EXHIBIT 29

Geotechnical Critical Areas Report

Green Mountain North

Camas, Washington

September 28, 2017

Geotechnical ▪ Environmental ▪ Special Inspections

Columbia West Engineering, Inc

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

www.columbiawestengineering.com

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Columbia West
Engineering, Inc

GEOTECHNICAL CRITICAL AREAS REPORT
GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON

Prepared For:	**Green Mountain Land, LLC** **Mr. John Schmidt** **333 S State Street V201** **Lake Oswego, Oregon 97034**
Site Location:	**East of NE Ingle Road and NE 43rd Circle** **Parcel Nos. 17170400, 171727000, 172341000, 171730000** **Camas, Washington**
Prepared By:	**Columbia West Engineering, Inc.** **11917 NE 95th Street** **Vancouver, Washington 98682** **Phone: 360-823-2900** **Fax: 360-823-2901**
Date Prepared:	**September 28, 2017**

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1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Green Mountain Land to prepare a geotechnical critical areas report for proposed development on tax parcel numbers 171704000, 171727000, 172341000, and 171730000 in Camas, Washington. The purpose of the investigation was to observe and assess subsurface soil conditions at specific locations and provide subsequent appropriate geotechnical engineering analyses to support property development feasibility, planning, and design recommendations. The specific scope of services was outlined in a proposal contract dated January 19, 2017. 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 F.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is comprised of tax parcel numbers 171704000, 171727000, 172341000, and 171730000 totaling approximately 124.5 acres. The parcels are located east of the intersection of NE Ingle Road and NE 43rd Circle in Camas, Washington. The regulatory jurisdictional agency is the City of Camas, Washington. The approximate latitude and longitude are N 45°39'10.93" and W 122°27'41.92", and the legal description is a portion of the SE and SW ¼ of Section 17 and NE ¼ of Section 20, T2N, R3E, Willamette Meridian.

1.2 Proposed Development

Review of preliminary site plans provided by Olson Engineering, the site civil engineering firm, indicates that proposed development may consist of approximately 160 residential lots, neighborhood streets, underground utilities, and stormwater management facilities. Columbia West has not reviewed preliminary grading plans but understands that minor cut and fill is proposed at the property. This report provides geotechnical analysis and recommendations for site development as shown on Figure 2. 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 on the western and southwest flank of a basaltic andesite 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 majority of the proposed development area is underlain by the Basaltic Andesite of Green Mountain (Qbgm), a Pleistocene-aged cinder cone comprised of olivine-phyric, nonscoriaceous, platy lava and weathered basaltic ash. The cinder cone that forms Green Mountain represents the northern portion of the Quaternary Boring Volcanic Field which intruded vertically through the conglomeratic members, solidified, and weathered in place.

Geologic mapping indicates that a small portion of the study area located along the western base of Green Mountain is underlain by Pleistocene to Pliocene unconsolidated to semi-consolidated, pebble to cobble conglomerate (QTc). The unnamed conglomerate unit is lithologically similar to the Pliocene or late Miocene Troutdale Formation, differing primarily in age of emplacement, degree of weathering, and the presence of hyaloclastite interbeds. Previously published geologic mapping has identified this unit as the Troutdale Formation.

The *Web Soil Survey* (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2017 Website) indicates the site is underlain by Olympic stony clay loam series soils. Olympic stony clay loam soils 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.

3.0 REGIONAL SEISMOLOGY

Recent research and subsurface mapping investigations within the Pacific Northwest 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 15 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.

Lacamas Lake-Sandy River Fault Zone

The northwest-trending Lacamas Creek Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 4 miles southeast of the site, and form part of the northeastern margin of the Portland basin. According to *Geology and Groundwater Conditions of Clark County Washington* (USGS Water Supply Paper 1600, Mundorff, 1964) and the *Geologic Map of the Lake Oswego Quadrangle* (Oregon DOGAMI Series GMS-59, 1989), the Lacamas 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.

Cascadia Subduction Zone

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

4.0 GEOTECHNICAL FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance, three soil borings (SB-1 through SB-3), and 14 test pits (TP-1 through TP-14) was conducted at the site on multiple dates, February 14, March 2, and March 3, 2017. 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.

4.1 Surface Investigation and Site Description

The site lies on the west and southwest flank of Green Mountain. The subject site is primarily undeveloped with the exception of two large residential structures with detached outbuildings near the western site boundary (with site address 4601 NE Ingle Road), two Bonneville Power Administration (BPA) easements trending east-west at the northern boundary and northwest through the center of the proposed development, as well as a municipal water well and well house used by Clark Public Utilies (CPU) located in the central portion of the site. The subject site is densely vegetated with large fir and deciduous trees and associated understory vegetation.

Generally, the development area can be broken into three terrain areas from west to east; the western valley, a central basalt bench, and the cinder cone slopes in the east. Elevations range from approximately 200 to 250 feet in the western valley area along Ingle Road, 250 to 400 feet in the central bench area, and 400 to 790 feet on the cinder cone. Proposed development is proposed up to approximately 550 feet in elevation in the southeast area.

Slope grades within the central bench area range from approximately 3 to 20 percent in the central north area of the site up to 45 percent in the southwestern area of the site. Near-vertical outcrops of basalt approximately 20 to 30 feet in height exist in the southwest area and separate the bench from the valley floor. Slopes above 400 feet on the west face of the Green Mountain cinder range from 25 to 40 percent.

The property has been impacted previously by the construction of Bonneville Power Administration transmission towers, old road cuts, a well house with associated conveyance piping, and the power installation and easement to the residential parcel.

4.2 Subsurface Exploration and Investigation

Test pit explorations TP-1 through TP-14 were advanced at the site to a maximum depth of 15 feet using a track-mounted excavator on February 14, 2017. Soil Boring explorations SB-1 through SB-3 were conducted to a maximum depth of 55 feet below ground surface using a track-mounted CME-850XR mud rotary drill rig on March 2, and March 3, 2017. 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 site is generally covered with 12 to 24-inches of topsoil and associated organic-rich root zone material at the locations observed. Underlying the topsoil layer, fine-textured silt and clay soils resembling native USDA Olympic stony clay loam were encountered. Olympic soils were generally underlain by weathered to competent basaltic andesite bedrock in locations observed. Subsurface lithology was reasonably consistent at all explored locations and may generally be described by soil types identified in the following text.

Soil Type 1 – Olympic Series Soils - Sandy *Lean CLAY to Elastic SILT with Sand*

Soil Type 1 was observed to consist of brown, moist, medium-dense to dense, silty SAND with gravel, elastic SILT with sand to sandy elastic SILT, SILT with sand, sandy lean CLAY, and sandy fat CLAY. Soil Type 1 represents weathered basaltic andesite bedrock at different phases in the weathering process. Soil Type 1 was encountered below the topsoil layer in all test pit and soil boring explorations except TP-4 and TP-10. Soil Type 1 extended to observed depths of 1 to 40 feet below ground surface.

Analytical laboratory testing conducted upon representative soil samples indicate approximately 38 to 80 percent by weight passing the No. 200 sieve and in situ moisture content ranging from 29 to 65 percent. Atterberg test results indicated a liquid limit ranging from 0 to 66 percent and a plasticity index ranging from 0 to 40 percent. Soil Type 1 is classified as SM, silty SAND with gravel, MH, elastic SILT with sand to sandy elastic SILT, ML, SILT with sand, CL, sandy lean CLAY, and CH, sandy fat CLAY according to USCS specifications and; A-6(2), A-7-5(3), A-7-5(11), A-7-6(35), A-7-5(21),A-4(0), A-6(5), A-7-6(16), and A-7-6(35) according to AASHTO specifications.

Soil Type 2: Basaltic Andesite Bedrock

Competent bedrock was encountered underlying Soil Type 1 in test pit explorations TP-1 through TP-3, TP-5, TP-8, TP-9, TP-11, TP-13, TP-14 and soil boring SB-1, SB-2, and SB-3. Soil Type 2 was encountered directly beneath the topsoil layer in test pit explorations TP-4 and TP-10. Soil Type 2 extended to the maximum depth of exploration in all test pit and soil boring explorations. The bedrock encountered generally resembled the descriptions of the Basaltic Andesite of Green Mountain (Qbgm), a Pleistocene-aged cinder cone comprised of olivine-phyric, nonscoriaceous, platy lava. The bedrock consisted of fine grained, olivine-phyric, basaltic andesite undergoing various phases of weathering with trace quartzite clast inclusions and minor vesicular texture.

4.2.2 Groundwater

Groundwater was encountered in test pit TP-5 at approximately 5 feet below ground surface and seepage was present in test pit TP-6 at approximately 9 feet below ground surface. According to *Clark County Maps Online*, the depth to groundwater in the vicinity of the subject site is approximately 10 to 30 feet below ground surface depending upon location and ground surface elevation. Due to the mud-rotary soil boring method, groundwater elevations were not ascertained from soil boring explorations. Ponded surface water was observed adjacent to NE Ingle Road near the intersection of NE 43rd Circle and significant seepage was observed above this area at an approximate elevation of 300 feet amsl and followed an old road cut down to the pond area.

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 two of these hazards: erosion hazards and areas of potential instability or slopes greater than 15 percent. Erosion hazards and slopes greater than 15 percent are generally mapped above the 400-foot elevation contour in the cinder cone portion of the development. Areas of potential slope instability are mapped in the area of near-vertical basaltic andesite outcrops on the southwest boundary between the central bench area and valley floor. The hazard map identifies the site as having very low to low susceptibility to soil liquefaction in the event of an earthquake.

Columbia West conducted a geologic hazard assessment for the subject site to assess the presence of potential geologic hazards and the effect proposed site development may have on identified hazards. The results of the geologic hazard review for slope and landslide hazards, seismic hazards, and erosion hazards are discussed in the following sections.

5.1 Slope and Landslide Hazards

To assess whether slope or landslide hazards are present at the site and to provide mitigation recommendations where hazards exist, Columbia West conducted a hazard assessment consisting of literature review, site reconnaissance, and slope stability analysis.

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 is expected to consist of Basaltic Andesite of Green Mountain (Qbgm), a Pleistocene-aged cinder cone comprised of olivine-phyric, nonscoriaceous, platy lava within the site boundary. Active or historic landslides are not mapped within the study area or 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 mapped as (2) areas of potential instability because of underlying geologic conditions and physical characteristics associated with steepness.

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 throughout the majority of the site with the exception of the mid-northern area where steep slopes are not present.

5.1.2 Slope and Subsurface Reconnaissance

To observe geomorphic conditions, Columbia West conducted visual and physical reconnaissance of slopes on the property. As previously described, test pit and soil boring explorations conducted throughout the site indicate approximately 1 to 40 feet of sandy lean clay to elastic silt with sand and trace areas of fat clay soils underlain by basaltic andesite bedrock to the maximum explored depth. As described in Section 4.1, *Surface Investigation and Site Description,* slope grades within the central bench area range from approximately 3 to 20 percent in the central-north area of the site up to 45 percent in the southwestern area of the site. Near-vertical outcrops of basalt approximately 20 to 30 feet in height exist in the southwest area and separate the bench from the valley floor. Slopes above 400 feet on the west face of the Green Mountain cinder cone average approximately 33 percent.

Most portions of the slopes are generally planar with no observed evidence of instability. Slopes currently support heavy vegetation consisting of established deep-rooted coniferous and deciduous trees with mixed understory vegetation. There was no observed direct evidence of large-scale, mass slope movements or historic landslides. No landslide debris was observed within subsurface soils explored near the slopes.

5.1.3 Rock-fall

Rock-fall hazards may be present in areas beneath near-vertical outcrops of basalt bedrock between the central bench and valley floor area. Rock-fall hazards are possible, but unlikely within the cinder cone area due to slope inclination and the lack of significant outcropping of boulders or intact rock. No marked trees, disturbed ground, or other evidence of recent rock-fall activity was observed within the proposed development area. 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 slopes.

Rock-fall hazard is typically associated with exposed 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.

Care should be taken during grading to protect down-gradient, developed areas from potential rock-fall. A small portion of Pods B3 and E1, which are proposed to be developed as part of Green Mountain - Phase 1, lie below proposed development areas in the Green Mountain North development. Although rock-fall from development of Green Mountian North is not anticipated to be generated, nor anticipated to impact the area proposed for Pods B3 and E1, further assessment should be completed once design elements and boundaries are known. 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.

5.1.4 Slope Stability Analysis

Detailed computer analysis of the slopes described above was performed using the program SLOPE/W, by Geo-Slope International. The purpose of the analysis was to assess slope stability, quantify factors of safety, and provide recommendations for setback distances for residential development in proximity to the slopes.

SLOPE/W uses limit equilibrium analyses to determine slope stability. The Morgenstern-Price method of slices, which satisfies force and moment equilibrium, was used to calculate the factor of safety against slope failure. Soils within a given layer were considered to be homogenous and isotropic. Radial, block, and composite slip surfaces were analyzed in determining critical slip surfaces. Slope stability methodology, input parameters, and program output, including critical failure surfaces and factors of safety, are provided in Appendix D.

5.1.5 SLOPE/W Input

Slope Geometry and Loading

The slope profile locations selected for slope stability analyses (A-A’ and B-B’) are indicated on Figure 2. The slope cross-sections are shown in Appendix D. The cross-sections were derived from a combination of topographical data obtained from Clark County Maps Online, site-specific survey information, and hand-held inclinometer measurements. The approximate delineations of individual soil layers were based on interpretation of the soil boring and test pit results, laboratory test results, and visual observation of the subject site. Selection of the cross-section locations were based upon several factors, including slope height, length, grade, and proximity to proposed development areas. Proposed grading contours were not available at the time of this analysis and, therefore, were not analyzed.

Soil Characteristics

Soil strength parameters and unit weights selected for computer modeling were based upon in-situ soil testing, SPT blow counts, analytical laboratory analysis, research of existing soil mechanics data, and visual observation. Input values were generally selected to provide for conservative analyses. SLOPE/W utilizes the individual soil layer moist unit weight, saturated unit weight, internal shear strength parameters, pore water pressure, and slope geometry to determine the location of the most critical failure plane. The soil and rock strength characteristics are summarized in Table 1.

Table 1. Soil and Rock Input Parameters for Slope Stability Analysis

Soil Type	Moist Unit Weight (pcf)	Cohesion (psf)	Drained Friction Angle (degrees)
SM, CL-CH Soil (silty SAND with gravel and sandy fat to sandy lean CLAY)	115	50	27
Bedrock (basaltic andesite)	150	1000	35

Groundwater

Groundwater was encountered in test pit TP-5 at approximately 5 feet below ground surface and seepage was present in test pit TP-6 at approximately 9 feet below ground surface. Groundwater flow in volcanic intrusions such as the Green Mountain cinder cone is extremely complex. To incorporate the potential effects of perched groundwater, saturated soil conditions, and reduced shear strength into the slope stability analysis, a piezometric surface was estimated to model a fully saturated condition.

Seismic Considerations

Seismic events can induce horizontal ground acceleration significantly in excess of static conditions, and should be modeled to predict slope stability. A pseudo-static analysis represents the potential effects of a seismic event by using a horizontal acceleration that effectively increases inertial inter-slice forces during computation. Both static and pseudo-static analyses were performed for slope cross-sections. The horizontal acceleration used in the analysis is approximately one-half times the peak ground acceleration (PGA) for an anticipated earthquake at the subject site with a 2-percent probability of being exceeded within 50 years (i.e. a 2,475-year return seismic event).

Vegetation

The presence of vegetation was not modeled in the analyses. Deep-rooted tree species, small bushes, grass, and other ground cover present on the slopes provide energy absorption for falling precipitation and soil-binding forces that fasten and secure near-surface soil layers together. This slightly increases the soil's strength and ability to withstand increased shear stress, and may also increase slope stability, particularly with respect to shallow surficial slumps.

5.1.6 SLOPE/W Results

A variety of scenarios and input parameters were analyzed to determine critical slip surfaces and corresponding factors of safety against slope failure. The factor of safety can generally be interpreted to indicate the ratio between the slope's stabilizing forces (the forces holding the slope in place) and the slope's mobilizing forces (the forces causing failure). A ratio, or factor of safety, of 1.0 indicates equilibrium. Because soil is rarely isotropic and homogenous, a factor of safety of at least 1.5 is generally recommended for slope stability under static conditions and a factor of safety of at least 1.1 is generally recommended under pseudo-static conditions.

Based upon results of the analyses as indicated in Appendix D, slope stability factors of safety for analyzed slopes exceeded the minimum recommended values of 1.5 for static conditions or 1.1 for pseudo-static conditions. Slope models in the cinder cone slopes (approximately 30 to 40 percent grade) of the A-A' and B-B' cross-sections indicate only minor surficial soil failures with high factors of safety in the existing condition for both static and pseudo-static models. Slope analysis in the slopes between the central bench and valley floor (typically greater than 40 percent) showed that critical failure surface entry points with factors of safety greater than 1.5 could be located behind the top of slope locations. This indicates the need for maintaining a horizontal setback distance for future structures and other loads in this area.

As discussed below, recommended horizontal setback distances from the top of slope in the area between the central bench and valley floor provide mitigation measures to limit structural loads within proximity to anticipated critical entry surfaces for static and pseudo-static factors of safety.

5.1.7 Geotechnical Mitigations for the Geologic Hazard Area

To reduce the risk of adverse impacts to slope stability within the geologic hazard area, residential structures and structural fill placement should maintain a horizontal setback distance of at least 35 feet from the top of slope break in the area between the central bench and valley floor. Generalized geotechnical setback lines are indicated on Figure 2. The top of slope and setback area should be identified on final approved construction documents. Additionally, significant changes to slope geometry are not recommended within the cinder cone slope area and beneath the slopes between the central bench and the valley floor. Specifically, significant removal of material near the toe of slopes should not occur. Minor grading is anticipated and may require additional analysis. Additional discussion is provided in Section 5.1.10, *Additional Analysis of Future Grading Plan.*

Some encroachment of residential foundations proposed within the zones described above may be feasible if evaluated on a case-by-case basis by the geotechnical engineer. The setback recommendations described above are intended to reduce potential for slope instability by restricting locations for large dynamic and static loads and associated impacts derived from earthwork, residential structures, retaining walls, roadways, stormwater facilities, and other significant developments.

5.1.8 Grading Recommendations within the Geotechnical Setback Zone

The slope setback area is intended to mitigate impacts to slope stability due to dynamic and static loading. Placement of engineered structural fill or stockpiles of disturbed soil is prohibited within the slope setback buffer zone. Soil excavation may be acceptable within this zone, as driving forces may be reduced by removing soil mass. Columbia West should review mass grading plans and construction techniques within the geotechnical setback zone prior to commencement of site construction. The geotechnical setback zone is not intended to be a do-not-disturb conservation area. Small disturbances such as minor landscaping, or fence building are acceptable.

Deep-rooted vegetation generally results in reduced slope erosion and increased near-surface soil shear strength. The risk of slope instability increases with disturbance or alteration of existing slope vegetation. Removal of established slope vegetation should be minimized. The text herein pertains only to the geotechnical aspect of construction within the recommended geotechnical setback zones.

5.1.9 Potential Encroachment within the Geologic Hazard Area

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. Feasibility of such encroachment will depend upon dimensions, locations, and specific design features of the proposed improvement. Often these data are not available until later in the design process. Encroachment within the geologic slope hazard area should be contingent upon a supplemental geotechnical investigation. The investigation should include additional exploratory activities and data analysis to develop appropriate design recommendations. Quantification of risk of slope instability and specialized design recommendations, if applicable or necessary, should be included.

5.1.10 Additional Analysis of Future Grading Plan

Future grading is anticipated to alter slope geometry in areas that require infrastructure elements such as road prism construction and stormwater collection and disposal features. While significant mass grading is discouraged in sloped areas, minor cuts and fills are anticipated on the cinder cone slopes and at the valley floor beneath sloped areas of the central bench that may affect adjacent slope geometry. Stormwater drainage may also affect slope stability due to the potential changes to current water flow at the site. Additional slope analysis may be necessary based upon the future grading plan. Final grading contours and infrastructure elements should be reviewed by Columbia West prior to construction.

5.1.11 Slope Stability Limitations and Risk

Columbia West's slope stability analysis as described in this report indicates some inherent risk associated with slope instability due to proposed residential development in proximity to steep slopes on portions of the site. This is typical for development near any sloped areas. Reduction of slope instability risk may be partially obtained by implementing horizontal building setback distances and applying proper site planning and engineering principles as described in this report.

The slope analysis described in this report does not take into consideration the effects that mass grading, stormwater disposal, or significant infrastructure improvements may have on global or local slope stability. Significant cuts and fills proposed should be analyzed by Columbia West if proposed within the geohazard area.

Due to multiple unknowns inherent in slope stability analysis it is often difficult or impossible to definitively predict stability. This slope stability 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 slope stability analysis may not be valid if building locations or other site plans are altered. Columbia West should review proposed drainage, building, and grading plans prior to final approval.

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.

Soil Liquefaction or Dynamic Settlement

According to the *Liquefaction Susceptibility Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), the site is mapped as bedrock with 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 presence of relatively shallow bedrock, 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.

Ground Shaking Amplification

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 B to C as defined in *2015 IBC Table 1613.3.2*. A designation of Site Class B to C indicates that minor amplification of seismic energy may occur during a seismic event due to subsurface conditions. 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], 2017 Website), and field observations, the erosion hazard for site soils ranges from slight to severe. The severe erosion hazard areas are associated with steeper sloped areas. 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 slopes identified in Section 5.0, the presence of shallow bedrock, and potential groundwater seepage.

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 average anticipated stripping depth for highly organic topsoil is anticipated to be 12 inches, based on the locations observed. 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.

Due to the presence of previously mentioned infrastructure elements and development on site, disturbed soil and undocumented fill may be present at the site. 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. Previously backfilled sanitary sewer pipe trenches may not contain structural fill that may need to be reconditioned so that they may 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. For fine-textured onsite native soils reused as structural fill, a field density at least equal to 95 percent of the maximum dry density, obtained from the standard Proctor moisture-density relationship test (ASTM D698), is recommended for structural fill placement in roadways and lot fills. For imported granular materials proposed for use as structural fill, 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 and 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. Most clean native soils may be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native fine-textured soils may require addition of moisture during late summer months or after extended periods of warm dry weather. Compacted fine-textured fill soils should be covered shortly after placement to maintain moisture conditions and minimize shrink swell potential. Native clay soils with a plasticity index greater than 25 should be evaluated and approved by Columbia West prior to re-use as structural fill. 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 fill consisting of well-graded granular material with a maximum particle size of three inches and no more than five percent passing the No. 200 sieve is recommended for structural fill.

Test pits were excavated during the subsurface exploration of the site and backfilled loosely with onsite soils on February 14, 2017. Test pits should be located and properly backfilled during site construction.

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 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 individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 4.

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

Based upon correspondence with the design team, residential foundations are anticipated to consist of shallow continuous perimeter or column spread footings. Footings should be designed by a licensed structural engineer and conform to the minimum requirements of the International Residential Code and the recommendations below. Typical building loads are not expected to exceed approximately 2 to 3 kips per foot for perimeter footings or 10 kips per column. If actual loading exceeds anticipated loading, additional analysis should be conducted for the specific load conditions and proposed footing dimensions. The existing ground surface should be prepared as described in Section 6.1, *Site Preparation and Grading*, and Section 6.2, *Engineered Structural Fill*. Foundations should bear upon firm native soil or engineered structural fill.

To evaluate bearing capacity for proposed structures, serviceability and reliability of shear resistance for subsurface soils was considered. Allowable bearing capacity is typically a function of footing dimension and subsurface soil properties, including settlement and shear resistance. Based upon in situ field testing and laboratory analysis, the estimated allowable bearing capacity for well-drained foundations prepared as described above is 1,500 psf. Bearing capacity may be increased by one-third for transient lateral forces such as seismic or wind. The estimated coefficient of friction between in situ compacted native soil or engineered structural fill and in-place poured concrete is 0.45. Lateral forces may also be resisted by an assumed passive soil equivalent fluid pressure of 250 psf/f against embedded footings. The upper six inches of soil should be neglected in passive pressure calculations.

Footings should extend to a depth at least 18 inches below lowest adjacent grade to provide adequate bearing capacity and protection against frost heave. Foundations constructed during wet weather conditions will require over-excavation of saturated subgrade soils and granular structural backfill prior to concrete placement. Over-excavation recommendations should be provided by Columbia West during foundation excavation and construction. Excavations adjacent to foundations should not extend within a 1.5H:1V angle projected down from the outside bottom footing edge without additional geotechnical analysis.

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

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 55 feet using a mud-rotary drill rig and a maximum depth of 15 feet using a track-mounted excavator. As mentioned previously, weathered and competent bedrock was encountered underlying fine-textured and gravelly soils. Groundwater seeps were also observed in a few locations and are anticipated to increase in frequency during periods of wet weather.

Due to the presence of subsurface and outcropped competent basaltic-andesite bedrock, drilling and blasting or specialized rock excavation techniques may be required. Difficult excavation and refusal of the excavator was occasionally encountered at shallow depths. Bedrock conditions range from massive or interlocking jointed crystalline bedrock to highly weathered, loosely jointed bedrock with soil-like excavation characteristics.

Based upon review of available seismic refraction literature, the estimated compression wave velocity for refusal by a standard 45,000-lb excavator is approximately 5,000 ft/sec. The *NAVFAC Manual 7.02* indicates that bedrock with compression wave velocities up to 7,500 ft/sec may be ripped with a single-shank heavy-duty bulldozer. Bedrock exceeding 8,000 ft/sec typically requires drilling and blasting.

Bedrock at the site may be suitable for crushing and use as structural fill or aggregate base. Specific soundness or durability tests have not been conducted at this time. However, based upon observation of excavation techniques and hand-held hammer energy required to cause fracture, much of the rock may meet typical specifications for construction crushed aggregate.

If significant excavation depths are proposed, Columbia West recommends that a blasting contractor review information presented in this report and design a drill pattern. A pre-blast survey of adjacent properties and review of City of Camas specifications for explosive blasting should also be conducted. It should be noted that excavation of fractured material may be difficult even after blasting. Blasting effects to existing BPA power transmission and CPU water well infrastructure in portions of the site may also need to be assessed, if blasting is required.

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, and in no case should excavation be conducted in excess of all applicable local, state, and federal laws.

6.8 Lateral Earth Pressure

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

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

Table 2. Lateral Earth Pressure Parameters for Level Backfill

Backfill Material	Equivalent Fluid Pressure for Level Backfill			Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive		
Undisturbed native silty SAND with gravel and sandy fat CLAY to sandy lean CLAY (Soil Type 1)	63 pcf	43 pcf	306 pcf	115 pcf	27°
Approved Structural Backfill Material	54 pcf	33 pcf	589 pcf	135 pcf	38°
WSDOT 9-03.12(2) compacted aggregate backfill					

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

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

A continuous one-foot-thick zone of free-draining, washed, open-graded 1-inch by 2-inch drain rock and a 4-inch perforated gravity drain pipe is assumed behind retaining walls. Geotextile filter fabric should be placed between the drain rock and backfill soil. Specifications for drainpipe design are presented in Section 6.11, *Drainage*. If walls cannot be gravity drained, saturated base conditions and/or applicable hydrostatic pressures should be assumed.

Final retaining wall design should be reviewed 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) *Seismic Design Maps Detailed Report* based on 2010 ASCE 7 (w/ March 2013 errata), the anticipated peak ground and maximum considered earthquake spectral response accelerations resulting from seismic activity for the subject site are summarized in Table 3.

The listed probabilistic ground motion values are based upon “very dense soil and soft 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.

The *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), indicates site soils may be represented by Site Class B to 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 B to 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.

Table 3. Approximate Probabilistic Ground Motion Values for 'very dense soil and soft 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	0.38 g

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

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

6.10 Dewatering

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

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

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.

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

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

Foundation drains and subdrains should be closely monitored after construction to assess their effectiveness. If additional surface or shallow subsurface seeps become evident, the drainage provisions may require modification or additional drains. Columbia West should be consulted to provide appropriate recommendations.

6.12 Bituminous Asphalt and Portland Cement Concrete

The site is anticipated to include asphalt concrete residential streets. Columbia West recommends adherence to City of Camas standards for pavement thickness sections for proposed roads and frontage street improvements.

For dry weather construction, 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 91 percent of maximum Rice density. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with Washington Department of Transportation and City of Camas specifications.

Portland cement concrete curbs should be installed in accordance with the City of Camas specifications. Aggregate base should be observed and proof-rolled in the presence of an experienced geotechnical engineer or designated representative. Soft areas that deflect or rut should be stabilized prior to pouring concrete. Concrete should be tested during installation in accordance with ASTM C171, C138, C231, C143, C1064, and C31. This includes casting of cylinder specimen at a frequency of four cylinders per 100 cubic yards of poured concrete. Recommended field and analytical laboratory concrete testing includes slump, air entrainment, temperature, and unit weight.

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

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

Crushed aggregate base should be compacted to at least 95 percent of maximum dry density according to the modified Proctor density test (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 80 percent by weight passing the No. 200 sieve and exhibit a plasticity index ranging from 11 to 40 percent. This indicates some potential for soil shrinking or swelling. To minimize potential risks, foundation embedment depth may be increased. An experienced Geotechnical Engineer or designated representative 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. Additional considerations for subsurface excavation are discussed in Section 6.7, *Excavation.*

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

Sincerely,

COLUMBIA WEST ENGINEERING, Inc.

Daniel E. Lehto, PE, GE
Principal

DANIEL E. LEHTO
44702

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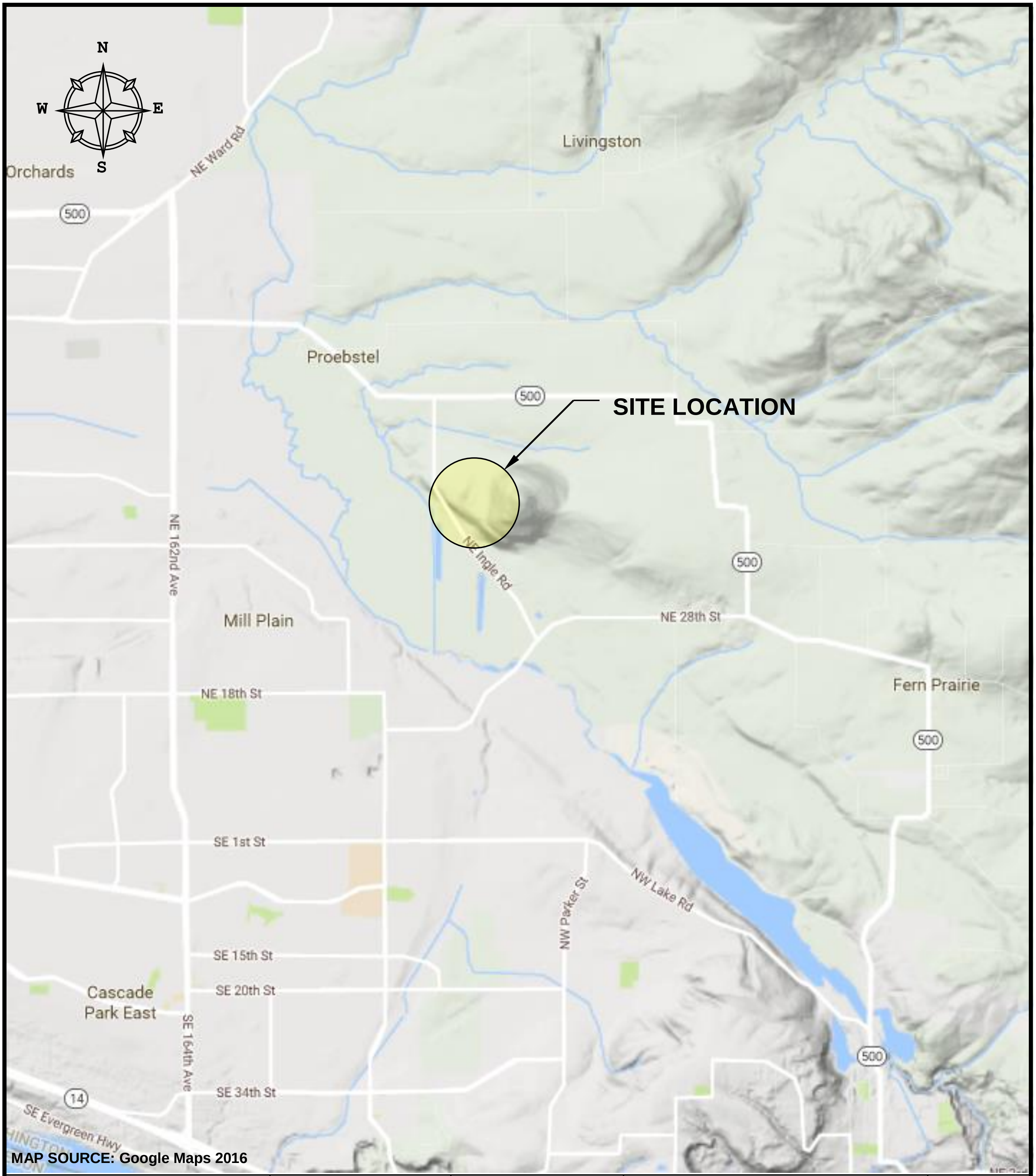
FIGURES



MAP SOURCE: Google Maps 2016

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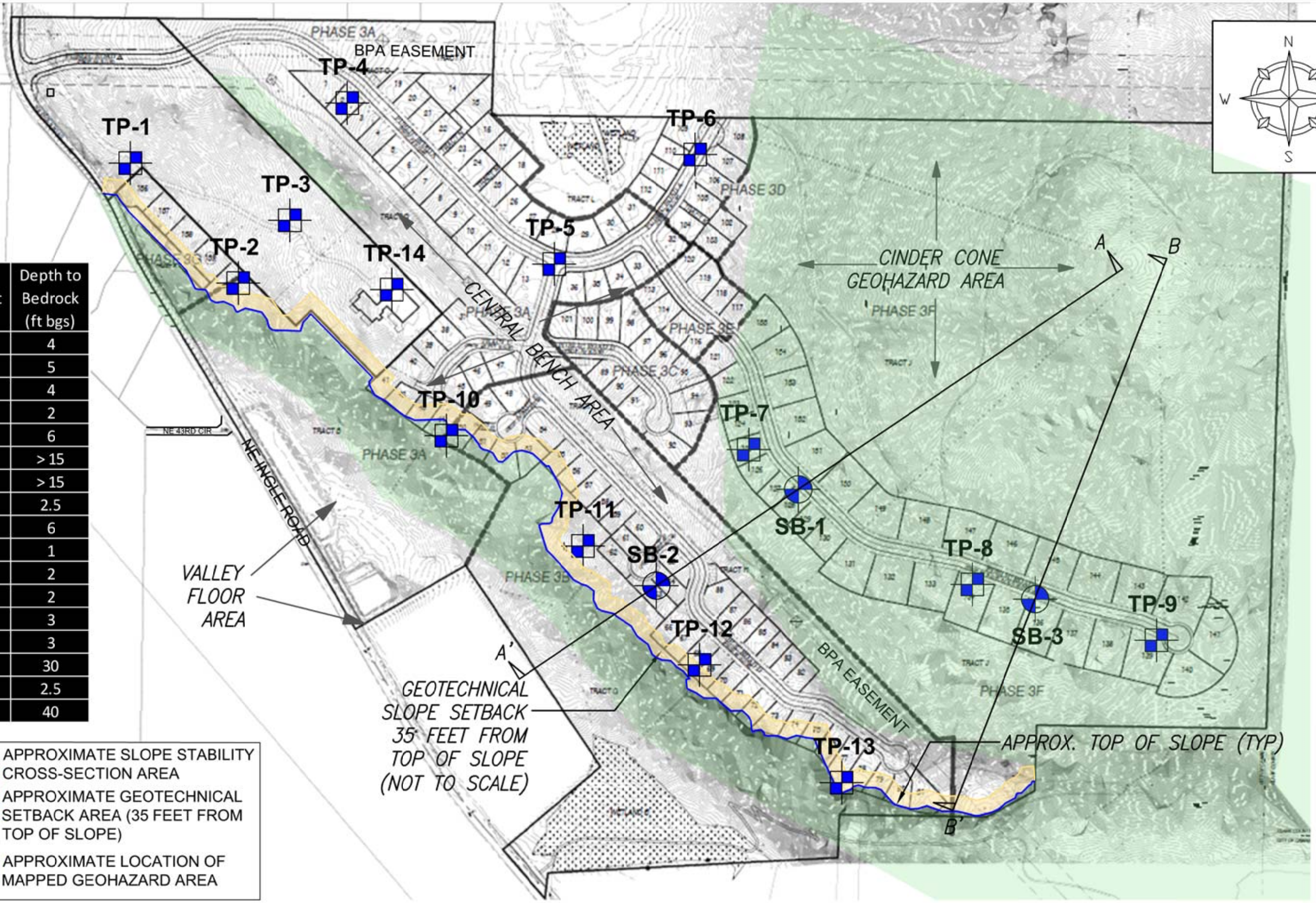
Design	Drawn: CWS		
Checked: ASR	Date: 03/22/17		
Client: GREEN MOUNTAIN	Rev	By	Date
Job No.: 17012			
CAD File: FIGURE 1			
Scale: NTS			

SITE LOCATION MAP

GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON

FIGURE
1



Test Pit	Depth to Bedrock (ft bgs)
TP-1	4
TP-2	5
TP-3	4
TP-4	2
TP-5	6
TP-6	> 15
TP-7	> 15
TP-8	2.5
TP-9	6
TP-10	1
TP-11	2
TP-12	2
TP-13	3
TP-14	3
SB-1	30
SB-2	2.5
SB-3	40

🟥	APPROXIMATE SLOPE STABILITY CROSS-SECTION AREA
🟨	APPROXIMATE GEOTECHNICAL SETBACK AREA (35 FEET FROM TOP OF SLOPE)
🟩	APPROXIMATE LOCATION OF MAPPED GEOHAZARD AREA

NOTES:
1. SITE LOCATION: EAST OF NE INGLE ROAD AND NE 43RD CIRCLE, CAMAS, WASHINGTON.
2. SITE CONSISTS OF PARCELS 171704000, 171270000, AND 172341000 TOTALING APPROXIMATELY 115 ACRES.
3. DRAWING IS NOT TO SCALE.
4. BASE MAP PROVIDED BY OLSON ENGINEERING.
5. SOIL EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
6. TEST PITS BACKFILLED LOOSELY WITH ONSITE SOILS ON FEBRUARY 14, 2017.

⊞ TEST PIT EXCAVATION

⊕ SOIL BORING EXCAVATION

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Client: GREEN MOUNTAIN LAND
Job No: 17012
CAD File: FIGURE 2
Scale: NONE

Drawn: CWS
Date: 09/27/17

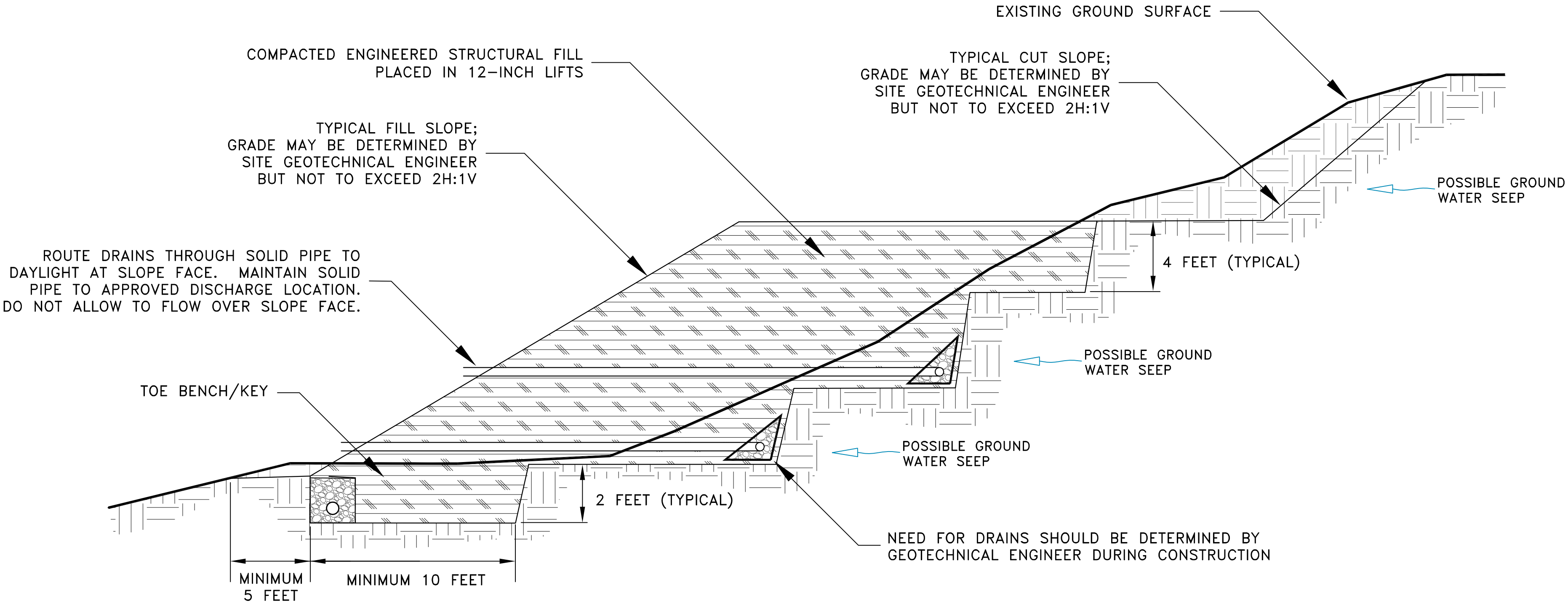
Rev	By	Date

EXPLORATION LOCATION MAP

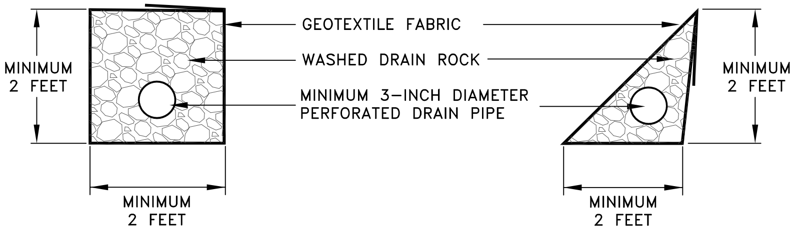
GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON

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:
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Client: GREEN MOUNTAIN LAND
Job No: 17012
CAD File: FIGURE 3
Scale: NONE

Drawn: CWS
Date: 02/21/17

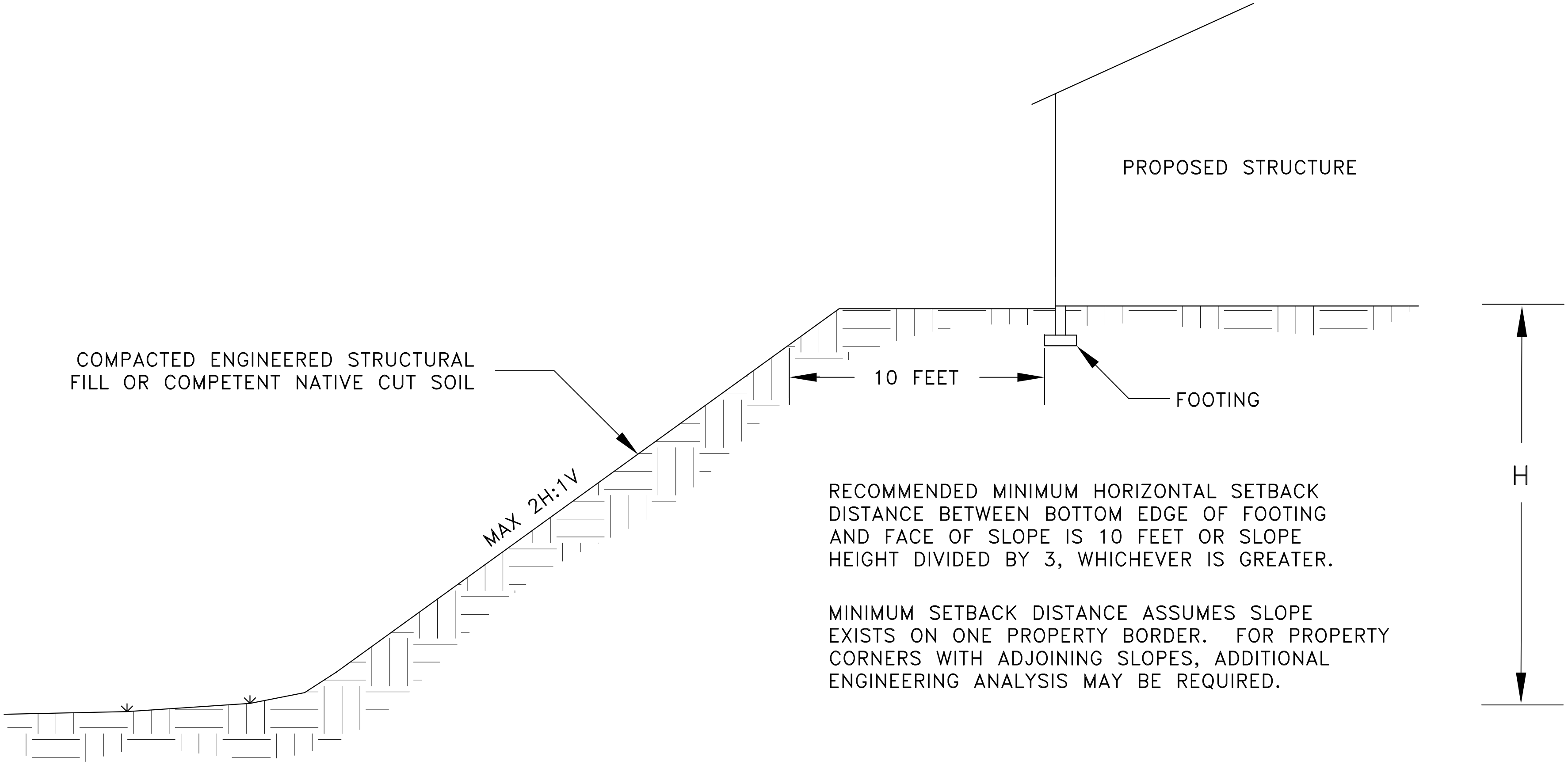
Rev	By	Date

TYPICAL CUT AND FILL SLOPE CROSS-SECTION

GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON

FIGURE
3

MINIMUM FOUNDATION SLOPE SETBACK DETAIL



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

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

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

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Client: GREEN MOUNTAIN LAND
Job No: 17012
CAD File: FIGURE 4
Scale: NONE
Drawn: CWS
Date: 02/21/17

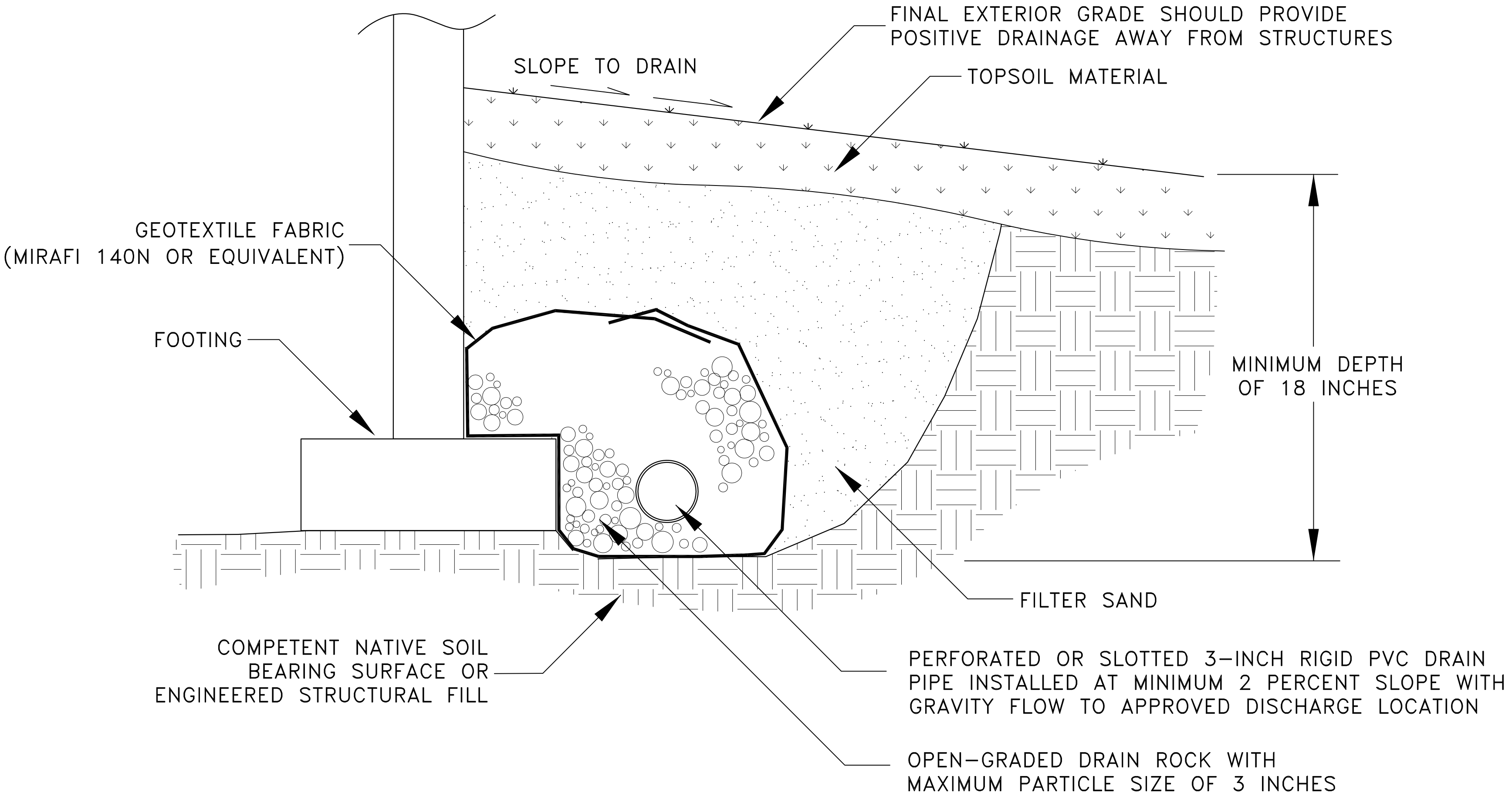
Rev	By	Date

TYPICAL MINIMUM SLOPE SETBACK DETAIL

GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON

FIGURE 4

TYPICAL PERIMETER FOOTING DRAIN DETAIL



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

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Client: GREEN MOUNTAIN LAND
Job No: 17012
CAD File: FIGURE 5
Scale: NONE

Drawn: CWS
Date: 02/21/17

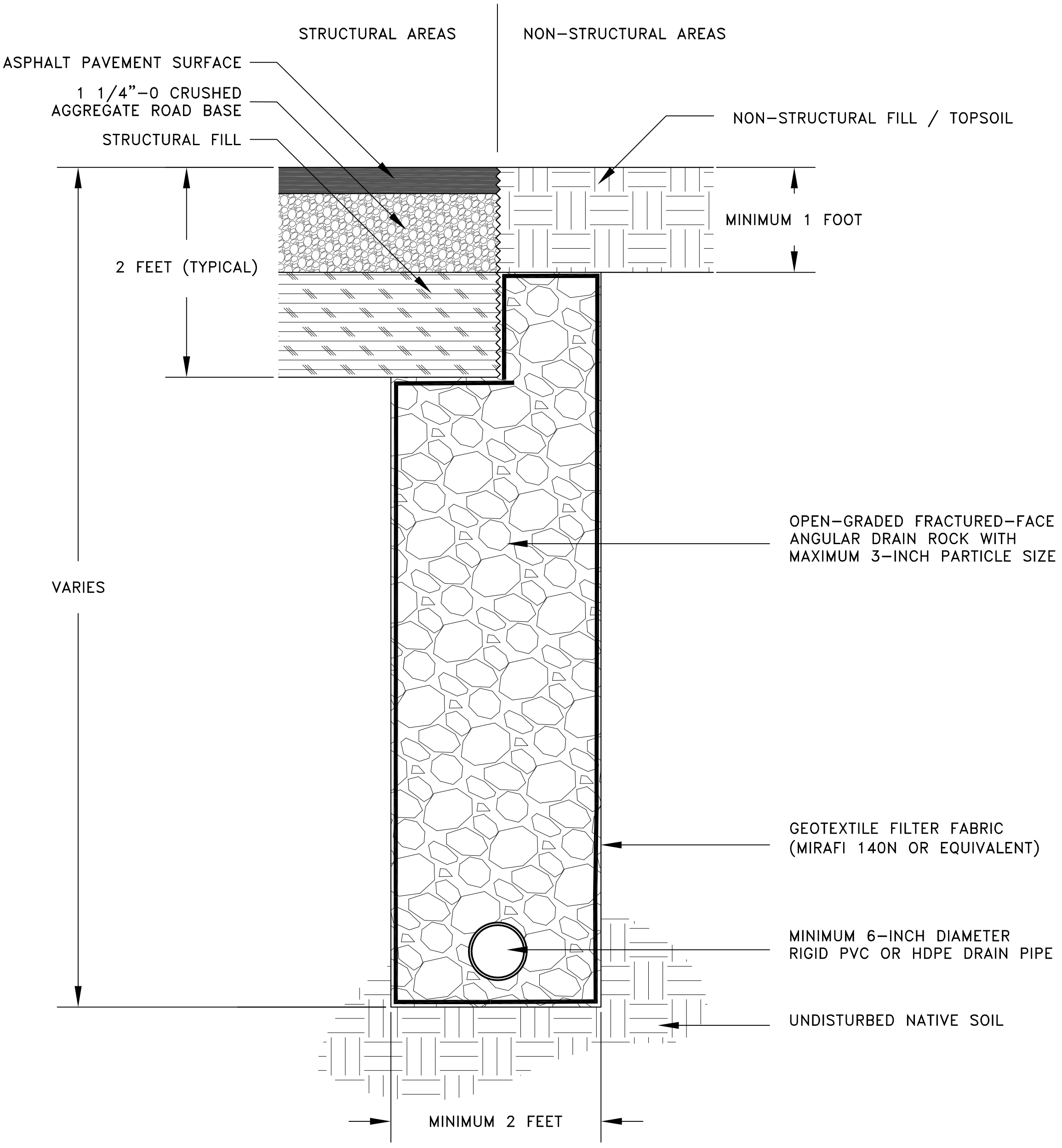
Rev	By	Date

TYPICAL PERIMETER FOOTING DRAIN DETAIL

GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON

FIGURE
5

TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL



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

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Checked: ASR	Date: 02/21/17		
Client: GREEN MOUNTAIN LAND	Rev	By	Date
Job No: 17012			
CAD File: FIGURE 6			
Scale: NONE			

TYPICAL PERFORATED
DRAIN PIPE TRENCH DETAIL

GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON

FIGURE
6

APPENDIX A
LABORATORY TEST RESULTS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT
Green Mountain North
Camas, Washington

CLIENT
Mr. John Schmidt
Green Mountain Land, LLC
17933 NW Evergreen Parkway, Suite 300
Beaverton, Oregon 97006

PROJECT NO. 17012
LAB ID S17-097
REPORT DATE 03/09/17
FIELD ID TP1.1
DATE SAMPLED 02/14/17
SAMPLED BY ASR

MATERIAL DATA

MATERIAL SAMPLED
Silty SAND with Gravel

MATERIAL SOURCE
Test Pit TP-01
depth = 3 feet

USCS SOIL TYPE
SM, Silty Sand with Gravel

SPECIFICATIONS
none

AASHTO SOIL TYPE
A-6(2)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter 637

TEST PROCEDURE
ASTM D6913, D422

ADDITIONAL DATA

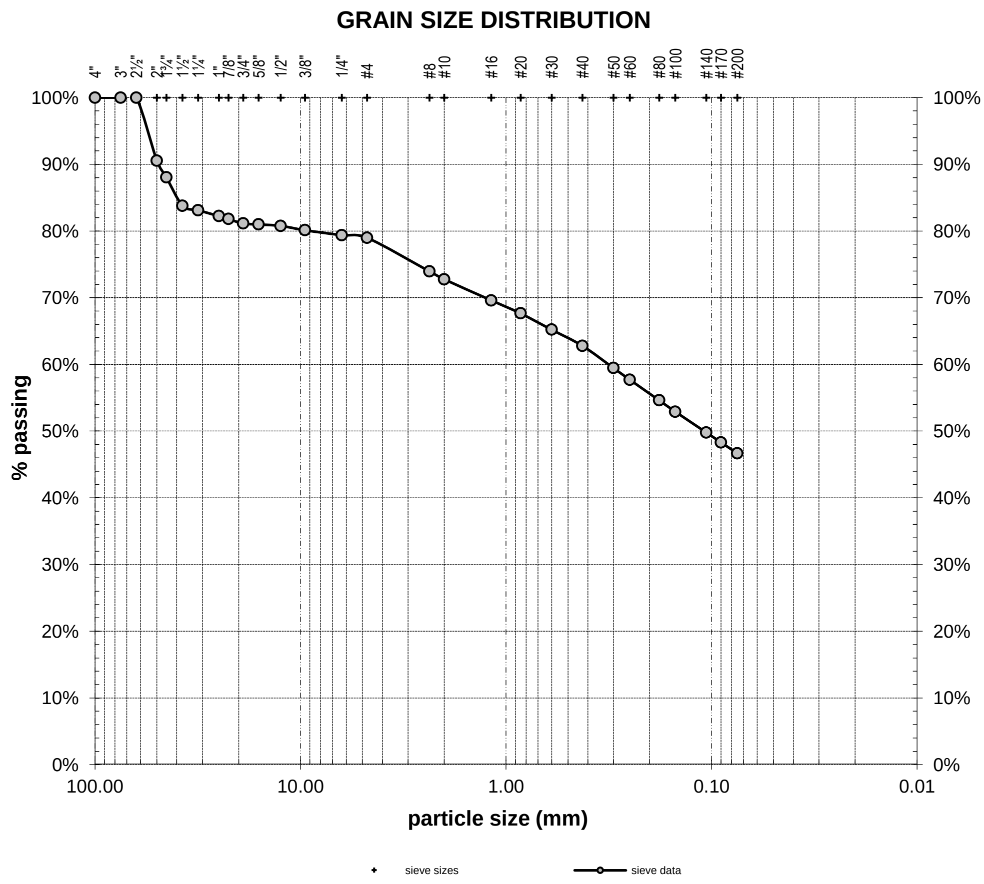
initial dry mass (g) = 1510.3
as-received moisture content = 32.6%
liquid limit = 37
plastic limit = 26
plasticity index = 11
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = 0.318 mm

SIEVE DATA

% gravel = 21.0%
% sand = 32.3%
% silt and clay = 46.7%

	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	90.5%			
	1.75"	45.0		88.1%		
	1.50"	37.5	83.8%			
	1.25"	31.5		83.1%		
	1.00"	25.0	82.2%			
	7/8"	22.4		81.8%		
	3/4"	19.0	81.1%			
	5/8"	16.0		81.0%		
	1/2"	12.5	80.8%			
	3/8"	9.50	80.1%			
	1/4"	6.30	79.4%			
	#4	4.75	79.0%			
SAND	#8	2.36		73.9%		
	#10	2.00	72.8%			
	#16	1.18		69.6%		
	#20	0.850	67.6%			
	#30	0.600		65.2%		
	#40	0.425	62.8%			
	#50	0.300		59.4%		
	#60	0.250	57.7%			
	#80	0.180		54.6%		
	#100	0.150	52.9%			
	#140	0.106		49.8%		
	#170	0.090		48.3%		
	#200	0.075	46.7%			



DATE TESTED 03/07/17
TESTED BY RTT/MJR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain North Camas, Washington	Mr. John Schmidt Green Mountain Land, LLC 17933 NW Evergreen Parkway, Suite 300 Beaverton, Oregon 97006	17012	S17-097
		REPORT DATE 03/09/17	FIELD ID TP1.1
		DATE SAMPLED 02/14/17	SAMPLED BY ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty SAND with Gravel	Test Pit TP-01 depth = 3 feet	SM, Silty Sand with Gravel

LABORATORY TEST DATA

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

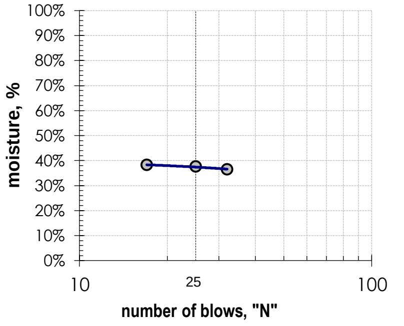
ATTERBERG LIMITS

liquid limit = 37
plastic limit = 26
plasticity index = 11

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	37.84	41.76	39.75	
dry soil + pan weight, g =	33.29	36.05	34.53	
pan weight, g =	20.81	20.88	20.87	
N (blows) =	32	25	17	
moisture, % =	36.5 %	37.6 %	38.2 %	

LIQUID LIMIT



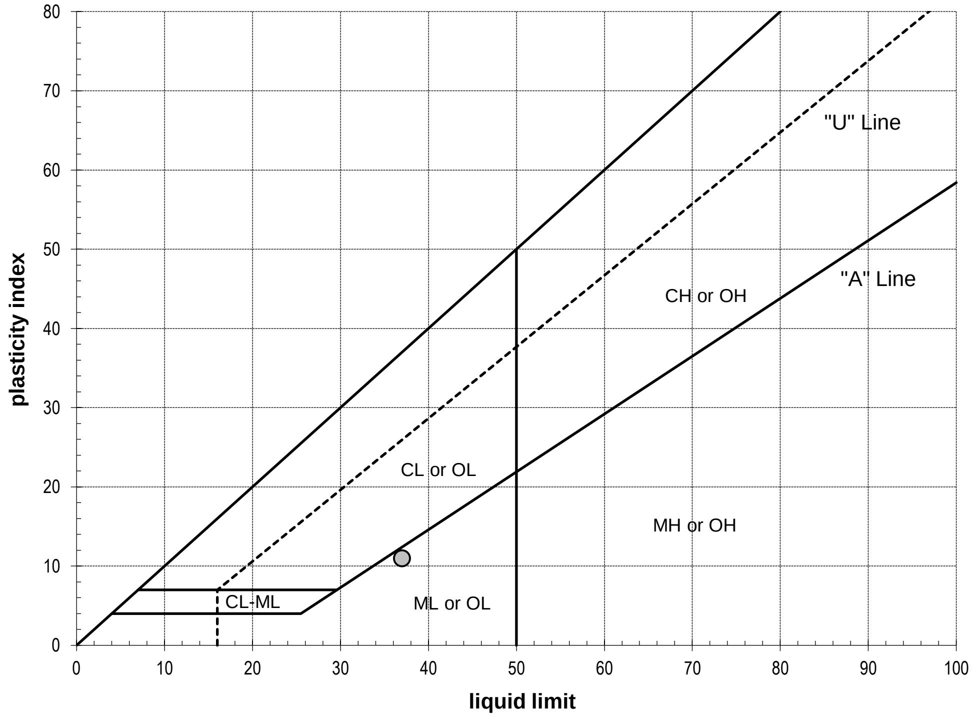
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.92	28.35		
dry soil + pan weight, g =	27.24	26.81		
pan weight, g =	20.81	20.85		
moisture, % =	26.1 %	25.8 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 21.0%
% sand = 32.3%
% silt and clay = 46.7%
% silt = n/a
% clay = n/a
moisture content = 32.6%

DATE TESTED	TESTED BY
03/08/17	RTT/MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT Green Mountain North Camas, Washington	CLIENT Mr. John Schmidt Green Mountain Land, LLC 17933 NW Evergreen Parkway, Suite 300 Beaverton, Oregon 97006	PROJECT NO. 17012 REPORT DATE 03/09/17 DATE SAMPLED 02/14/17	LAB ID S17-098 FIELD ID TP6.2 SAMPLED BY ASR

MATERIAL DATA

MATERIAL SAMPLED Sandy Fat CLAY	MATERIAL SOURCE Test Pit TP-06 depth = 10 feet	USCS SOIL TYPE CH, Sandy Fat Clay
SPECIFICATIONS none		AASHTO SOIL TYPE A-7-6(16)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter 637

TEST PROCEDURE
ASTM D6913, D422

ADDITIONAL DATA

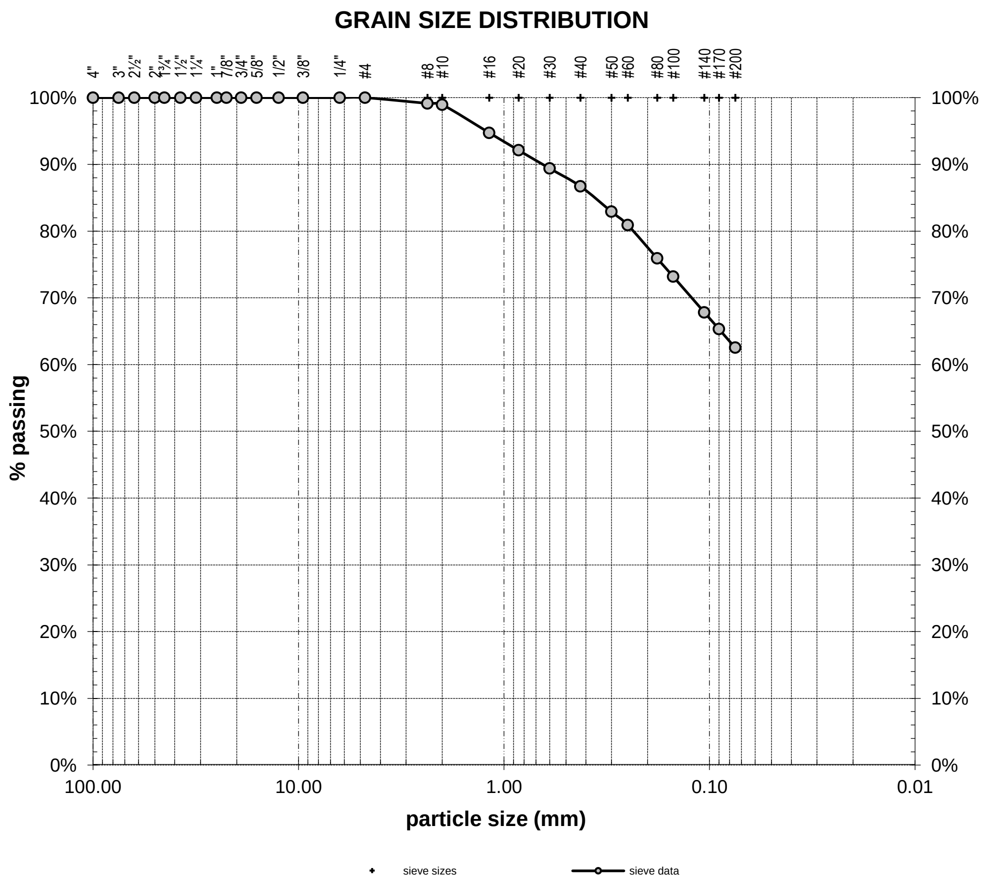
initial dry mass (g) = 185.7
as-received moisture content = 29.2%
liquid limit = 53
plastic limit = 24
plasticity index = 29
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = n/a

SIEVE DATA

% gravel = 0.0%
% sand = 37.5%
% silt and clay = 62.5%

	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		99.1%		
	#10	2.00	98.9%			
	#16	1.18		94.7%		
	#20	0.850	92.1%			
	#30	0.600		89.4%		
	#40	0.425	86.7%			
	#50	0.300		82.9%		
	#60	0.250	80.9%			
	#80	0.180		75.9%		
	#100	0.150	73.2%			
	#140	0.106		67.8%		
	#170	0.090		65.3%		
	#200	0.075	62.5%			



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain North Camas, Washington	Mr. John Schmidt Green Mountain Land, LLC 17933 NW Evergreen Parkway, Suite 300 Beaverton, Oregon 97006	17012	S17-098
		REPORT DATE	FIELD ID
		03/09/17	TP6.2
		DATE SAMPLED	SAMPLED BY
		02/14/17	ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Fat CLAY	Test Pit TP-06 depth = 10 feet	CH, Sandy Fat Clay

LABORATORY TEST DATA

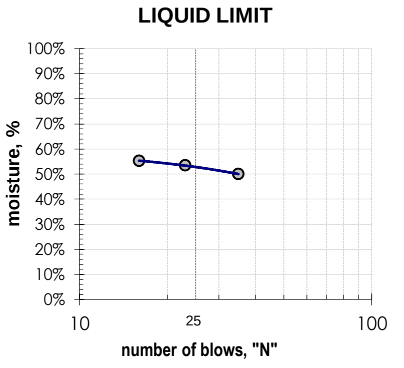
LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

ATTERBERG LIMITS

liquid limit = 53
plastic limit = 24
plasticity index = 29

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	38.24	36.93	36.53	
dry soil + pan weight, g =	32.45	31.29	30.89	
pan weight, g =	20.86	20.73	20.69	
N (blows) =	35	23	16	
moisture, % =	50.0 %	53.4 %	55.3 %	



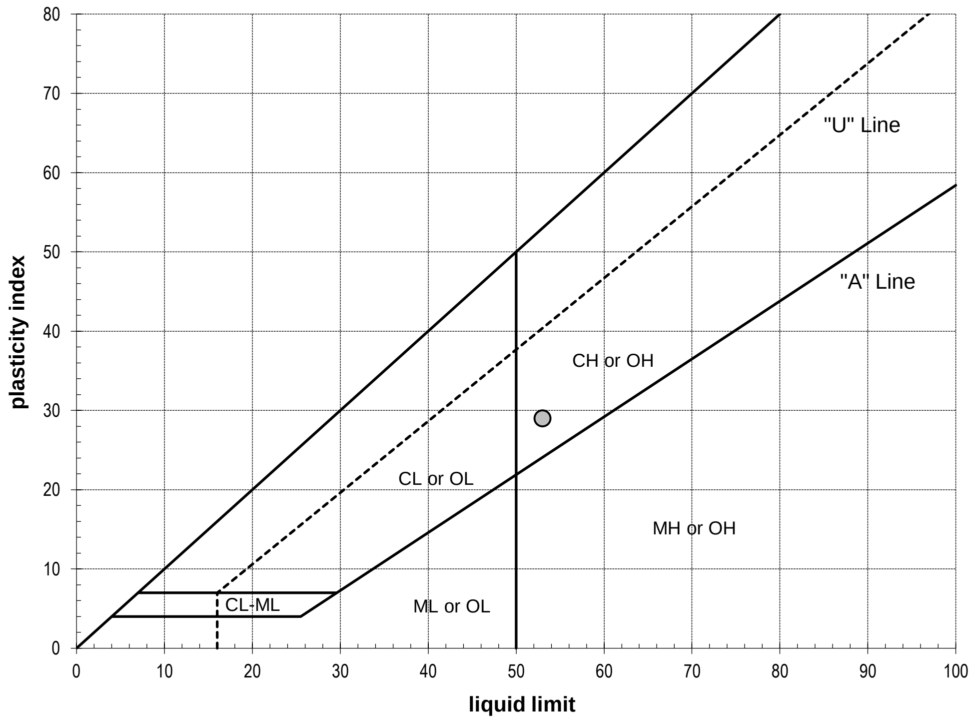
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.16	28.02		
dry soil + pan weight, g =	26.73	26.60		
pan weight, g =	20.73	20.59		
moisture, % =	23.8 %	23.6 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	37.5%
% silt and clay =	62.5%
% silt =	n/a
% clay =	n/a
moisture content =	29.2%

DATE TESTED	TESTED BY
03/07/17	RTT/MJR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain North Camas, Washington	Mr. John Schmidt Green Mountain Land, LLC 17933 NW Evergreen Parkway, Suite 300 Beaverton, Oregon 97006	17012	S17-099
		REPORT DATE	FIELD ID
		03/09/17	TP9.1
		DATE SAMPLED	SAMPLED BY
		02/14/17	ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY with Sand	Test Pit TP-09 depth = 4 feet	CH, Fat Clay with Sand

LABORATORY TEST DATA

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

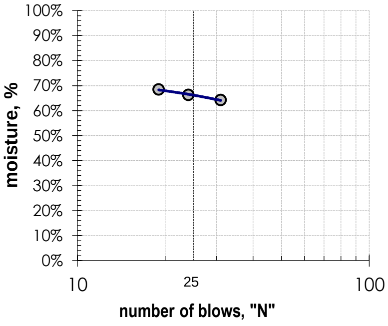
ATTERBERG LIMITS

liquid limit = 66
plastic limit = 26
plasticity index = 40

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.70	32.53	34.74	
dry soil + pan weight, g =	28.08	27.81	29.02	
pan weight, g =	20.88	20.69	20.66	
N (blows) =	31	24	19	
moisture, % =	64.2 %	66.3 %	68.4 %	

LIQUID LIMIT



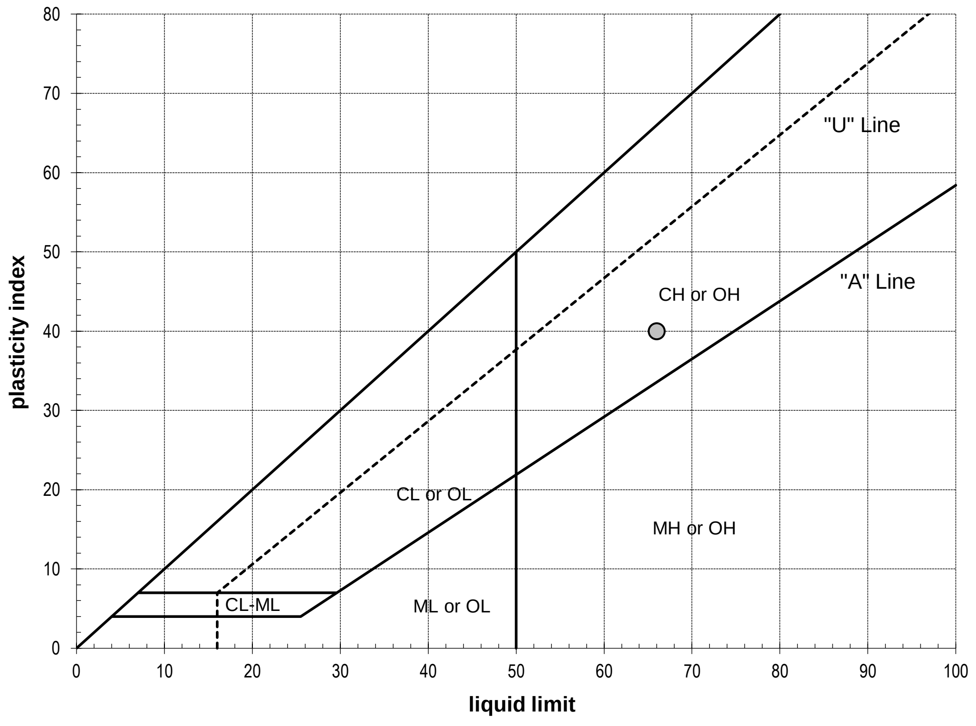
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.01	27.91		
dry soil + pan weight, g =	26.50	26.39		
pan weight, g =	20.79	20.60		
moisture, % =	26.4 %	26.3 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 19.6%
% silt and clay = 80.4%
% silt = n/a
% clay = n/a
moisture content = 41.4%

DATE TESTED	TESTED BY
03/07/17	RTT/MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT
Green Mountain North
Camas, Washington

CLIENT
Mr. John Schmidt
Green Mountain Land, LLC
17933 NW Evergreen Parkway, Suite 300
Beaverton, Oregon 97006

PROJECT NO. 17012
LAB ID S17-099
REPORT DATE 03/09/17
FIELD ID TP9.1
DATE SAMPLED 02/14/17
SAMPLED BY ASR

MATERIAL DATA

MATERIAL SAMPLED
Fat CLAY with Sand

MATERIAL SOURCE
Test Pit TP-09
depth = 4 feet

USCS SOIL TYPE
CH, Fat Clay with Sand

SPECIFICATIONS
none

AASHTO SOIL TYPE
A-7-6(35)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter 637

TEST PROCEDURE
ASTM D6913, D422

ADDITIONAL DATA

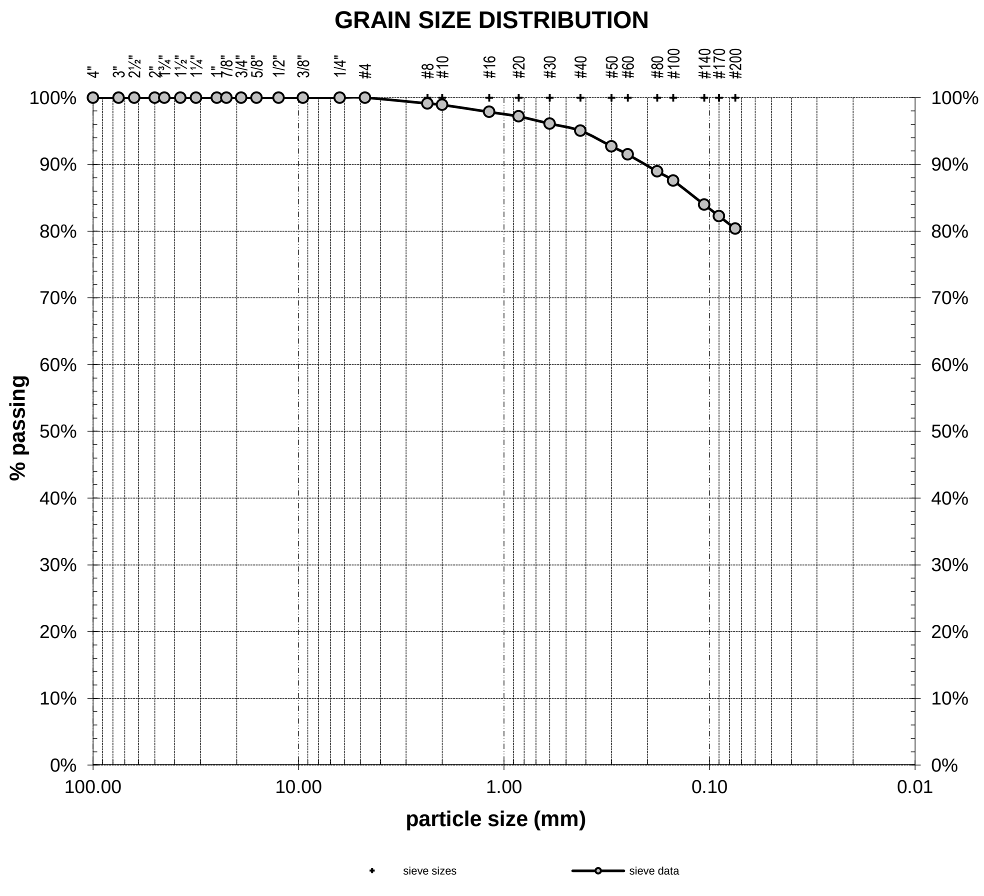
initial dry mass (g) = 139.7
as-received moisture content = 41.4%
liquid limit = 66
plastic limit = 26
plasticity index = 40
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = n/a

SIEVE DATA

% gravel = 0.0%
% sand = 19.6%
% silt and clay = 80.4%

	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		99.1%		
	#10	2.00	98.9%			
	#16	1.18		97.8%		
	#20	0.850	97.2%			
	#30	0.600		96.1%		
	#40	0.425	95.0%			
	#50	0.300		92.7%		
	#60	0.250	91.5%			
	#80	0.180		88.9%		
	#100	0.150	87.5%			
	#140	0.106		83.9%		
	#170	0.090		82.3%		
	#200	0.075	80.4%			



DATE TESTED 03/06/17
TESTED BY RTT/MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT
Green Mountain North Development
2817 NE Ingle Road
Camas, Washington

CLIENT
Mr. John Schmidt
Green Mountain Land, LLC
17933 NW Evergreen Parkway, Suite 300
Beaverton, Oregon 97006

PROJECT NO. 17012	LAB ID S17-100
REPORT DATE 03/09/17	FIELD ID TP14.1
DATE SAMPLED 02/14/17	SAMPLED BY ASR

MATERIAL DATA

MATERIAL SAMPLED
Sandy Lean CLAY

MATERIAL SOURCE
Test Pit TP-14
depth = 3.5 feet

USCS SOIL TYPE
CL, Sandy Lean Clay

SPECIFICATIONS
none

AASHTO SOIL TYPE
A-6(5)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter 637

TEST PROCEDURE
ASTM D6913, D422

ADDITIONAL DATA

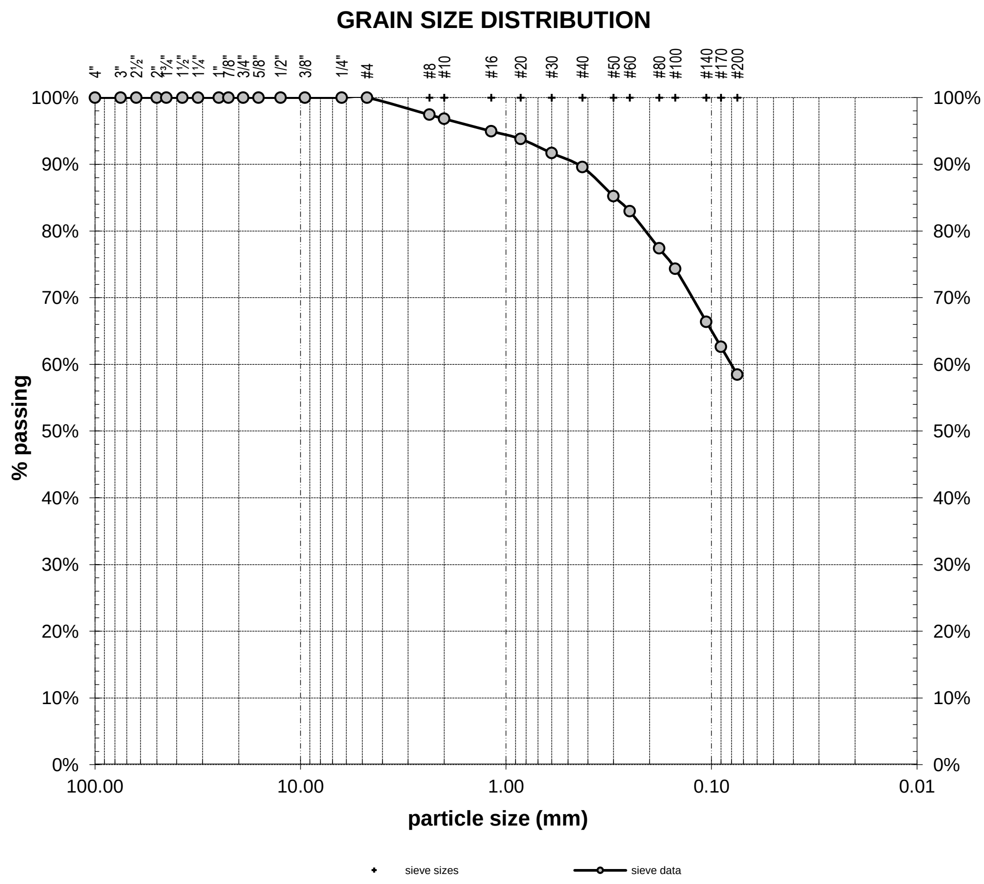
initial dry mass (g) = 179.2
as-received moisture content = 29.8%
liquid limit = 34
plastic limit = 22
plasticity index = 12
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = 0.080 mm

SIEVE DATA

% gravel = 0.0%
% sand = 41.5%
% silt and clay = 58.5%

	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		97.4%		
	#10	2.00	96.8%			
	#16	1.18		95.0%		
	#20	0.850	93.8%			
	#30	0.600		91.7%		
	#40	0.425	89.6%			
	#50	0.300		85.2%		
	#60	0.250	82.9%			
	#80	0.180		77.4%		
	#100	0.150	74.3%			
	#140	0.106		66.4%		
	#170	0.090		62.6%		
	#200	0.075	58.5%			



DATE TESTED 03/06/17	TESTED BY RTT/MJR

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain North Development 2817 NE Ingle Road Camas, Washington	Mr. John Schmidt Green Mountain Land, LLC 17933 NW Evergreen Parkway, Suite 300 Beaverton, Oregon 97006	17012	S17-100
		REPORT DATE	FIELD ID
		03/09/17	TP14.1
		DATE SAMPLED	SAMPLED BY
		02/14/17	ASR

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Lean CLAY	Test Pit TP-14 depth = 3.5 feet	CL, Sandy Lean Clay

LABORATORY TEST DATA

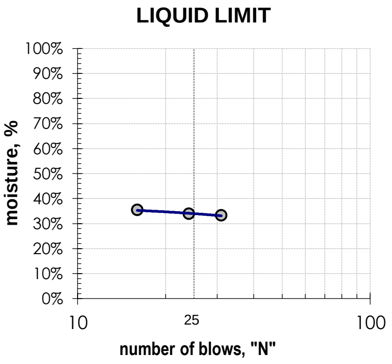
LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

ATTERBERG LIMITS

liquid limit = 34
plastic limit = 22
plasticity index = 12

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	41.68	37.11	34.46	
dry soil + pan weight, g =	36.46	33.01	30.83	
pan weight, g =	20.75	20.91	20.57	
N (blows) =	31	24	16	
moisture, % =	33.2 %	33.9 %	35.4 %	



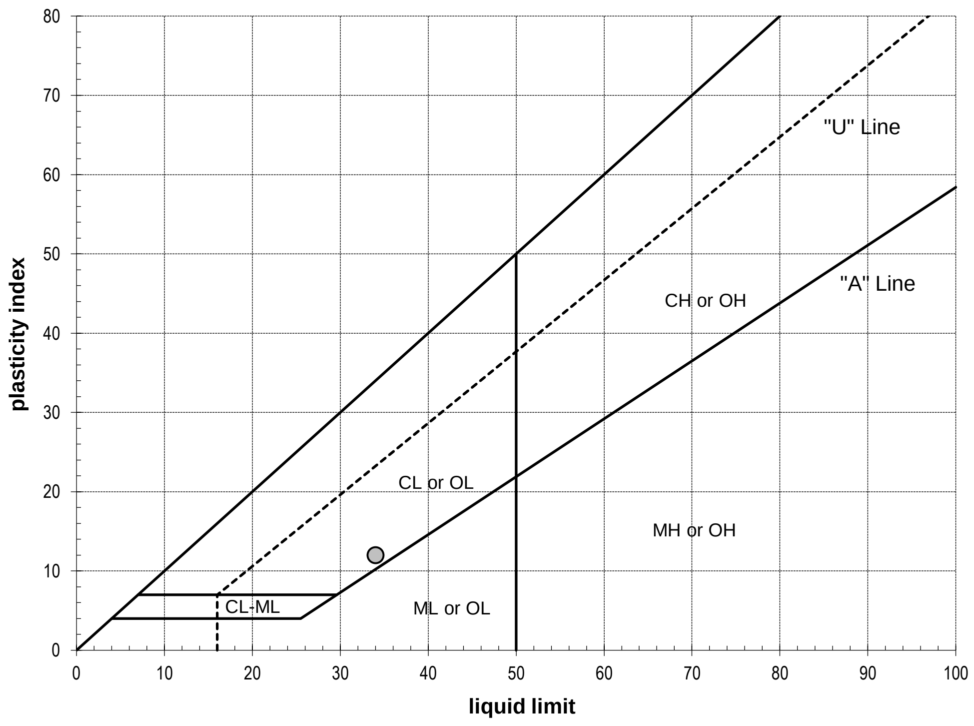
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.28	28.89		
dry soil + pan weight, g =	26.89	27.43		
pan weight, g =	20.69	20.85		
moisture, % =	22.4 %	22.2 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	41.5%
% silt and clay =	58.5%
% silt =	n/a
% clay =	n/a
moisture content =	29.8%

DATE TESTED	TESTED BY
03/08/17	RTT/MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT
Green Mountain North Development
2817 NE Ingle Road
Camas, Washington

CLIENT
Mr. John Schmidt
Green Mountain Land, LLC
17933 NW Evergreen Parkway, Suite 300
Beaverton, Oregon 97006

PROJECT NO. 17012
LAB ID S17-112
REPORT DATE 03/23/17
FIELD ID SB1.5
DATE SAMPLED 03/02/17
SAMPLED BY CWS

MATERIAL DATA

MATERIAL SAMPLED
Sandy Elastic SILT

MATERIAL SOURCE
Soil Boring SB-01
depth = 20 feet

USCS SOIL TYPE
MH, Sandy Elastic Silt

SPECIFICATIONS
none

AASHTO SOIL TYPE
A-7-5(11)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter 637

TEST PROCEDURE
ASTM D6913, D422

ADDITIONAL DATA

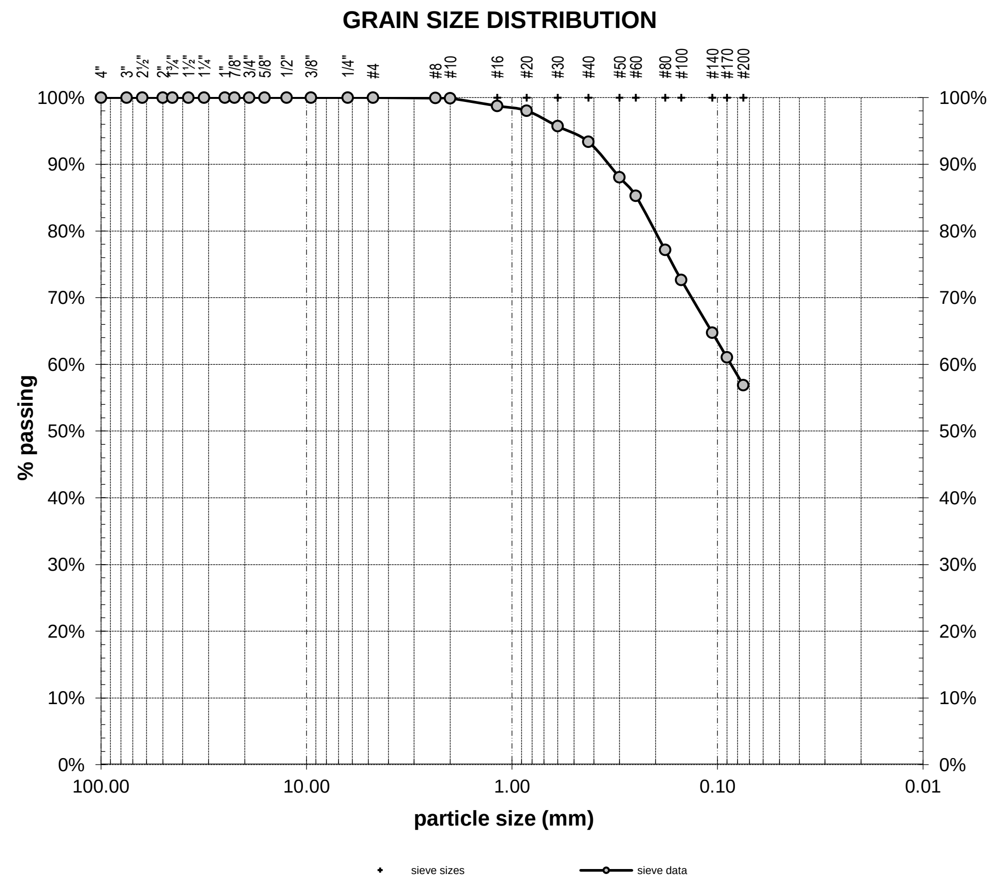
initial dry mass (g) = 121.8
as-received moisture content = 64.8%
liquid limit = 56
plastic limit = 35
plasticity index = 21
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = 0.086 mm

SIEVE DATA

% gravel = 0.0%
% sand = 43.1%
% silt and clay = 56.9%

	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		98.7%		
	#20	0.850	98.0%			
	#30	0.600		95.7%		
	#40	0.425	93.4%			
	#50	0.300		88.0%		
	#60	0.250	85.3%			
	#80	0.180		77.1%		
	#100	0.150	72.6%			
	#140	0.106		64.8%		
	#170	0.090		61.0%		
	#200	0.075	56.9%			



DATE TESTED
03/14/17

TESTED BY
RTT/JJC

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain North Development 2817 NE Ingle Road Camas, Washington	Mr. John Schmidt Green Mountain Land, LLC 17933 NW Evergreen Parkway, Suite 300 Beaverton, Oregon 97006	17012	S17-112
		REPORT DATE	FIELD ID
		03/23/17	SB1.5
		DATE SAMPLED	SAMPLED BY
		03/02/17	CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Elastic SILT	Soil Boring SB-01 depth = 20 feet	MH, Sandy Elastic Silt

LABORATORY TEST DATA

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

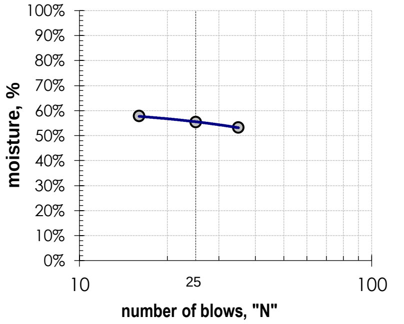
ATTERBERG LIMITS

liquid limit = 56
plastic limit = 35
plasticity index = 21

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	36.73	35.51	35.32	
dry soil + pan weight, g =	31.23	30.20	29.99	
pan weight, g =	20.89	20.62	20.76	
N (blows) =	35	25	16	
moisture, % =	53.2 %	55.4 %	57.8 %	

LIQUID LIMIT



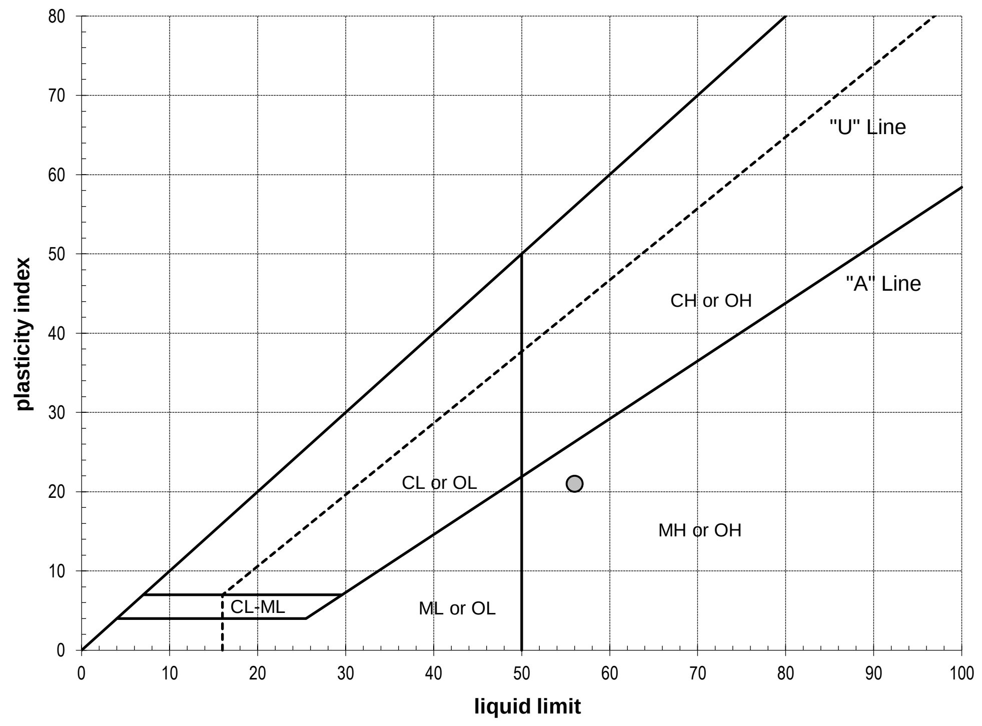
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.75	27.89		
dry soil + pan weight, g =	25.95	26.07		
pan weight, g =	20.69	20.84		
moisture, % =	34.2 %	34.8 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 43.1%
% silt and clay = 56.9%
% silt = n/a
% clay = n/a
moisture content = 64.8%

DATE TESTED	TESTED BY
03/14/17	RTT/JJC

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain North Development 2817 NE Ingle Road Camas, Washington	Mr. John Schmidt Green Mountain Land, LLC 17933 NW Evergreen Parkway, Suite 300 Beaverton, Oregon 97006	17012	S17-113
		REPORT DATE 03/23/17	FIELD ID SB1.6
		DATE SAMPLED 03/02/17	SAMPLED BY CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
SILT with Sand	Soil Boring SB-01 depth = 25.5 feet	ML, Silt with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-4(0)

LABORATORY TEST DATA

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

ADDITIONAL DATA

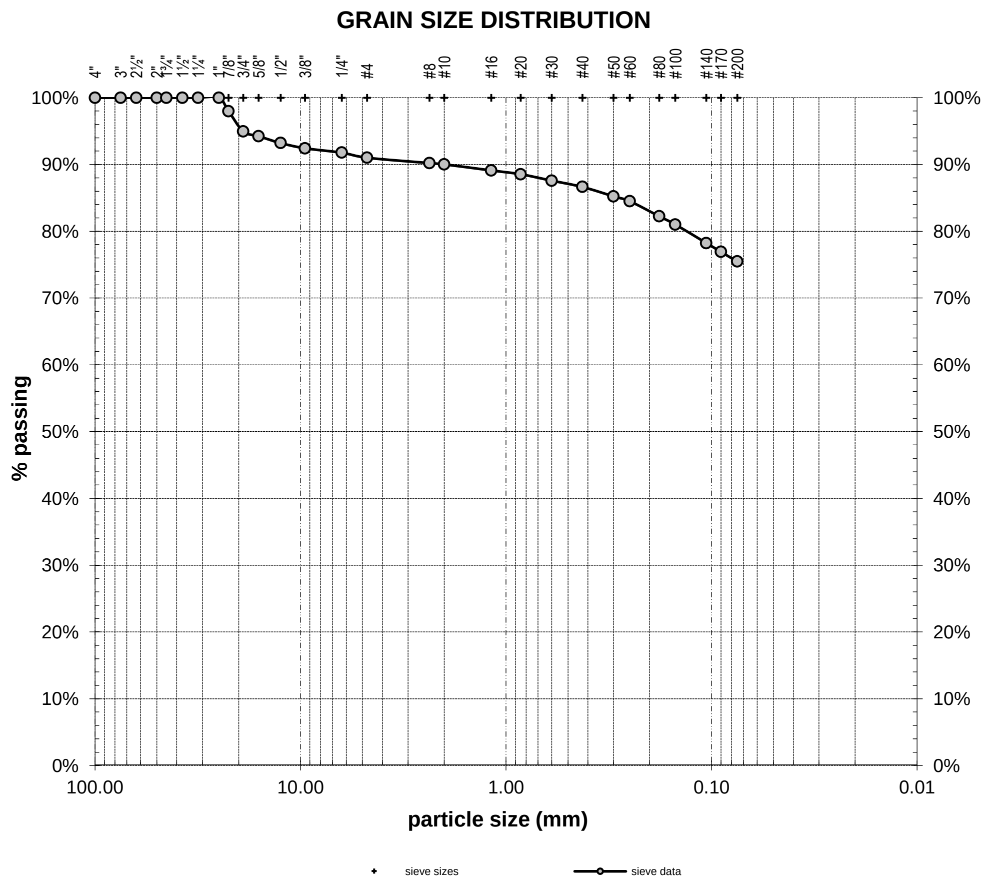
initial dry mass (g) = 395.9
as-received moisture content = 44.5%
liquid limit = -
plastic limit = -
plasticity index = NP
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = n/a

SIEVE DATA

% gravel = 9.0%
% sand = 15.5%
% silt and clay = 75.5%

	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		98.0%		
	3/4"	19.0	94.9%			
	5/8"	16.0		94.2%		
	1/2"	12.5	93.2%			
	3/8"	9.50	92.4%			
	1/4"	6.30	91.8%			
	#4	4.75	91.0%			
SAND	#8	2.36		90.2%		
	#10	2.00	90.0%			
	#16	1.18		89.1%		
	#20	0.850	88.5%			
	#30	0.600		87.6%		
	#40	0.425	86.6%			
	#50	0.300		85.2%		
	#60	0.250	84.5%			
	#80	0.180		82.3%		
	#100	0.150	81.0%			
	#140	0.106		78.2%		
	#170	0.090		76.9%		
	#200	0.075	75.5%			



DATE TESTED	TESTED BY
03/14/17	RTT/JJC

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ATTERBERG LIMITS REPORT

PROJECT
Green Mountain North Development
2817 NE Ingle Road
Camas, Washington

CLIENT
Mr. John Schmidt
Green Mountain Land, LLC
17933 NW Evergreen Parkway, Suite 300
Beaverton, Oregon 97006

PROJECT NO. 17012	LAB ID S17-113
REPORT DATE 03/23/17	FIELD ID SB1.6
DATE SAMPLED 03/02/17	SAMPLED BY CWS

MATERIAL DATA

MATERIAL SAMPLED
SILT with Sand

MATERIAL SOURCE
Soil Boring SB-01
depth = 25.5 feet

USCS SOIL TYPE
ML, Silt with Sand

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Liquid Limit Machine, Hand Rolled

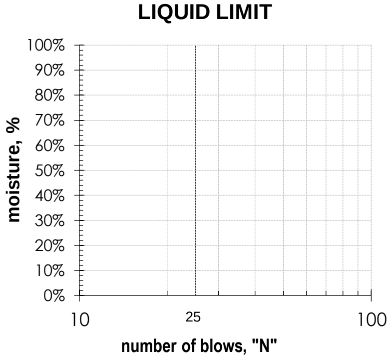
TEST PROCEDURE
ASTM D4318

ATTERBERG LIMITS

liquid limit = 0
plastic limit = 0
plasticity index = 0

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =				
dry soil + pan weight, g =				
pan weight, g =				
N (blows) =				
moisture, % =				



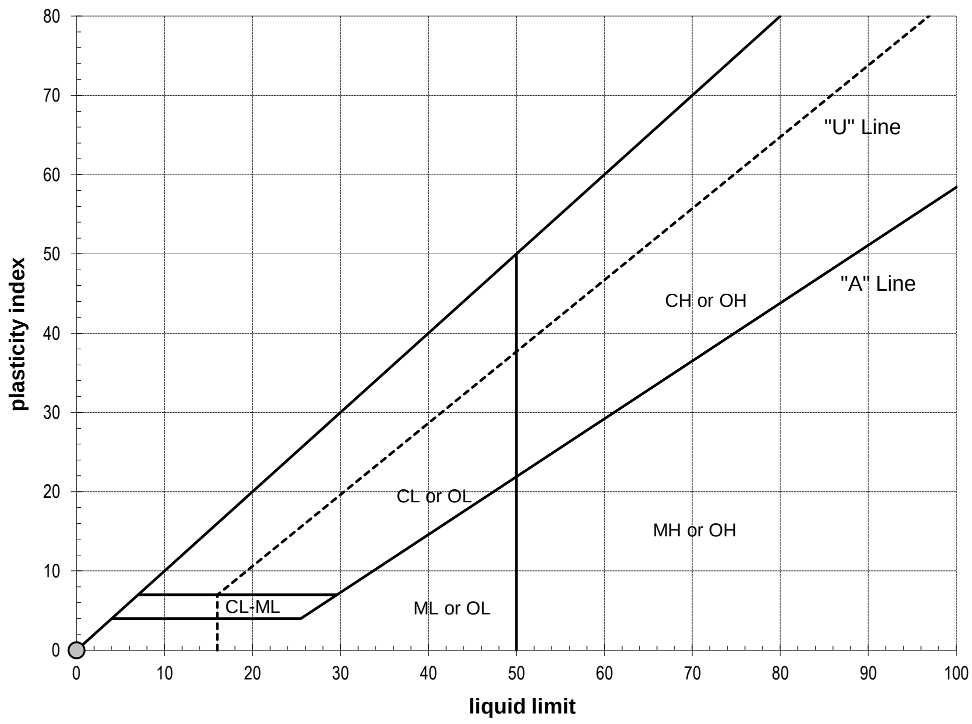
LIQUID LIMIT

SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =				
dry soil + pan weight, g =				
pan weight, g =				
moisture, % =				



PLASTICITY CHART

ADDITIONAL DATA

% gravel = 9.0%
% sand = 15.5%
% silt and clay = 75.5%
% silt = n/a
% clay = n/a
moisture content = 44.5%

DATE TESTED	TESTED BY

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT
Green Mountain North Development
2817 NE Ingle Road
Camas, Washington

CLIENT
Mr. John Schmidt
Green Mountain Land, LLC
17933 NW Evergreen Parkway, Suite 300
Beaverton, Oregon 97006

PROJECT NO. 17012	LAB ID S17-114
REPORT DATE 03/23/17	FIELD ID SB3.5
DATE SAMPLED 03/03/17	SAMPLED BY CWS

MATERIAL DATA

MATERIAL SAMPLED
Elastic SILT with Sand

MATERIAL SOURCE
Soil Boring SB-03
depth = 15 feet

USCS SOIL TYPE
MH, Elastic Silt with Sand

SPECIFICATIONS
none

AASHTO SOIL TYPE
A-7-5(21)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter 637

TEST PROCEDURE
ASTM D6913, D422

ADDITIONAL DATA

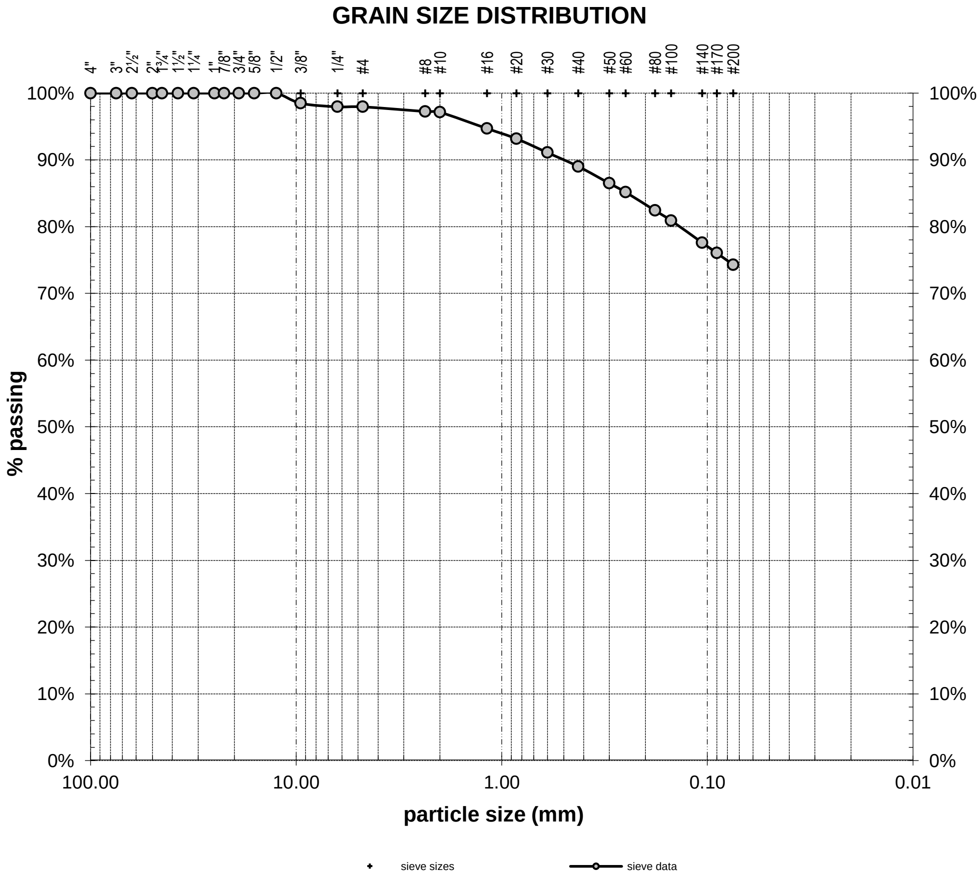
initial dry mass (g) = 375.2
as-received moisture content = 58.1%
liquid limit = 58
plastic limit = 31
plasticity index = 27
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = n/a

SIEVE DATA

% gravel = 2.1%
% sand = 23.6%
% silt and clay = 74.3%

	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	98.5%			
	1/4"	6.30	97.9%			
	#4	4.75	97.9%			
SAND	#8	2.36		97.3%		
	#10	2.00	97.1%			
	#16	1.18		94.7%		
	#20	0.850	93.2%			
	#30	0.600		91.1%		
	#40	0.425	89.0%			
	#50	0.300		86.5%		
	#60	0.250	85.2%			
	#80	0.180		82.4%		
	#100	0.150	80.9%			
	#140	0.106		77.6%		
	#170	0.090		76.0%		
	#200	0.075	74.3%			



DATE TESTED 03/17/17	TESTED BY RTT/JJC

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain North Development 2817 NE Ingle Road Camas, Washington	Mr. John Schmidt Green Mountain Land, LLC 17933 NW Evergreen Parkway, Suite 300 Beaverton, Oregon 97006	17012	S17-114
		REPORT DATE 03/23/17	FIELD ID SB3.5
		DATE SAMPLED 03/03/17	SAMPLED BY CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Elastic SILT with Sand	Soil Boring SB-03 depth = 15 feet	MH, Elastic Silt with Sand

LABORATORY TEST DATA

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

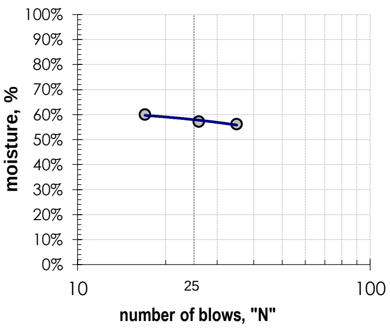
ATTERBERG LIMITS

liquid limit = 58
plastic limit = 31
plasticity index = 27

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.48	32.84	36.36	
dry soil + pan weight, g =	29.55	28.50	30.44	
pan weight, g =	20.76	20.91	20.57	
N (blows) =	35	26	17	
moisture, % =	56.1 %	57.2 %	60.0 %	

LIQUID LIMIT



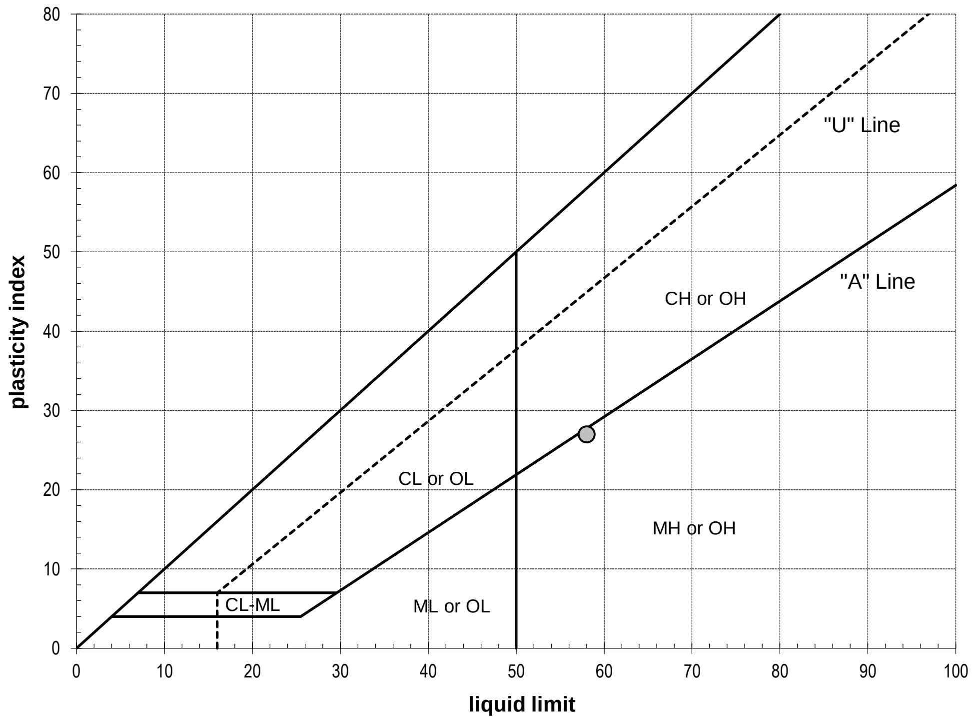
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.87	28.11		
dry soil + pan weight, g =	26.15	26.38		
pan weight, g =	20.70	20.84		
moisture, % =	31.6 %	31.2 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 2.1%
% sand = 23.6%
% silt and clay = 74.3%
% silt = n/a
% clay = n/a
moisture content = 58.1%

DATE TESTED	TESTED BY
03/17/17	RTT/JJC

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Green Mountain North Development 2817 NE Ingle Road Camas, Washington	Mr. John Schmidt Green Mountain Land, LLC 17933 NW Evergreen Parkway, Suite 300 Beaverton, Oregon 97006	17012	S17-115
		REPORT DATE 03/23/17	FIELD ID SB3.9
		DATE SAMPLED 03/03/17	SAMPLED BY CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty SAND with Gravel	Soil Boring SB-03 depth = 35 feet	SM, Silty Sand with Gravel

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-5(3)

LABORATORY TEST DATA

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

ADDITIONAL DATA

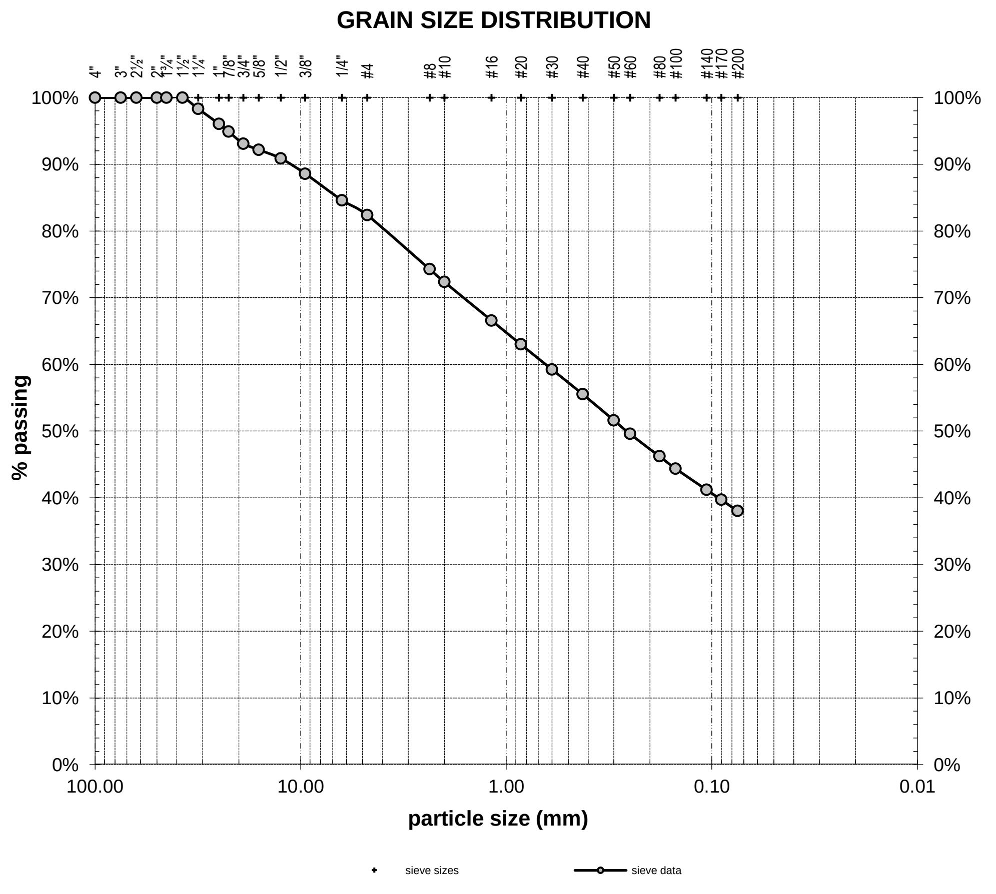
initial dry mass (g) = 693.7
as-received moisture content = 45.1%
liquid limit = 56
plastic limit = 35
plasticity index = 21
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = 0.644 mm

SIEVE DATA

% gravel = 17.6%
% sand = 44.4%
% silt and clay = 38.0%

	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		98.3%		
	1.00"	25.0	96.1%			
	7/8"	22.4		94.9%		
	3/4"	19.0	93.1%			
	5/8"	16.0		92.2%		
	1/2"	12.5	90.9%			
	3/8"	9.50	88.6%			
	1/4"	6.30	84.6%			
	#4	4.75	82.4%			
SAND	#8	2.36		74.3%		
	#10	2.00	72.3%			
	#16	1.18		66.6%		
	#20	0.850	63.0%			
	#30	0.600		59.2%		
	#40	0.425	55.5%			
	#50	0.300		51.6%		
	#60	0.250	49.6%			
	#80	0.180		46.2%		
	#100	0.150	44.4%			
	#140	0.106		41.2%		
	#170	0.090		39.7%		
	#200	0.075	38.0%			



DATE TESTED	TESTED BY
03/16/17	MJR

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT
Green Mountain North Development
2817 NE Ingle Road
Camas, Washington

CLIENT
Mr. John Schmidt
Green Mountain Land, LLC
17933 NW Evergreen Parkway, Suite 300
Beaverton, Oregon 97006

PROJECT NO. 17012
LAB ID S17-115
REPORT DATE 03/23/17
FIELD ID SB3.9
DATE SAMPLED 03/03/17
SAMPLED BY CWS

MATERIAL DATA

MATERIAL SAMPLED
Silty SAND with Gravel

MATERIAL SOURCE
Soil Boring SB-03
depth = 35 feet

USCS SOIL TYPE
SM, Silty Sand with Gravel

SPECIFICATIONS
none

AASHTO SOIL TYPE
A-7-5(3)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter 637

TEST PROCEDURE
ASTM D6913, D422

ADDITIONAL DATA

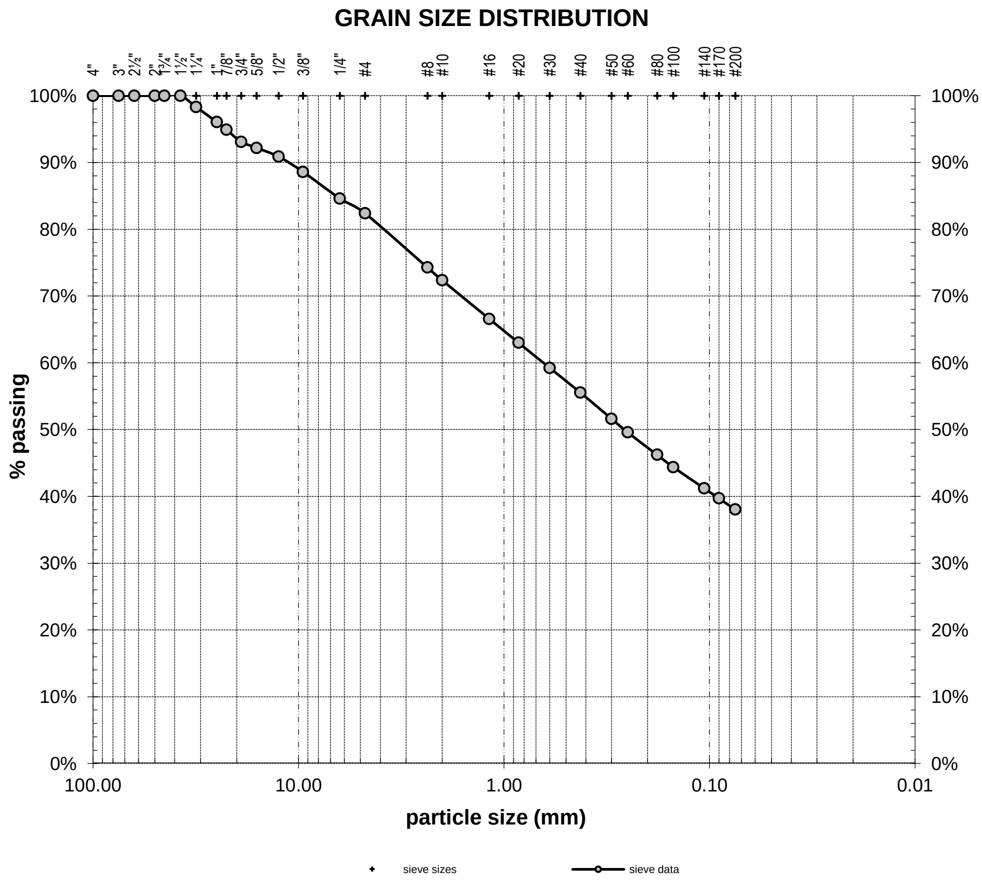
initial dry mass (g) = 693.7
as-received moisture content = 45.1%
liquid limit = 56
plastic limit = 35
plasticity index = 21
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = 0.644 mm

SIEVE DATA

% gravel = 17.6%
% sand = 44.4%
% silt and clay = 38.0%

	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		98.3%		
	1.00"	25.0	96.1%			
	7/8"	22.4		94.9%		
	3/4"	19.0	93.1%			
	5/8"	16.0		92.2%		
	1/2"	12.5	90.9%			
	3/8"	9.50	88.6%			
	1/4"	6.30	84.6%			
	#4	4.75	82.4%			
SAND	#8	2.36		74.3%		
	#10	2.00	72.3%			
	#16	1.18		66.6%		
	#20	0.850	63.0%			
	#30	0.600		59.2%		
	#40	0.425	55.5%			
	#50	0.300		51.6%		
	#60	0.250	49.6%			
	#80	0.180		46.2%		
	#100	0.150	44.4%			
	#140	0.106		41.2%		
	#170	0.090		39.7%		
	#200	0.075	38.0%			



DATE TESTED 03/16/17
TESTED BY MJR

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

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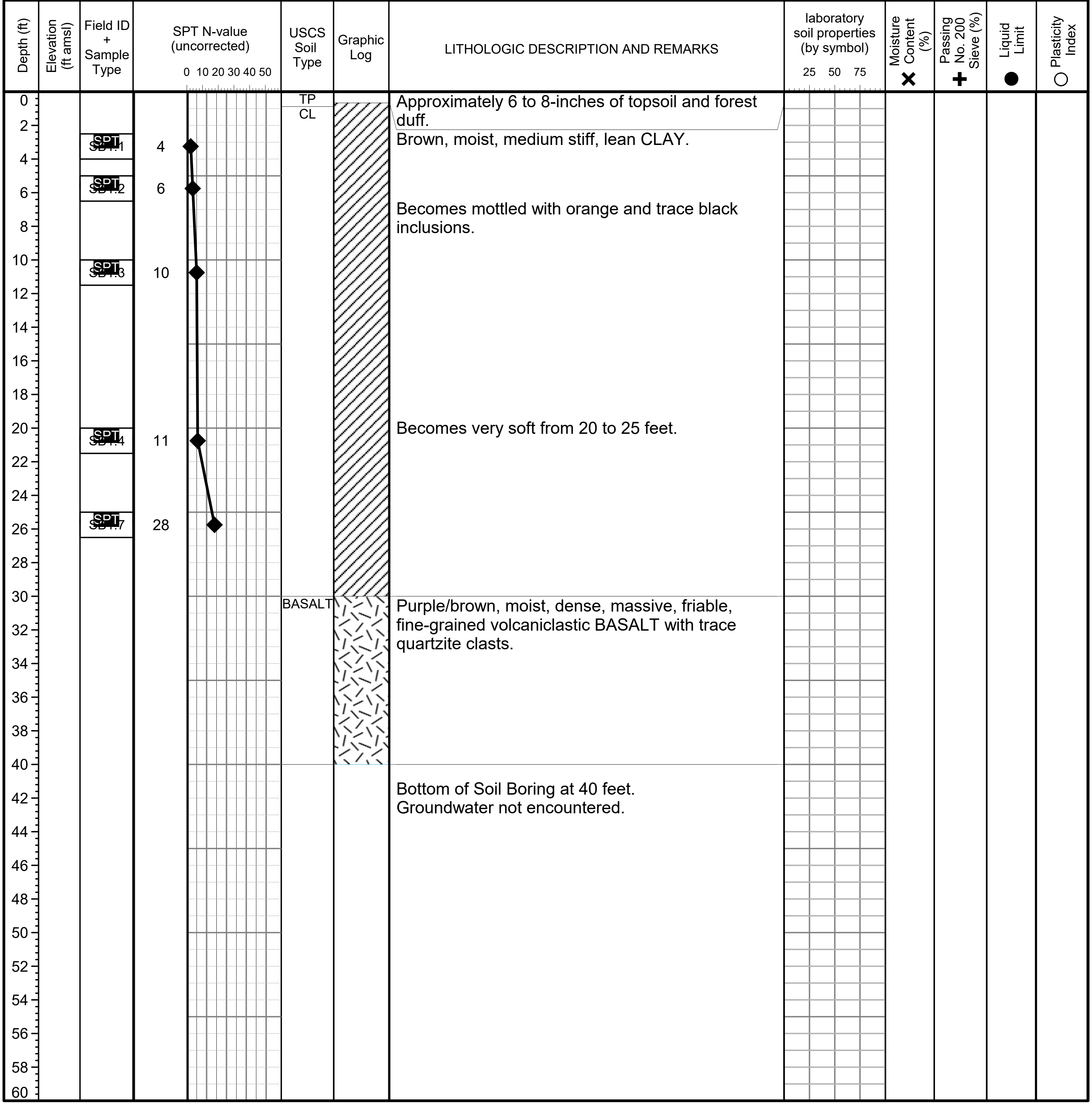
SOIL BORING LOG

PROJECT NAME Green Mountain North	CLIENT Green Mountain Land, LLC.	PROJECT NO. 17012	BORING NO. SB-1
PROJECT LOCATION Camas, Wa	DRILLING CONTRACTOR Western States DRILL RIG CME 850	ENGINEER/GEOLOGIST CWS	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD Mud Rotary SAMPLING METHOD SPT	START DATE 3/2/17	START TIME 1430
REMARKS	APPROX. SURFACE ELEVATION GROUNDWATER DEPTH Not Encountered	FINISH DATE 3/2/17	FINISH TIME 1700



Field ID + Sample Type	SPT N-value (uncorrected)
SPT SB1.1	4
SPT SB1.2	6
SPT SB1.3	10
SPT SB1.4	11
SPT SB1.7	28

USCS Soil Type	LITHOLOGIC DESCRIPTION AND REMARKS
TP	Approximately 6 to 8-inches of topsoil and forest duff.
CL	Brown, moist, medium stiff, lean CLAY. Becomes mottled with orange and trace black inclusions. Becomes very soft from 20 to 25 feet.
BASALT	Purple/brown, moist, dense, massive, friable, fine-grained volcaniclastic BASALT with trace quartzite clasts.
	Bottom of Soil Boring at 40 feet. Groundwater not encountered.

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SOIL BORING LOG

PROJECT NAME Green Mountain North	CLIENT Green Mountain Land, LLC.		PROJECT NO. 17012	BORING NO. SB-2
PROJECT LOCATION Camas, Wa	DRILLING CONTRACTOR Western States	DRILL RIG CME 850	ENGINEER/GEOLOGIST CWS	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD Mud Rotary	SAMPLING METHOD SPT	START DATE 3/2/17	START TIME 1042
REMARKS	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH Not Encountered	FINISH DATE 3/2/17	FINISH TIME 1345

Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected) 0 10 20 30 40 50	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	laboratory soil properties (by symbol) 25 50 75	Moisture Content (%) ✕	Passing No. 200 Sieve (%) ✚	Liquid Limit ●	Plasticity Index ○
0				TP		Approximately 6 to 8-inches of topsoil and forest duff.					
				CL	▨	Brown/gray with orangle mottling, moist, medium stiff, lean CLAY mixed with fractured basalt.					
		SPT SB2.1	50/3"	BASALT		Gray, moist, dense, friable, moderately weathered olivine-phyric andesitic BASALT.					
		SPT SB2.2	50/2"								
						Bottom of Soil Boring at 15 feet. Groundwater not encountered.					

Depth (ft): 0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60

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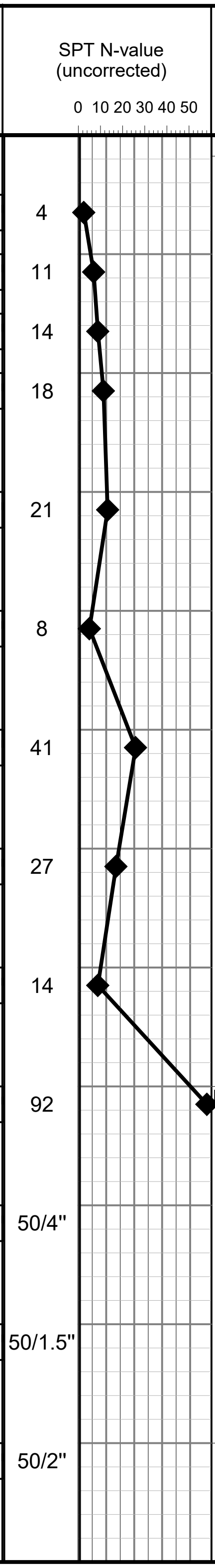
SOIL BORING LOG

PROJECT NAME Green Mountain North	CLIENT Green Mountain Land, LLC.		PROJECT NO. 17012	BORING NO. SB-3
PROJECT LOCATION Camas, Wa	DRILLING CONTRACTOR Western States	DRILL RIG CME 850	ENGINEER/GEOLOGIST CWS	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD Mud Rotary	SAMPLING METHOD SPT	START DATE 3/3/17	START TIME 0900
REMARKS	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH Not Encountered	FINISH DATE 3/3/17	FINISH TIME 1300



Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	LITHOLOGIC DESCRIPTION AND REMARKS	laboratory soil properties (by symbol) 25 50 75	Moisture Content (%) ✕	Passing No. 200 Sieve (%) ✚	Liquid Limit ●	Plasticity Index ○
0				TP	Approximately 6 to 8-inches of topsoil and forest duff.					
1				CL	Brown/blue, moist, medium stiff, lean CLAY with gravels.					
3		SPT SB3.1	4							
4					Becomes brown/red with orange mottling and black inclusions.					
6		SPT SB3.2	11							
8		SPT SB3.3	14							
10		SPT SB3.4	18							
15					Becomes brown/gray.					
16		SPT SB3.5	21							
20		SPT SB3.6	8							
24					Becomes brown/gray/purple, stiff, mixed weathered basalt fragments. Weathering increases with depth.					
26		SPT SB3.7	41							
30		SPT SB3.8	27							
36		SPT SB3.9	14							
40		SPT SB3.10	92	BASALT	Gray, moist, dense, friable, moderately weathered olivine-phyric andesitic BASALT.					
46		SPT SB3.11	50/4"							
50		SPT SB3.12	50/1.5"							
56		SPT SB3.13	50/2"		Bottom of Soil Boring at 50 feet. Groundwater not encountered.					

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-1
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 280 ft amsl	GROUNDWATER DEPTH Not encountered	START TIME 0800	FINISH TIME 0830

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				TS		Approximately two feet of topsoil intermixed with forest duff, understory vegatation, and one to four foot diameter boulders.					
	TP-1.1		A-6(2)	SM		Light brown, moist, medium stiff, medium plasticity, silty SAND with gravels.	32.6	46.7	37	11	
				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderately weathered basaltic ANDESITE.					
5						Bottom of test pit at 4.5 feet due to excavator refusal. Groundwater not encountered.					
10											
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-2
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 270 ft amsl	GROUNDWATER DEPTH Not encountered	START TIME 0831	FINISH TIME 0845

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0				TS		Approximately two feet of topsoil intermixed with forest duff, understory vegatation, and one to four foot diameter boulders.					
				CL-SM		Light brown, moist, medium stiff, medium plasticity, lean CLAY with gravels.					
5				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT.					
						Bottom of test pit at 7 feet due to excavator refusal. Groundwater not encountered.					
10											
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-3
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 325 ft amsl	GROUNDWATER DEPTH Not encountered	START TIME 0840	FINISH TIME 0856

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				TS		Approximately two feet of topsoil intermixed with forest duff, understory vegatation, and one to two foot diameter boulders.					
				CL-SM		Light brown, moist, medium stiff, medium plasticity, lean CLAY with gravels.					
				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT.					
5						Bottom of test pit at 5 feet due to excavator refusal. Groundwater not encountered.					
10											
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-4
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 370 ft amsl	GROUNDWATER DEPTH Not encountered	START TIME 1225	FINISH TIME 1240

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				TS		Approximately two feet of topsoil intermixed with forest duff, understory vegatation, and cobbles.					
				BA		Gray, moist, medium dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT.					
						Bottom of test pit at 4 feet due to excavator refusal. Groundwater not encountered.					
5											
10											
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-5
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 380 ft amsl	GROUNDWATER DEPTH 5 feet bgs	START TIME 1200	FINISH TIME 1220

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				TS		Approximately 12-inches of topsoil intermixed with forest duff and understory vegatation.					
▼ 5				CL-SM		Light brown, moist, medium stiff, medium plasticity, lean CLAY with gravels.					
10				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weatheredandesitic BASALT.					
15						Bottom of test pit at 11 feet due to excavator refusal. Groundwater encountered at approximately 5 feet.					

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-6
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 390 ft amsl	GROUNDWATER DEPTH Seeps at 9 feet bgs	START TIME 0930	FINISH TIME 1000

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				TS		Approximately 12-inches of topsoil intermixed with forest duff, understory vegatation, and basaltic ANDESITE fragements.					
				CL-CH	▨	Light brown, moist, medium stiff, medium plasticity, lean CLAY with gravels.					
5											
▼											
10	TP-6.2		A-7-6(16)				29.2	62.5	53	29	
15						Bottom of test pit at 15 feet. Seepage encountered at approximately 9 feet.					

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TEST PIT LOG

PROJECT NAME: Green Mountain North Development
CLIENT: John Schmidt
PROJECT NO.: 17012
TEST PIT NO.: TP-7
PROJECT LOCATION: Camas, Washington
CONTRACTOR: L&S Excavating
EQUIPMENT: Excavator
ENGINEER: CWS
DATE: 2/14/17
TEST PIT LOCATION: See Figure 2
APPROX. SURFACE ELEVATION: 440 ft amsl
GROUNDWATER DEPTH: Not encountered
START TIME: 1005
FINISH TIME: 1048

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				TS		Approximately two feet of topsoil intermixed with forest duff, and understory vegatation.					
				CL-SM	▨	Light brown, moist, medium stiff, medium plasticity, lean CLAY with gravels.					
5											
10											
15						Bottom of test pit at 15 feet. Groundwater not encountered.					

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt	PROJECT NO. 17012	TEST PIT NO. TP-8
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 515 ft amsl GROUNDWATER DEPTH Not encountered	START TIME 1053	FINISH TIME 1107

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				TS		Approximately two feet of topsoil intermixed with forest duff, understory vegatation, and one to two foot diameter boulders.					
				CL-SM		Light brown, moist, medium stiff, medium plasticity, lean CLAY with gravels.					
				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT.					
						Bottom of test pit at 3 feet due to excavator refusal. Groundwater not encountered.					
5											
10											
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-9
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 500 ft amsl	GROUNDWATER DEPTH Not encountered	START TIME 1115	FINISH TIME 1145

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				TS		Approximately 12-inches of topsoil intermixed with forest duff and understory vegatation.					
				CL-SM	▨	Light brown, moist, medium stiff, medium plasticity, lean CLAY with gravels.					
	TP-9.1		A-7-6(35)		▨		41.4	80.4	66	40	
5					▨						
				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT intermixed with light brown, moist, medium stiff, medium plasticity, lean CLAY [Soil Type 3].					
10						Bottom of test pit at 10 feet due to excavator refusal. Groundwater not encountered.					
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-10
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 280 ft amsl	GROUNDWATER DEPTH Not encountered	START TIME 1245	FINISH TIME 0950

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0				TS		Approximately 12-inches of topsoil intermixed with forest duff and understory vegatation.					
				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT.					
5											
						Bottom of test pit at 5.5 feet due to excavator refusal. Groundwater not encountered.					
10											
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-11
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 340 ft amsl	GROUNDWATER DEPTH Not encountered	START TIME 1300	FINISH TIME 1310

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				TS		Approximately 12-inches of topsoil intermixed with forest duff and understory vegatation.					
				CL-SM		Light brown, moist, medium stiff, medium plasticity, lean CLAY with gravels and 1 to four-inch cobbles.					
				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT.					
5						Bottom of test pit at 5 feet due to excavator refusal. Groundwater not encountered.					
10											
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt	PROJECT NO. 17012	TEST PIT NO. TP-12
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 350 ft amsl GROUNDWATER DEPTH Not encountered	START TIME 1352	FINISH TIME 1358

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				TS		Approximately two feet of topsoil intermixed with forest duff, understory vegatation, one to two foot diameter boulders, cobbles, and gravels.					
				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT.					
						Bottom of test pit at 3.5 feet due to excavator refusal. Groundwater not encountered.					
5											
10											
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-13
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 295 ft amsl	GROUNDWATER DEPTH Not encountered	START TIME 1400	FINISH TIME 1420

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				TS		Approximately 12-inches of topsoil intermixed with forest duff, understory vegatation, one to two foot diameter boulders, cobbles, and gravels.					
				CL-SM	▨	Light brown, moist, medium stiff, medium plasticity, lean CLAY intermixed with two to three foot diameter boulders, cobbles, and gravels.					
				BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT.					
5						Bottom of test pit at 3.5 feet due to excavator refusal. Groundwater not encountered.					
10											
15											

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TEST PIT LOG

PROJECT NAME Green Mountain North Development	CLIENT John Schmidt		PROJECT NO. 17012	TEST PIT NO. TP-14
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Excavating	EQUIPMENT Excavator	ENGINEER CWS	DATE 2/14/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 225 ft amsl	GROUNDWATER DEPTH Not encountered	START TIME 1329	FINISH TIME 1345

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				TS		Approximately 12-inches of topsoil intermixed with forest duff and understory vegatation.					
				CL-SM		Light brown, moist, medium stiff, medium plasticity, silty lean CLAY with gravels.					
5	TP-14.1		A-6(5)	BA		Gray, moist, medium dense to dense, friable basaltic volcanic ASH and fragements of moderatley weathered andesitic BASALT.	29.8	58.5	34	12	
10 15						Bottom of test pit at 6 feet due to excavator refusal. Groundwater not encountered.					

APPENDIX C
SOIL CLASSIFICATION INFORMATION

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

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

Consistency for Cohesive Soil

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

Relative Density for Granular Soil

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

Moisture Designations

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

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

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

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

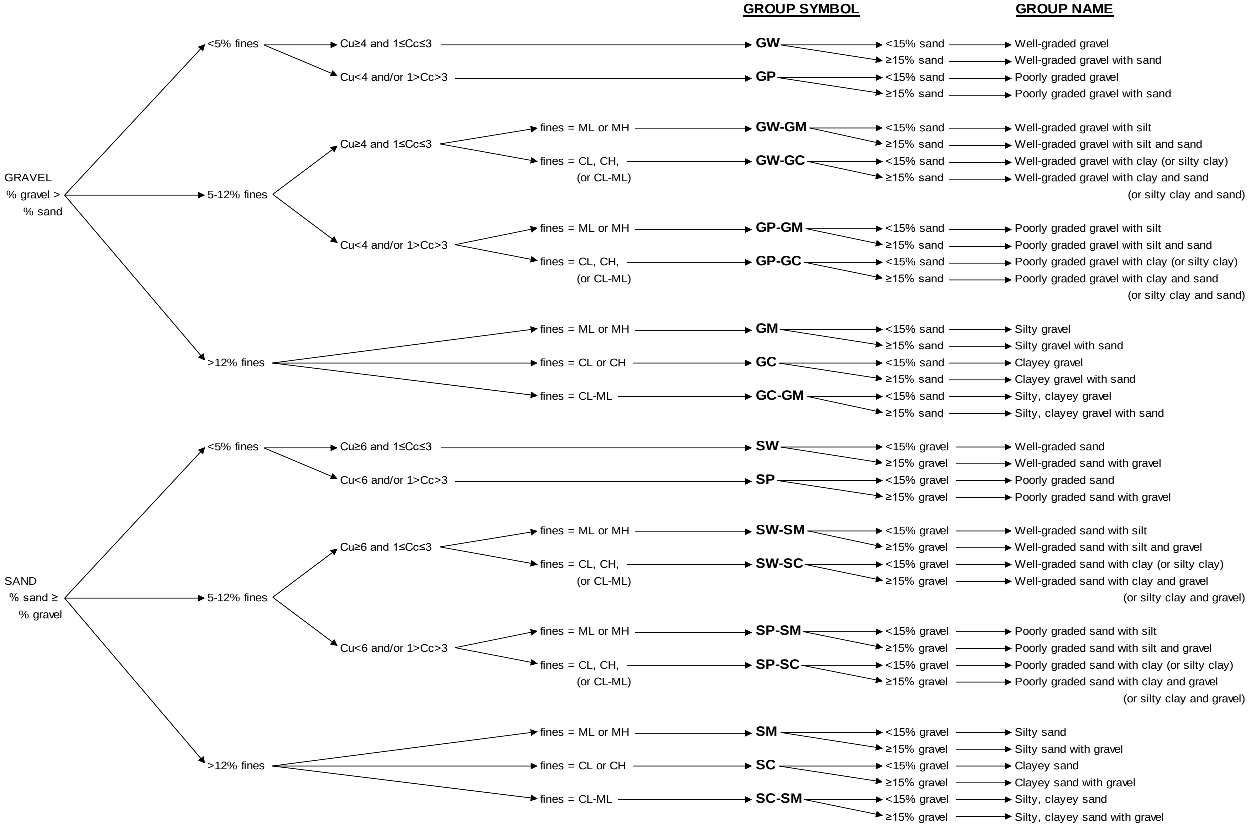
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

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

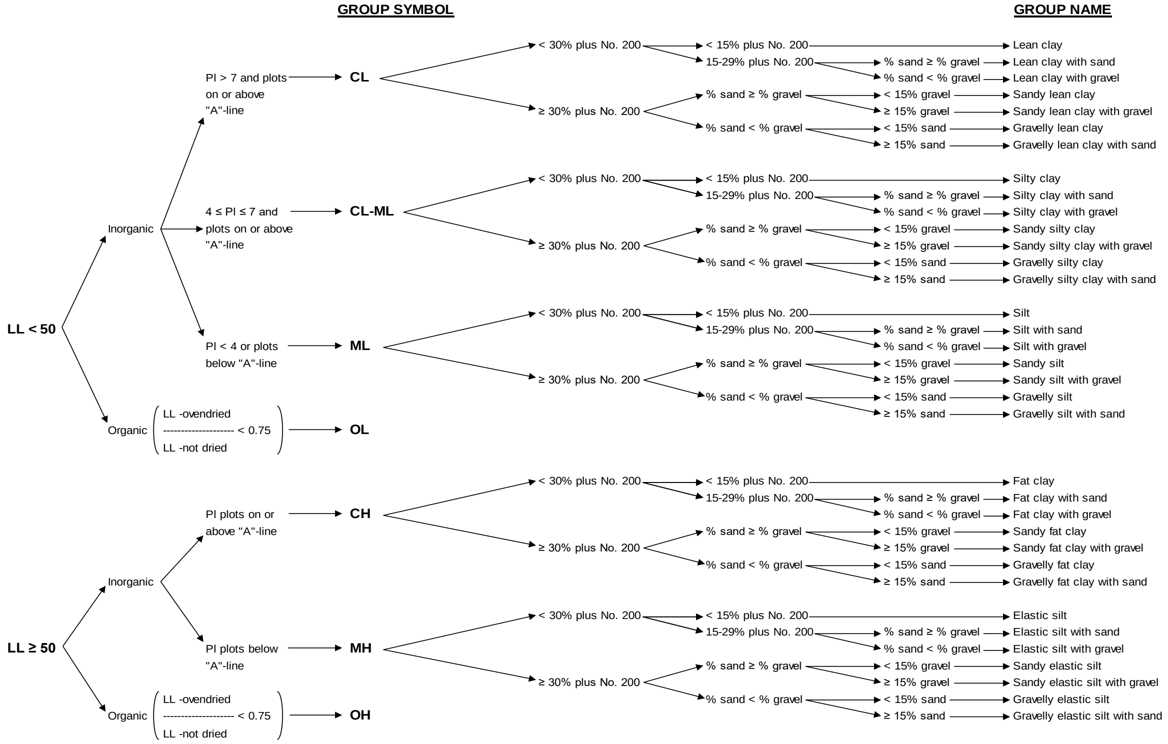
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

USCS SOIL CLASSIFICATION SYSTEM



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



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

APPENDIX D
SLOPE STABILITY ANALYSIS

APPENDIX D: SLOPE STABILITY ANALYSIS
GREEN MOUNTAIN NORTH, CAMAS, WASHINGTON

Analysis method

Slope stability analyses were performed using the slope stability modeling software SLOPE/W from Geo-Slope International, Ltd. SLOPE/W uses limit equilibrium method of slices to calculate factors of safety. The general limit equilibrium Morgenstern-Price method, which satisfies both force and moment equilibrium, was used for the analyses included in this report.

Soil Layers and Parameters

Two soil layers were used to describe existing conditions of the slopes:

Soil Type	Moist Unit Weight (pcf)	Cohesion (psf)	Drained Friction Angle (degrees)
ML-CL Soil (Silt and Clay surficial soils)	115	50	27
Basalt Bedrock	150	1000	35

Soil parameters used in the analyses were estimated based upon laboratory test results, soil and geologic data, and field observations. Soil parameters were generally conservatively chosen to account for the non-homogeneous nature of soils. Layer thicknesses were estimated and interpolated using information from boring, test pit, and topographic data as well as field observations. Not all soil layers were encountered in all soil borings.

Piezometric Surface

Piezometric surfaces were estimated based upon field observations of soils, groundwater, springs and seeps, and topography and review of well logs.

Determination of Seismic Coefficient

Various guidelines exist for selection of the seismic coefficient for use in pseudostatic slope stability analyses. Based upon the type of development, the size and geometry of the slope, existing soil and rock conditions, and local seismicity, a horizontal acceleration was selected for pseudostatic slope stability analyses. The horizontal acceleration is one-half of the peak ground acceleration for a 2,475-year return seismic event for the subject site (i.e., a seismic event with a 2 percent chance of occurring in the next 50 years).

Interpretation of Results

Columbia West analyzed two existing conditions cross-sections to determine slope stability at the property. Graphical results for cross-section A-A’ and B-B’ (see Figure 2 for cross-section location) are attached depicting critical slip surfaces for static and pseudostatic conditions at minimum acceptable factors of safety. Generally, slip surfaces emanating from behind the top of slope generally approached or exceeded the acceptable minimum factors of safety of 1.1 for pseudostatic conditions and 1.5 for static conditions, indicating potential failure along the slopes that separate the bench area from the valley floor.

In the following cross-section figures, individual soil layers are designated by color. Piezometric surfaces are indicated as dashed blue lines. Entry and exit ranges for slip surfaces are shown as red lines on the ground surface and were varied to determine worse-case scenarios. Soil boring locations are shown where appropriate. On each cross-section, the critical slip surface and the individual slices analyzed for the critical slip surface are shown in green. The lowest factor of safety for each analysis is also indicated. A discussion of the analyses is provided below.

Slope Analysis A-A'

Analysis A-A' represents one of the tallest west-facing slopes within the proposed development area in its existing geometry. Based upon the analysis, the factors of safety (FOS) for the critical slip surfaces behind the top of slope are 2.47 in a static condition and 1.59 in a pseudostatic condition, which exceed the minimum accepted values for slope stability. Analyzed critical slip surfaces encroach behind the top of slope in the static condition indicating the potential for impact to the development area. Therefore, a 35-foot setback is recommended at this location, as depicted on Figure 2.

Slope Analysis B-B'

Analysis B-B' represents a typical cinder cone slope within the proposed development area. Based upon this analysis, the factor of safety (FOS) for the critical slip surfaces behind the top of slope are 2.07 in a static condition and 1.22 in a pseudostatic condition, which exceed the minimum accepted values for slope stability. Because final grading contours are unknown at this time, additional analysis should be conducted once final grading contours are known.

APPENDIX E
PHOTO LOG

GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON
PHOTO LOG

Site Aerial Terrain, Facing Northwest

Site Aerial Terrain, Facing South.

GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON
PHOTO LOG

Conducting Soil Boring Exploration (SB-2).

GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON
PHOTO LOG

Typical Soil Profile Observed on the Site (TP-11).

GREEN MOUNTAIN NORTH
CAMAS, WASHINGTON
PHOTO LOG

Typical Basalt Coring Observed on the Site (SB-2, 5-15’BGS).

APPENDIX F
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Date: September 28, 2017
Project: Green Mountain North
Camas, Washington

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

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

Report Conclusions and Preliminary Nature

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

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

Additional Investigation and Construction QA/QC

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

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

Collected Samples

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

Report Contents

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

Report Limitations for Contractors

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

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Consultant Responsibility

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