPLED BY ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-06 depth = 7 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

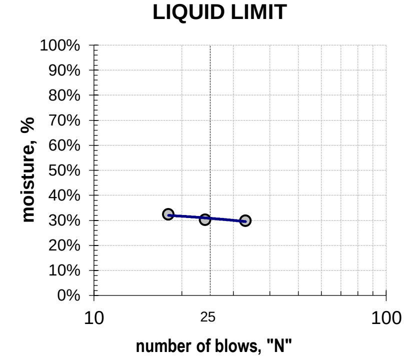
ATTERBERG LIMITS

liquid limit =	31
plastic limit =	19
plasticity index =	12

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	35.06	34.18	33.92	
dry soil + pan weight, g =	31.79	31.08	30.68	
pan weight, g =	20.82	20.83	20.69	
N (blows) =	33	24	18	
moisture, % =	29.8 %	30.2 %	32.4 %	

LIQUID LIMIT



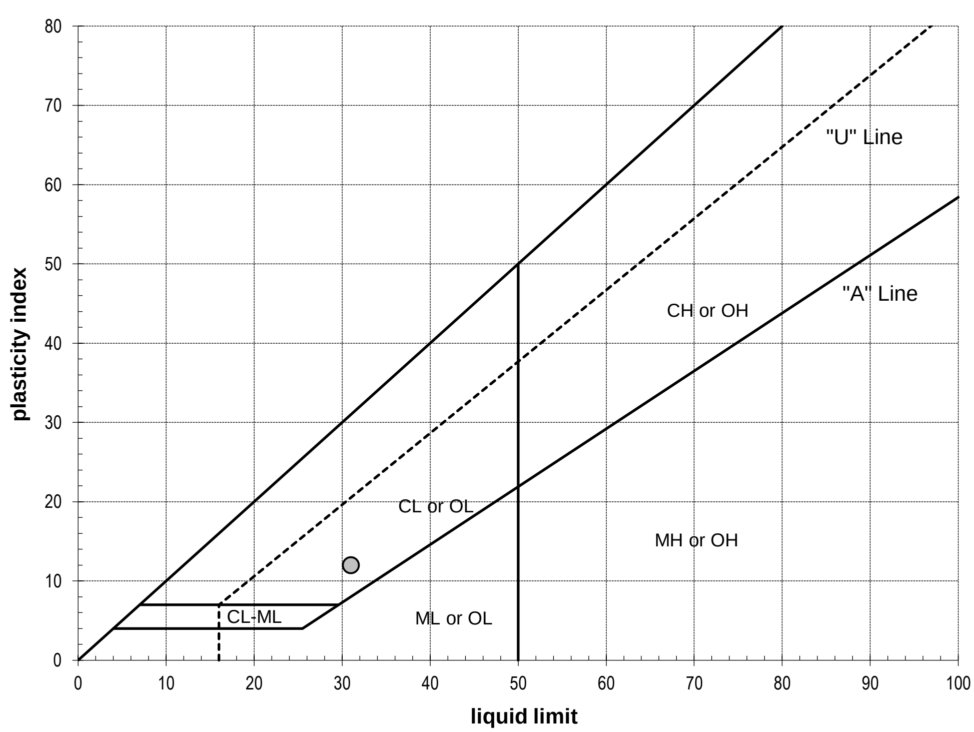
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.77	27.93		
dry soil + pan weight, g =	26.62	26.77		
pan weight, g =	20.51	20.65		
moisture, % =	18.8 %	19.0 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	29.1%
% silt and clay =	70.9%
% silt =	n/a
% clay =	n/a
moisture content =	35.1%

DATE TESTED	TESTED BY
04/20/16	MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-229
		REPORT DATE: 04/25/16	FIELD ID: TP9.2
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy SILT	Test Pit TP-09 depth = 9 feet	ML, Sandy Silt

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-4(1)

LABORATORY TEST DATA

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

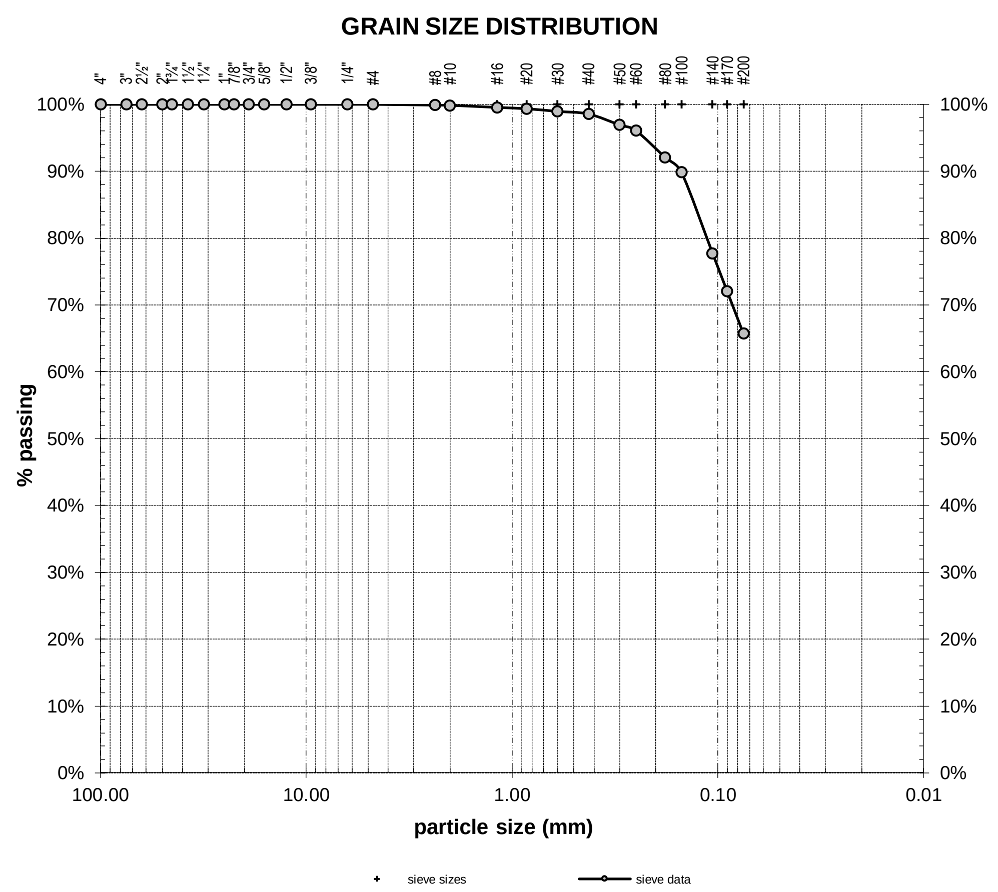
ADDITIONAL DATA

initial dry mass (g) =	199.7		
as-received moisture content =	31.1%	coefficient of curvature, C_C =	n/a
liquid limit =	30	coefficient of uniformity, C_U =	n/a
plastic limit =	27	effective size, $D_{(10)}$ =	n/a
plasticity index =	3	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 34.3%
% silt and clay = 65.7%

	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	100.0%			
SAND	#8	2.36		99.9%		
	#10	2.00	99.9%			
	#16	1.18		99.5%		
	#20	0.850	99.3%			
	#30	0.600		98.9%		
	#40	0.425	98.6%			
	#50	0.300		96.9%		
	#60	0.250	96.1%			
	#80	0.180		92.0%		
	#100	0.150	89.8%			
	#140	0.106		77.7%		
	#170	0.090		72.0%		
	#200	0.075	65.7%			



DATE TESTED	TESTED BY
04/19/16	JMR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-229
		REPORT DATE: 04/25/16	FIELD ID: TP9.2
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy SILT	Test Pit TP-09 depth = 9 feet	ML, Sandy Silt

LABORATORY TEST DATA

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

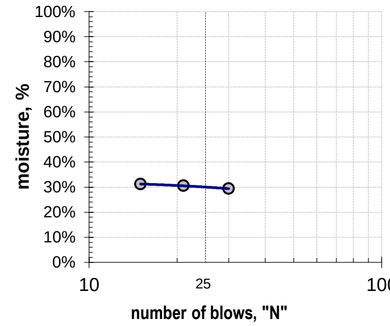
ATTERBERG LIMITS

liquid limit = 30
plastic limit = 27
plasticity index = 3

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	35.95	35.49	36.64	
dry soil + pan weight, g =	32.52	32.06	32.83	
pan weight, g =	20.86	20.82	20.65	
N (blows) =	30	21	15	
moisture, % =	29.4 %	30.5 %	31.3 %	

LIQUID LIMIT



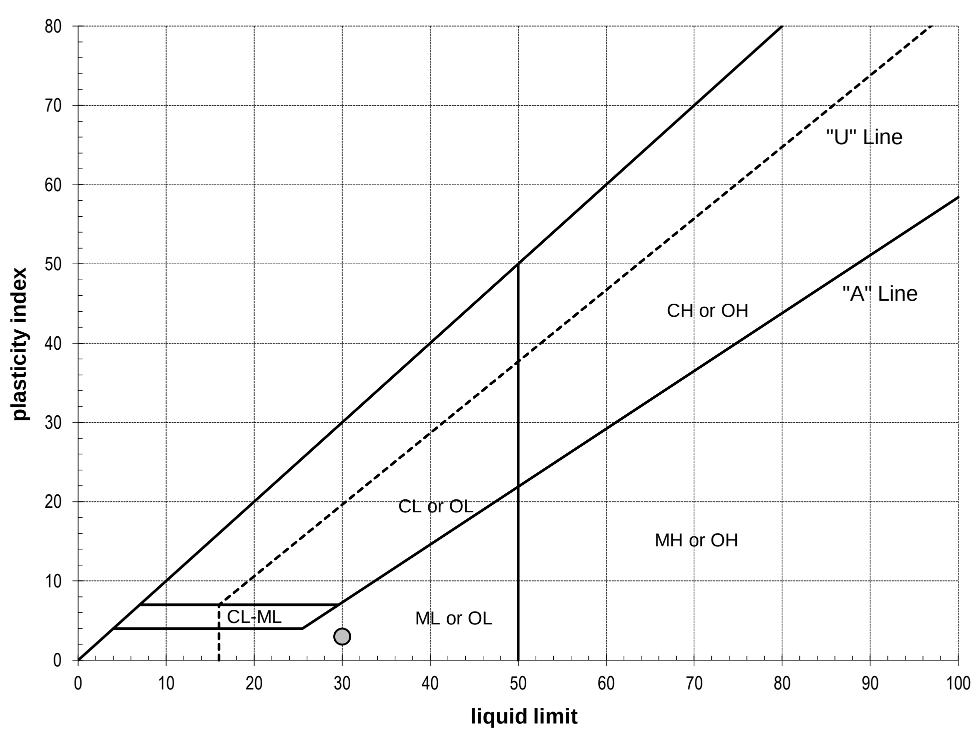
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.80	30.77		
dry soil + pan weight, g =	29.43	28.56		
pan weight, g =	20.82	20.50		
moisture, % =	27.5 %	27.4 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 34.3%
% silt and clay = 65.7%
% silt = n/a
% clay = n/a
moisture content = 31.1%

DATE TESTED	TESTED BY
04/22/16	MJR/JMR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-228
		REPORT DATE: 04/25/16	FIELD ID: TP9.1
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty GRAVEL with Sand	Test Pit TP-09 depth = 4 feet	GM, Silty Gravel with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-2-4(0)

LABORATORY TEST DATA

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

ADDITIONAL DATA

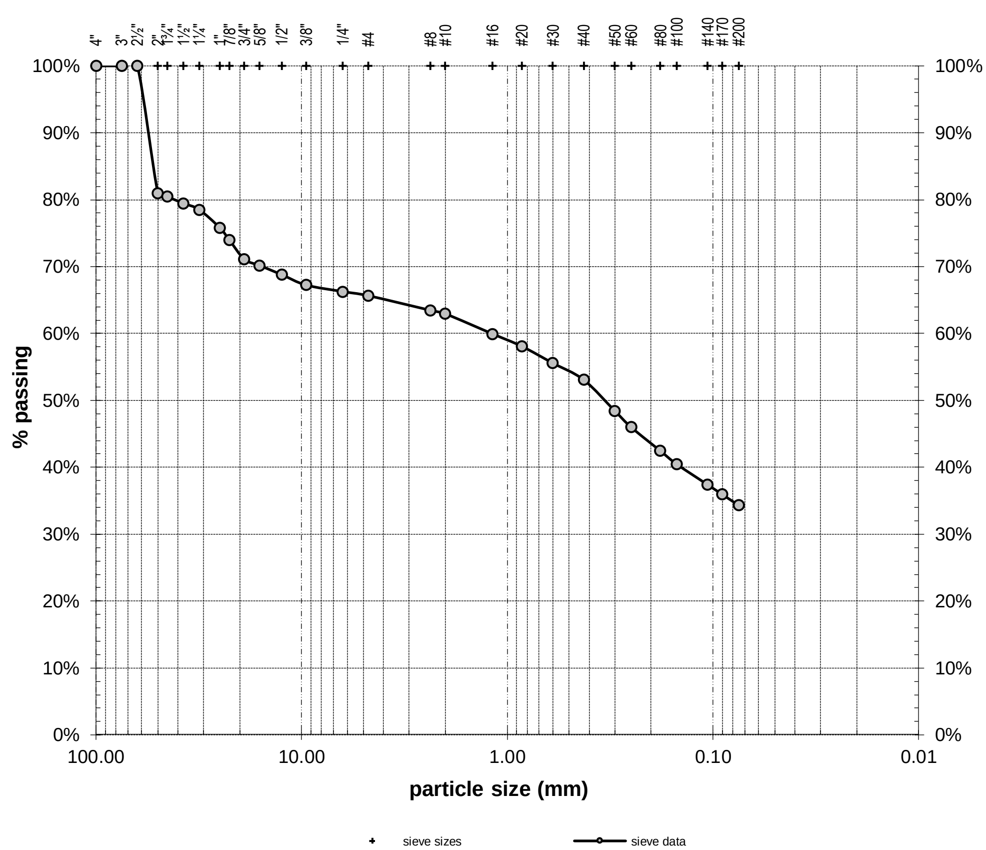
initial dry mass (g) =	2139.9		
as-received moisture content =	30.4%	coefficient of curvature, C_C =	n/a
liquid limit =	38	coefficient of uniformity, C_U =	n/a
plastic limit =	28	effective size, $D_{(10)}$ =	n/a
plasticity index =	10	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	1.191 mm

SIEVE DATA

% gravel = 34.3%
% sand = 31.3%
% silt and clay = 34.3%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0	100.0%			
	2.00"	50.0	81.0%			
	1.75"	45.0		80.4%		
	1.50"	37.5		79.5%		
	1.25"	31.5	78.5%			
	1.00"	25.0	75.8%			
	7/8"	22.4		73.9%		
	3/4"	19.0	71.1%			
	5/8"	16.0		70.2%		
	1/2"	12.5	68.8%			
	3/8"	9.50	67.2%			
	1/4"	6.30	66.3%			
	#4	4.75	65.7%			
SAND	#8	2.36		63.5%		
	#10	2.00	62.9%			
	#16	1.18		59.9%		
	#20	0.850	58.1%			
	#30	0.600		55.6%		
	#40	0.425	53.1%			
	#50	0.300		48.4%		
	#60	0.250	46.0%			
	#80	0.180		42.4%		
	#100	0.150	40.5%			
	#140	0.106		37.4%		
	#170	0.090		35.9%		
	#200	0.075	34.3%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
04/20/16	JMR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-228
		REPORT DATE: 04/25/16	FIELD ID: TP9.1
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty GRAVEL with Sand	Test Pit TP-09 depth = 4 feet	GM, Silty Gravel with Sand

LABORATORY TEST DATA

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

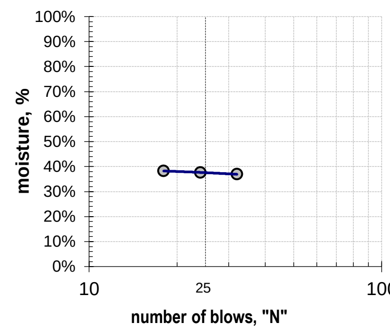
ATTERBERG LIMITS

liquid limit = 38
plastic limit = 28
plasticity index = 10

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	30.51	31.12	29.89	
dry soil + pan weight, g =	27.89	28.31	27.39	
pan weight, g =	20.80	20.83	20.86	
N (blows) =	32	24	18	
moisture, % =	37.0 %	37.6 %	38.3 %	

LIQUID LIMIT



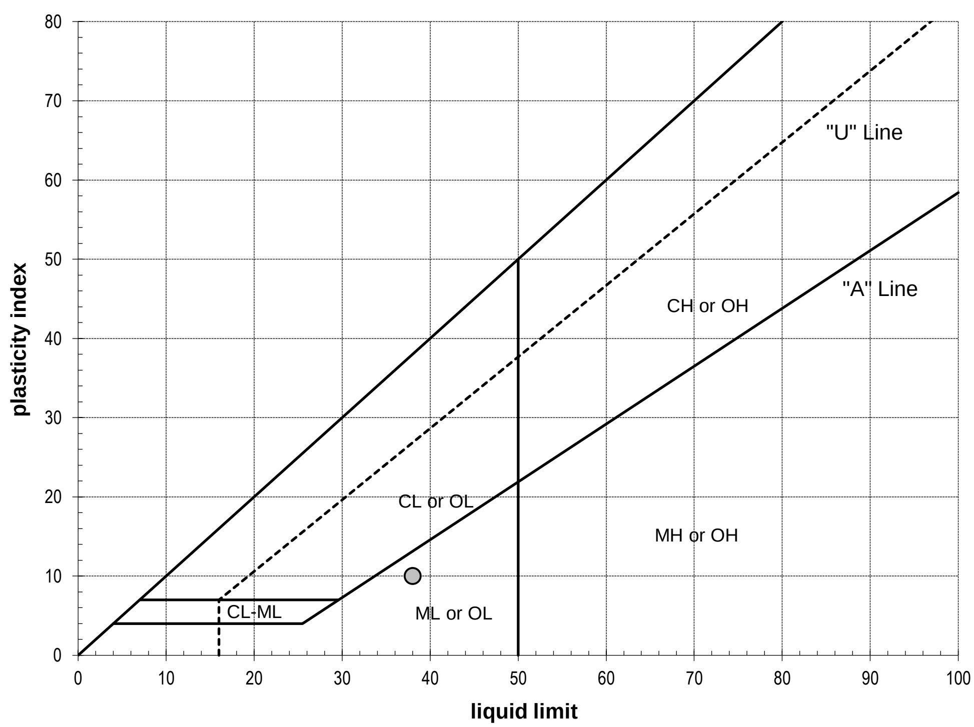
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	29.03	28.99		
dry soil + pan weight, g =	27.19	27.19		
pan weight, g =	20.71	20.77		
moisture, % =	28.4 %	28.0 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 34.3%
% sand = 31.3%
% silt and clay = 34.3%
% silt = n/a
% clay = n/a
moisture content = 30.4%

DATE TESTED	TESTED BY
04/22/16	JMR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-230
		REPORT DATE: 04/25/16	FIELD ID: TP10.1
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey SAND	Test Pit TP-10 depth = 5 feet	SC, Clayey Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(5)

LABORATORY TEST DATA

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

ADDITIONAL DATA

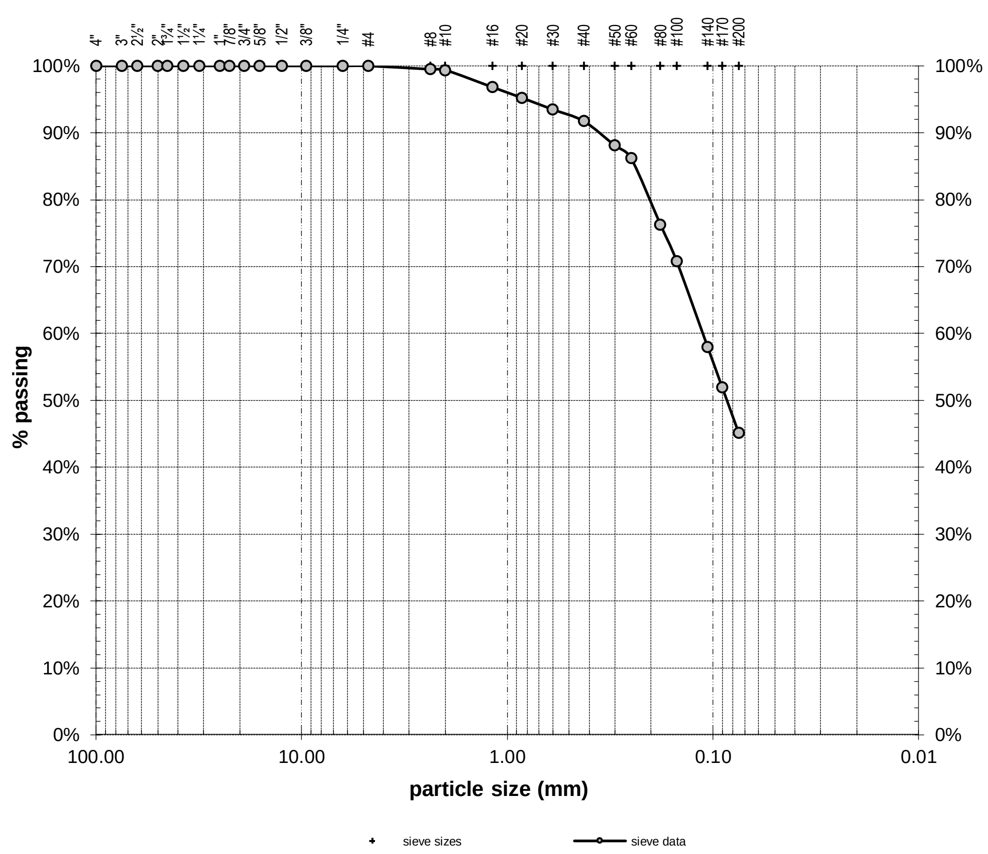
initial dry mass (g) =	192.8		
as-received moisture content =	33.4%	coefficient of curvature, C_C =	n/a
liquid limit =	46	coefficient of uniformity, C_U =	n/a
plastic limit =	27	effective size, $D_{(10)}$ =	n/a
plasticity index =	19	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	0.112 mm

SIEVE DATA

% gravel = 0.0%
% sand = 54.8%
% silt and clay = 45.2%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		99.5%		
	#10	2.00	99.4%			
	#16	1.18		96.8%		
	#20	0.850	95.2%			
	#30	0.600		93.5%		
	#40	0.425	91.7%			
	#50	0.300		88.1%		
	#60	0.250	86.2%			
	#80	0.180		76.3%		
	#100	0.150	70.8%			
	#140	0.106		58.0%		
	#170	0.090		51.9%		
	#200	0.075	45.2%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
04/20/16	JMR

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PRELIMINARY

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-230
		REPORT DATE: 04/25/16	FIELD ID: TP10.1
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey SAND	Test Pit TP-10 depth = 5 feet	SC, Clayey Sand

LABORATORY TEST DATA

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

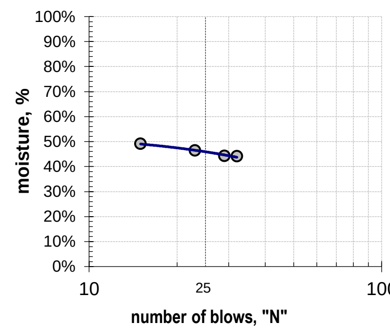
ATTERBERG LIMITS

liquid limit =	46
plastic limit =	27
plasticity index =	19

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.57	28.58	28.09	28.75
dry soil + pan weight, g =	26.20	26.16	25.73	26.15
pan weight, g =	20.83	20.69	20.65	20.86
N (blows) =	32	29	23	15
moisture, % =	44.1 %	44.2 %	46.5 %	49.2 %

LIQUID LIMIT



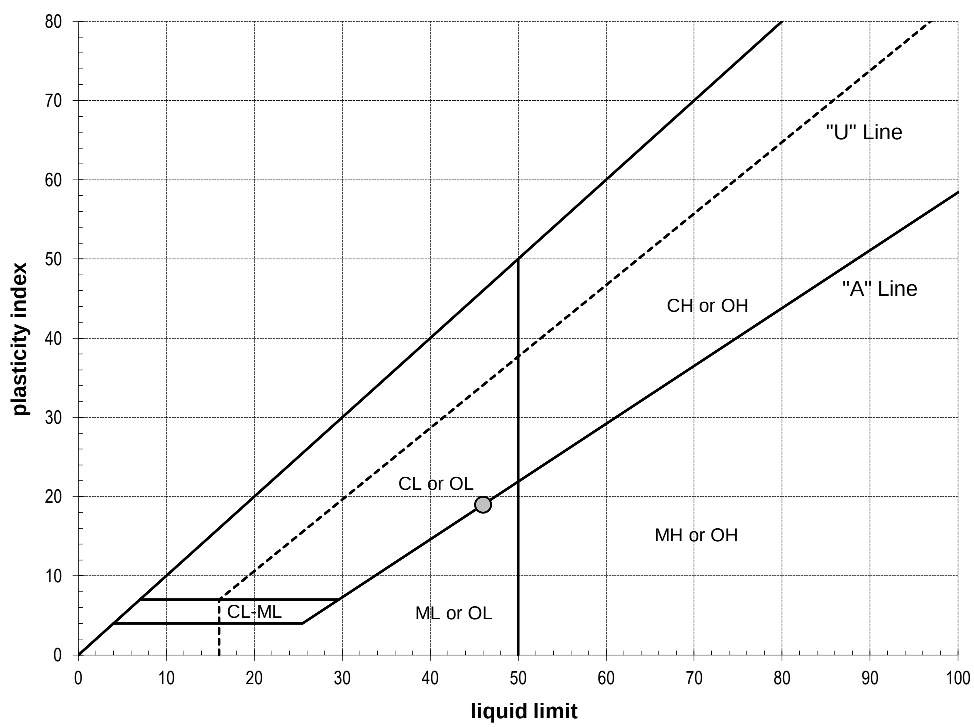
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.22	30.25		
dry soil + pan weight, g =	28.92	28.22		
pan weight, g =	20.58	20.77		
moisture, % =	27.6 %	27.3 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	54.8%
% silt and clay =	45.2%
% silt =	n/a
% clay =	n/a
moisture content =	33.4%

DATE TESTED	TESTED BY
04/19/16	JMR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain Master Plan Pods B3 and E1 2817 NE Ingle Road Camas, Washington	Mr. Ralph Emerson CLB Washington Option Solutions, LLC 2817 NE Ingle Road Vancouver, Washington 98607	16081	S16-231
		REPORT DATE: 04/25/16	FIELD ID: TP10.2
		DATE SAMPLED: 04/14/16	SAMPLED BY: ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty SAND	Test Pit TP-10 depth = 10 feet	SM, Silty Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-2-4(0)

LABORATORY TEST DATA

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

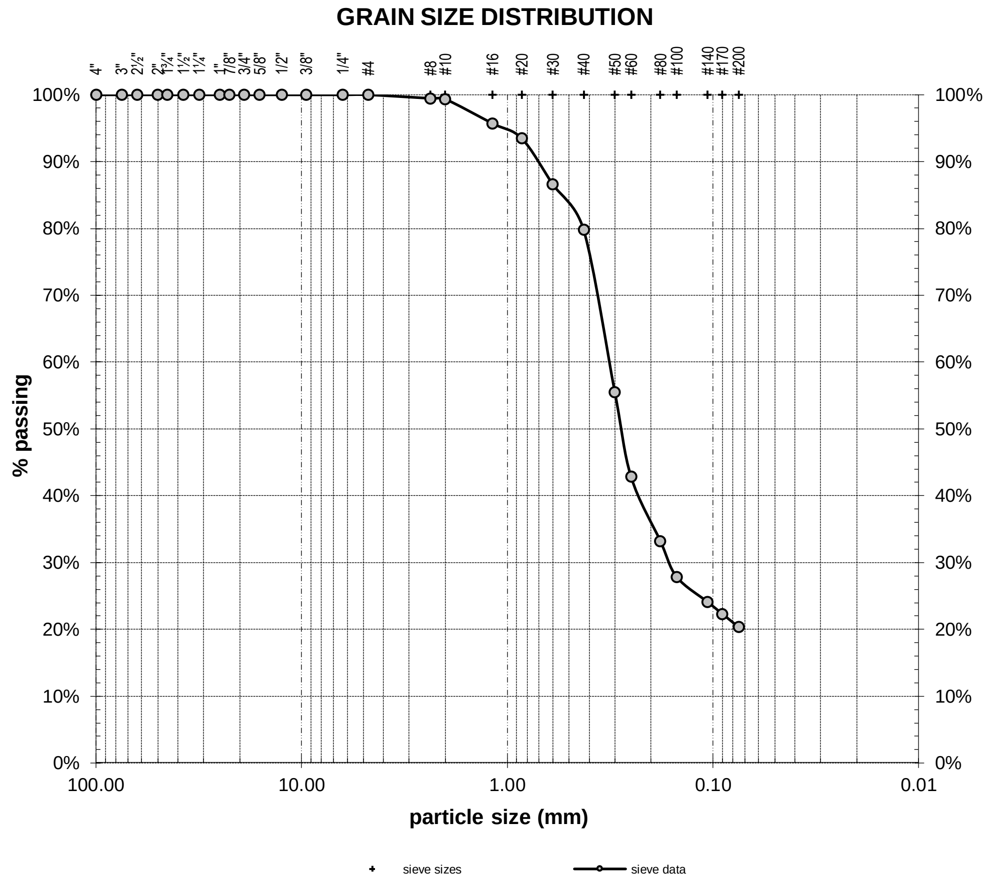
ADDITIONAL DATA

initial dry mass (g) =	195.1		
as-received moisture content =	31.6%	coefficient of curvature, C_C =	n/a
liquid limit =	0	coefficient of uniformity, C_U =	n/a
plastic limit =	0	effective size, $D_{(10)}$ =	n/a
plasticity index =	0	$D_{(30)}$ =	0.161 mm
fineness modulus =	n/a	$D_{(60)}$ =	0.320 mm

SIEVE DATA

% gravel = 0.0%
% sand = 79.6%
% silt and clay = 20.4%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		99.5%		
	#10	2.00	99.3%			
	#16	1.18		95.7%		
	#20	0.850	93.5%			
	#30	0.600		86.6%		
	#40	0.425	79.8%			
	#50	0.300		55.5%		
	#60	0.250	42.8%			
	#80	0.180		33.2%		
	#100	0.150	27.9%			
	#140	0.106		24.1%		
	#170	0.090		22.3%		
	#200	0.075	20.4%			



DATE TESTED	TESTED BY
04/20/16	JMR

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-1
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**ENGINEER**	**DATE**
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	not surveyed	not encountered	8:10	8:40

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of sod and organic-rich topsoil.					
				ML		Brown sandy SILT, moist, medium stiff, low plasticity. [Soil Type 1]					
5				ML		Light brown sandy SILT with grey and orange mottling, moist, medium stiff to stiff, low plasticity. Cemented sandy lenses, minor iron staining. [Soil Type 1]					
10				SM		Bluish-grey silty SAND, moist to wet, medium dense to dense, non-plastic. [Soil Type 3]					
15						Bottom at 13.0 feet. Groundwater not encountered.					

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-2
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**ENGINEER**	**DATE**
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	not surveyed	not encountered	8:45	9:25

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 18 to 24 inches of dark brown organic-rich silt and topsoi mixed with angular to subangular basalt cobbles (2-inch to 6-inch).					
	TP2.1		A-6(8)	CL		Orange-brown sandy lean CLAY, moist, stiff, moderate plasticity. [Soil Type 1]	25.4	62.6	38	23	
5											
	TP2.2		A-2-7(0)	GC		Orange-brown clayey GRAVEL with sand, moist, dense, high plasticity (Weathered Conglomerate). [Soil Type 3]	17.7	13.4	52	28	
10											
						Bottom at 11.0 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-3
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	not encountered	9:30	10:15

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of black topsoil mixed with angular basalt cobbles (4-inch to 12-inch).					
				SC		Grey-brown clayey SAND with gravel, moist, dense, low to moderate plasticity. Moderate amount of angular basalt cobbles (4-inch to 12-inch). [Soil Type 2]					
5				GC		Orange-brown clayey GRAVEL with sand, moist, dense, high plasticity (Weathered Conglomerate). 1/2-inch to 6-inch gravels. [Soil Type 3]					
10						Bottom at 10.0 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-4
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	not encountered	10:20	11:00

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of topsoil and root zone.					
				ML		Light brown sandy SILT, moist, medium stiff, low plasticity. [Soil Type 1]					
5											
				GC		Orange-brown clayey GRAVEL with sand, moist, dense, high plasticity (Weathered Conglomerate). 1/2-inch to 6-inch gravels. [Soil Type 3]					
10											
						Bottom at 11.0 feet. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-5
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	not encountered	11:10	12:00

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of topsoil and root zone.					
				ML		Light brown sandy SILT, moist, medium stiff, low plasticity. [Soil Type 1]					
				ML		Light brown sandy SILT with grey and orange mottling, moist, medium stiff to stiff, low plasticity. Cemented sandy lenses, minor iron staining. Sparse 6-inch cobbles at depth. [Soil Type 3]					
5											
10						Bottom at 10.0 feet Heavy seeps at 2.5 feet					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-6
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	not encountered	12:20	12:50

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of topsoil and root zone.					
	TP6.1		A-2-4(0)	SC		Reddish-brown clayey SAND with gravel, moist, dense, low plasticity. Sparse angular to subangular basalt gravels (<3 inches). Two large 4-foot boulders encountered. [Soil Type 2]	25.0	34.1	33	10	
5											
	TP6.2		A-6(7)	CL		Gold-brown lean CLAY with sand, moist, stiff, low plasticity. Minor iron banding. [Soil Type 3]	35.1	70.9	31	12	
10						Bottom at 10.0 feet Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-7
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	not encountered	12:55	1:25

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 18 inches of topsoil and tree roots.					
				SC		Reddish-brown clayey SAND with gravel, moist, dense, low plasticity. Sparse angular to subangular basalt gravels (<3 inches). [Soil Type 2]					
				CL		Gold-brown lean CLAY with sand, moist, stiff, low plasticity. Minor iron banding. [Soil Type 3]					
5											
10						Bottom at 10.0 feet Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-8
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	not encountered	1:30	1:50

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						Large angular basalt boulders on surface. Approximately 18 inches of topsoil and tree roots.					
				SC		Reddish-brown clayey SAND with gravel, moist, dense, low plasticity. Sparse angular to subangular basalt gravels (<3 inches). [Soil Type 2]					
				CL		Gold-brown lean CLAY with sand, moist, stiff, low plasticity. Minor iron banding. [Soil Type 3]					
5											
10											
						Bottom at 11.0 feet. Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-9
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	not encountered	1:55	2:15

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						Large angular basalt boulders on surface. Approximately 18 inches of topsoil and tree roots.					
				GM		Reddish-brown silty GRAVEL with sand, moist, dense, low plasticity. Moderate amount of <3-inch gravels. [Soil Type 2]					
	TP9.1		A-2-4(0)				30.4	34.3	38	10	
5											
				ML		Orange-gold sandy SILT, moist, stiff, low plasticity. [Soil Type 3]					
	TP9.2		A-4(1)				31.1	65.7	30	3	
10											
						Bottom at 11.0 feet. Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-10
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**ENGINEER**	**DATE**
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	not surveyed	not encountered	2:20	2:50

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						Large basalt boulders on surface. Approximately 12 inches of topsoil and tree roots.					
				GM		Reddish-brown silty GRAVEL with sand, moist, dense, low plasticity. Moderate amount of <3-inch gravels. [Soil Type 2]					
				SC to SM		Orange-gold clayey SAND, moist, stiff, moderate plasticity. Transitions to silty sand (decreasing plasticity) with depth. [Soil Type 3]					
5	TP10.1		A-7-6(5)				33.4	45.2	46	19	
10	TP10.2		A-2-4(0)				31.6	20.4	0	0	
15						Bottom at 14.0 feet. Groundwater not encountered.					

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-11
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	4/14/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	not encountered	2:55	3:15

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				GP		Large basalt boulder talus. 6-inch to 4-foot angular basalt boulders with minimal matrix soil. Dry, loose, non-plastic. [Soil Type 2]					
5				SC to SM		Orange-gold clayey SAND, moist, stiff, moderate plasticity. Transitions to silty sand (decreasing plasticity) with depth. [Soil Type 3]					
10						Bottom at 10.0 feet Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-12
PROJECT LOCATION: Camas, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	ENGINEER: ASR	DATE: 8/15/2016
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: not surveyed	GROUNDWATER DEPTH: static groundwater not encountered	START TIME:	FINISH TIME:

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of topsoil and tree roots.					
				CL		Orange-brown sandy lean CLAY, moist, stiff, moderate plasticity.					
5				GC		Orange-brown clayey GRAVEL with sand, moist, dense, moderate plasticity (Weathered Conglomerate). Rounded to sub-rounded 1/2" to 6" gravels in partially cemented clay matrix. Wet at 15 feet. Very dense (almost refusal) at 19 feet.					
10											
15											
20						Bottom at 19.0 feet. Groundwater not encountered. Very wet at 15 feet.					

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-13
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**ENGINEER**	**DATE**
Camas, Washington	L&S Contractors	Excavator	ASR	8/15/2016
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	not surveyed	static groundwater not encountered		

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of topsoil and tree roots.					
				GC		Orange-brown clayey GRAVEL with sand, moist, dense, moderate plasticity.					
				CL		Tan lean CLAY with fine sand, moist, medium stiff, low plasticity. Minor iron banding. No gravels observed.					
5											
10											
				CL		Orange-brown lean CLAY with gravel, moist, stiff, moderate plasticity. Iron banding.					
15											
						Bottom at 18.0 feet. Groundwater not encountered.					
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-14
PROJECT LOCATION: Camas, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	ENGINEER: ASR	DATE: 8/15/2016
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: not surveyed	GROUNDWATER DEPTH: static groundwater not encountered	START TIME:	FINISH TIME:

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of topsoil and root zone.					
				ML		Brown sandy SILT, moist, medium stiff, low plasticity.					
5				CL		Tan lean CLAY with fine sand, moist, medium stiff, low plasticity. Minor iron banding. No gravels observed.					
10				CL		Tan- brown lean to fat CLAY with minor cemented gravel, moist, very stiff, moderate to high plasticity,					
15				GC		Orange-brown clayey GRAVEL with sand, partially cemented, moist, dense, moderate plasticity (Weathered Conglomerate). Rounded to sub-rounded gravels. Very dense (almost refusal) at 17 feet. Slow excavating at depth.					
20						Bottom at 19.0 feet. Groundwater not encountered.					

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-15
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**ENGINEER**	**DATE**
Camas, Washington	L&S Contractors	Excavator	ASR	8/15/2016
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	not surveyed	static groundwater not encountered		

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of topsoil and root zone.					
				GM		Reddish-brown silty GRAVEL with sand, moist, dense, low plasticity.					
5				GC		Orange-brown clayey GRAVEL with sand, partially cemented, moist, dense, moderate plasticity (Weathered Conglomerate). Slow excavating.					
10				SC		Orange-gold clayey SAND, moist, stiff, moderate plasticity. Transitions to tan color, decreasing plasticity with depth. No gravels observed.					
15											
20						Bottom at 20 feet. Groundwater not encountered.					

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-16
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	8/15/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	static groundwater not encountered		

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						Large angular basalt boulders on surface. Approximately 24 inches of topsoil and tree roots.					
				SC		Reddish-brown clayey SAND with gravel, damp, dense, low plasticity. Sparse angular to subangular basalt gravels (<3").					
5											
				SC		Orange-gold clayey SAND, moist, dense, low plasticity. Minor iron banding. No gravels observed.					
10											
15											
20						Bottom at 19.0 feet. Groundwater not encountered.					

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-17
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	8/15/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	static groundwater not encountered		

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of topsoil and root zone.					
				ML		Brown sandy SILT, moist, medium stiff, low plasticity.					
5											
				GC		Orange-brown clayey GRAVEL with sand, partially cemented, moist, dense, moderate plasticity (Weathered Conglomerate). Rounded to sub-rounded gravels. Very dense (almost refusal) at 19 feet. Slow excavating at depth.					
10											
15											
						Bottom at 19.0 feet. Groundwater not encountered.					
20											

PRELIMINARY

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Green Mountain	CLB Washington Option Solutions, LLC		16081	TP-18
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	ASR	8/15/2016
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	not surveyed	static groundwater not encountered		

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 12 inches of topsoil and root zone.					
				ML		Brown sandy SILT, moist, medium stiff, low plasticity.					
5											
				GC		Orange-brown clayey GRAVEL with sand, partially cemented, moist, dense, moderate plasticity (Weathered Conglomerate). Rounded to sub-rounded gravels. Very dense (almost refusal) at 19 feet. Slow excavating at depth.					
10											
15											
						Bottom at 18.0 feet. Groundwater not encountered.					
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			

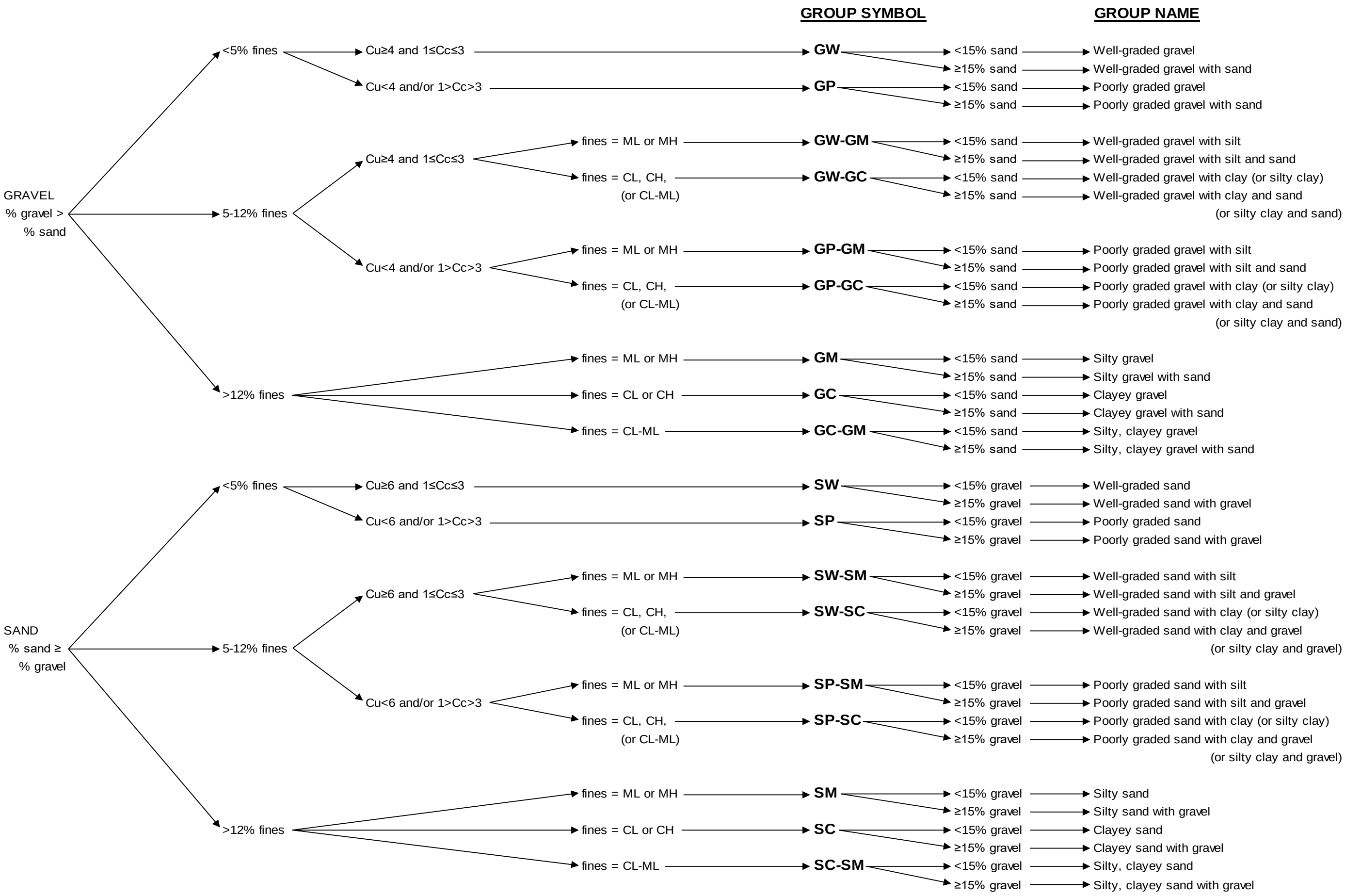
Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

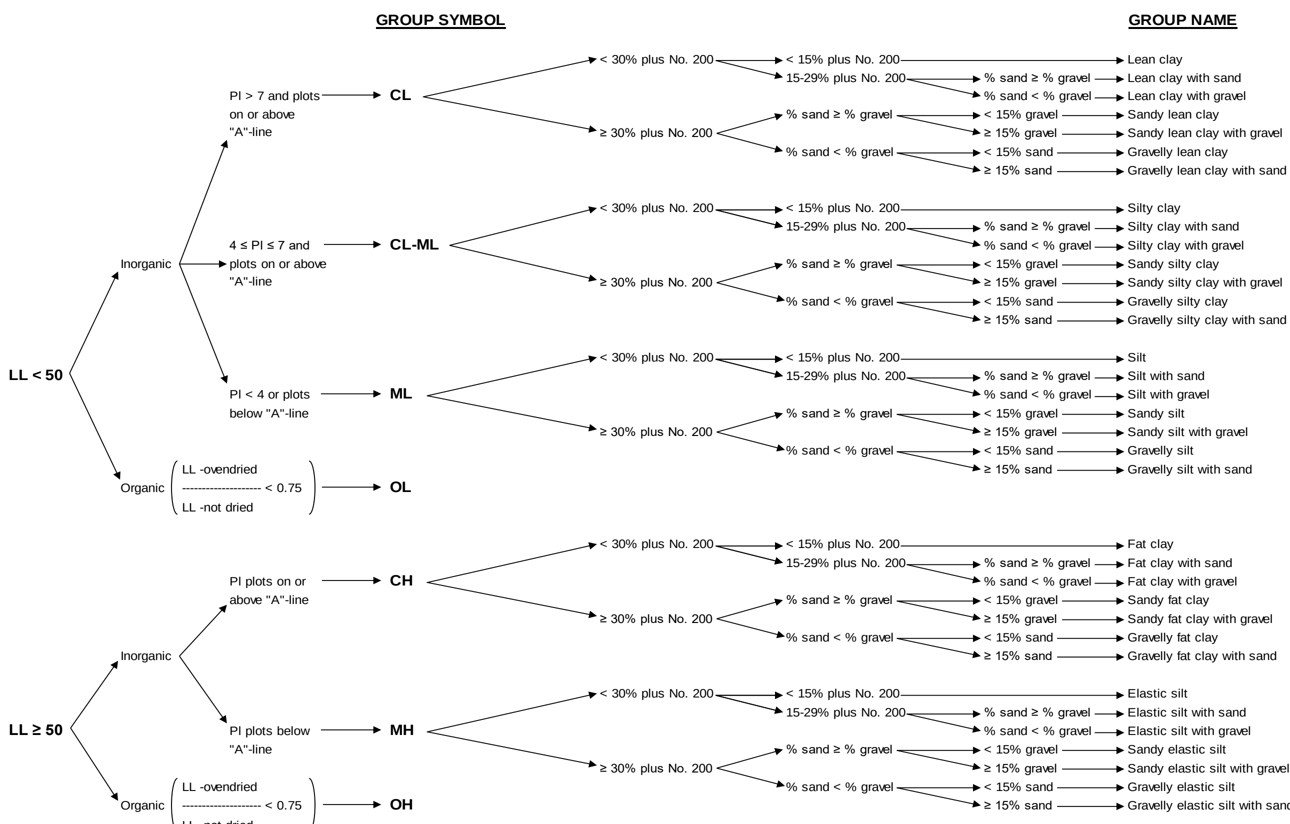
General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00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

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PHOTO LOG
Green Mountain Pods B3 & E1
Camas, Washington

Aerial View of Green Mountain – Phase 1
Facing Northeast

Aerial View of Pod E1 (foreground-right) and Pod B3 (background-center)
Facing West

Weathered Conglomerate soils in TP-2

Weathered Basalt soils over Weathered Conglomerate soils in TP-3

Sandy Weathered Conglomerate soils in TP-10

Weathered Talus soils in TP-11

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Date: September 27, 2016
Project: Green Mountain Pods B3 & E1
Camas, Washington

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

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

Report Conclusions and Preliminary Nature

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

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

Additional Investigation and Construction QA/QC

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

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

Collected Samples

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

Report Contents

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

Report Limitations for Contractors

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

Report Ownership

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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.