Geotechnical Site Investigation

Stephens Property

La Center, Washington

October 20, 2017

Geotechnical ▪ Environmental ▪ Special Inspections

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GEOTECHNICAL SITE INVESTIGATION
STEPHENS PROPERTY
LA CENTER, WASHINGTON

Prepared For:	**Mark, Perry, and Carleen Stephens, Mary Rerick** **24600 NE 98th Court** **Battle Ground, Washington 98604**
Site Location:	**34700 Northeast North Fork Avenue** **Parcel Nos. 258901000, 258919000, 258922000, 258971000, and 258972000** **La Center, Washington**
Prepared By:	**Columbia West Engineering, Inc.** **11917 NE 95th Street** **Vancouver, Washington 98682** **Phone: 360-823-2900** **Fax: 360-823-2901**
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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

GEOTECHNICAL SITE INVESTIGATION
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1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Stephens-Rerick to conduct a geotechnical site investigation for the proposed Stephens Property project located in La Center, Washington. The purpose of the investigation was to observe and assess subsurface soil conditions at specific locations and provide geotechnical engineering analyses, planning, and design recommendations for proposed development. The specific scope of services was outlined in a proposal contract dated August 24, 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 D.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located at 34700 NE North Fork Avenue in La Center Washington. The site is bounded by rural residential development to the north, and west, NE North Fork Avenue to the east, residential development and undeveloped acreage to the south.

The site is comprised of tax parcels 258901000, 258919000, 258922000, 258971000, and 258972000 totaling approximately 42.03 acres. The regulatory jurisdictional agency for tax parcel 258901000 is the City of La Center, Washington. The regulatory jurisdictional agency for tax parcels 258919000, 258922000, 258971000, and 258972000 is Clark County, Washington. However, these parcels are scheduled to be annexed by the City of La Center, Washington concurrent with proposed development. The approximate latitude and longitude of the proposed subdivision is N 45° 52' 19" and W 122° 40' 15", and the legal description is a portion of the SW ¼ of Section 34, T5N, R1E, Willamette Meridian.

1.2 Proposed Development

Preliminary correspondence with the client indicates that proposed development will consist of single-family residential development. Proposed development is shown on Figure 2. Columbia West has not reviewed a preliminary grading plan, but understand that cut and fill areas are likely. This report is based upon proposed development as described above and may not be applicable if modified.

2.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

The subject site lies within the Willamette Valley/Puget Sound Lowland, a wide physiographic depression flanked by the mountainous Coast Range on the west and the Cascade Range on the east. Inclined or uplifted structural zones within the Willamette Valley/Puget Sound Lowland constitute highland areas and depressed structural zones form sediment-filled basins. The site is located in the northern portion of the Portland/Vancouver

Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide.

According to the *Geologic Map of the Ridgefield Quadrangle, Clark County, Washington* (R.C. Evarts, Washington Division of Geology and Earth Resources, Scientific Investigations Map 2844, 2004), near surface soils are expected to consist of Miocene-aged, poorly indurated, sedimentary sandstone, siltstone, claystone, and pebbly conglomerate deposits associated with the Sandy River Mudstone formation (Tsr), Pliocene and (or) Miocene aged, semi-consolidated, massive, poorly to well-sorted, pebble and cobble conglomerate with sparse lenses of friable sandstone associated with the Troutdale formation (Ttf), Pleistocene and (or) Pliocene aged, semi-consolidated, poorly sorted to moderately well-sorted, pebble and gravel cobble that is clast supported and commonly imbricated (QTc), and upper-Pleistocene, fine-grained facies consisting of unconsolidated clay, silt, and fine to medium sand, typically inconspicuously bedded, slack-water deposits derived from catastrophic outburst floods of glacial lake Missoula (Qfs).

The *Web Soil Survey* (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2017 Website) identifies surface soils primarily as Hillsboro silt loam with minor areas along the southeast and northwest corners mapped as Gee silt loam, a minor area in the northeast corner as Hesson gravelly clay loam, and a minor area in the southeast corner as Odne silt loam.

Although soil conditions may vary from the broad USDA descriptions, Gee soils generally consist of moisture sensitive fine-textured silts and clays with very low permeability, moderate to high water capacity, moderate shrink-swell potential, and low shear strength. Hillsboro soils generally consist of fine-textured sands, silts, and clays with low permeability, high water capacity, low to moderate shrink-swell potential, and low shear strength. Odne soils generally consist of fine-textured sands, silts, and clays with low permeability, moderate to high water capacity, and low shear strength. Onde soils are generally moisture sensitive and moderately compressible. Hesson soils are generally fine-textured, well drained soils with moderately slow permeability and moderate shrink-swell potential. All identified surface soils exhibit a slight erosion hazard based primarily on slope grade.

3.0 REGIONAL SEISMOLOGY

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

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

Portland Hills Fault Zone

The Portland Hills Fault Zone consists of several northwest-trending faults located along the northeastern margin of the Tualatin Mountains, also known as the Portland Hills, and the southwest margin of the Portland Basin. The fault zone is approximately 25 to 30 miles in length and is located approximately 20 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 the fault zone forms the southwest margin of the Tualatin basin. Possible late-Quaternary geomorphic surface deformation may exist along the structural zone (Geomatrix Consultants, 1995).

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

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

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

The northwest-trending Lacamas Lake Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 20 miles southeast of the site, and form part of the northeastern margin of the Portland basin. According to *Geology and Groundwater Conditions of Clark County Washington* (USGS Water Supply Paper 1600, Mundorff, 1964) and the *Geologic Map of the Lake Oswego Quadrangle* (Oregon DOGAMI Series GMS-59, 1989), the Lacamas Lake fault zone consists of shear contact between the Troutdale Formation and underlying Oligocene andesite-basalt bedrock. Secondary shear contact associated with the fault zone may have produced a series of prominent northwest-southeast geomorphic lineaments in proximity to the site.

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

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

<u>Cascadia Subduction Zone</u>

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

4.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance and 11 test pits (TP-1 through TP-11) was conducted at the site on September 8, 2017. Test pits were explored with a track-mounted excavator. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications. Disturbed soil samples were collected from relevant soil horizons and submitted for laboratory analysis. Analytical laboratory test results are presented in Appendix A. Exploration locations are indicated on

Figure 2. Subsurface exploration logs are presented in Appendix B. Soil descriptions and classification information are provided in Appendix C. A photo log is presented in Appendix E.

4.1 Surface Investigation and Site Description

The site consists of tax parcels 258901000, 258922000, 258971000, 258972000, and 258919000 totaling approximately 42.03 acres. The site is bounded by rural residential development to the north, and west, NE North Fork Avenue to the east, residential development and undeveloped acreage to the south. The site is primarily open and covered with grass.

Existing residential structures were observed throughout the site; one with a detached outbuilding in the southeast corner, accessed via gravel driveway from NE North Fork Avenue; one in the north-to-northwest corner with detached outbuildings, accessed from NE 348th Street; one with an attached garage in the central-south area of the site, accessed from NE 348th Street via an overgrown gravel driveway; one in the south west corner of the site with a detached outbuilding, accessed via NW 348th Street.

A natural drainage feature trends south through the western side of the site and west through a portion of the southern side of the site. Field observations and review of site topographic mapping indicate that grades of approximately 7 to 50 percent characterize the gently to steep sloping site with steeper grades associated with the drainage feature mapped on the western side of the site. Development is proposed in the gently to moderately sloping, grass-covered portion of the site. Elevations range from approximately 200 feet above mean sea level (amsl) in the southwestern corner of the site to 350 feet amsl in the northeastern corner of the site.

4.2 Subsurface Exploration and Investigation

Test pit explorations TP-1 through TP-11 were advanced at the site to a maximum depth of 15 feet below ground surface (bgs). Exploration locations were selected to observe subsurface soil characteristics in proximity to proposed development areas and are indicated on Figure 2.

4.2.1 Soil Type Description

The field investigation indicated the site is generally covered with approximately 8 to 10-inches of sod and topsoil in the observed locations. Underlying topsoil, subsurface soils resembling native USDA Hillsboro silt loam, Gee silt loam, and Hesson gravelly clay loam were encountered. Subsurface lithology was reasonably consistent at all explored locations and may generally be described by soil types identified in the following text.

Soil Type 1 – Lean Clay with Sand

Soil Type 1 was observed to primarily consist of brown with slight orange and grey mottling, dry to moist, medium stiff, lean CLAY with sand. Soil Type 1 was observed below the topsoil layer in test pits TP-1 through TP-9 and beneath Soil Type 3 in test pits TP-9 and TP-11 and extended to observed depths of 5 to 12 feet below ground surface.

Analytical laboratory testing conducted upon a representative soil sample obtained from test pit TP-1 indicated approximately 78 percent by weight passing the No. 200 sieve and in situ moisture content of 17 percent. Atterberg Limits analysis indicated a liquid limit of 39 percent and a plasticity index of 20 percent. The laboratory tested sample of Soil Type 1 is classified CL according to USCS specifications and A-6(15) according to AASHTO specifications.

Soil Type 2 – Fat Clay to Fat Clay with Sand

Soil Type 2 was observed to primarily consist of brown with orange/grey mottling, moist, medium stiff, fat CLAY to fat CLAY with sand. Trace gravel was occasionally observed within the soil unit. Soil Type 2 was encountered below Soil Type 1 in test pits TP-1 through TP-9 and beneath Soil Type 3 in test pit TP-11 and extended to observed depths ranging from 2 feet below ground surface to the maximum depth of exploration.

Analytical laboratory testing conducted upon representative soil samples obtained from test pits TP-1 through TP-3 indicate approximately 82 to 87 percent by weight passing the No. 200 sieve and in situ moisture contents ranging from 23 to 31 percent. Atterberg Limits analysis indicated a liquid limit ranging from 50 to 62 percent and a plasticity index ranging from 33 to 44 percent. The laboratory tested samples of Soil Type 2 are classified CH according to USCS specifications and A-7-6(30), A-7-6(36), and A-7-6(40) according to AASHTO specifications.

Soil Type 3 – Clayey Gravel with Sand and Cobbles

Soil Type 3 was observed to primarily consist of brown, moist, medium dense, clayey GRAVEL with sand and cobbles. Soil Type 3 was encountered below Soil Type 2 in test pits TP-4 through TP-6 and TP-8, beneath Soil Type 1 in test pit TP-9, and beneath the topsoil layer in test pits TP-10 and TP-11 and extended to observed depths ranging from 2 feet below ground surface to the maximum depth of exploration.

Analytical laboratory testing conducted upon a representative soil sample obtained from test pit TP-4 indicated approximately 21 percent by weight passing the No. 200 sieve and in situ moisture content of 27 percent. Atterberg Limits analysis indicated a liquid limit of 44 percent and a plasticity index of 18 percent. The laboratory tested sample of Soil Type 3 is classified GC according to USCS specifications and A-2-7(0) according to AASHTO specifications.

Soil Type 4 – Clayey Sand

Soil Type 4 was observed to primarily consist of golden brown, moist, medium dense, clayey SAND. Soil Type 4 was observed below Soil Type 3 in test pit TP-10 and extended to the maximum depth of exploration.

4.2.2 Groundwater

Groundwater was not encountered within test pit explorations TP-1 through TP-11 to the maximum explored depth. Review of nearby well logs obtained from the State of Washington Department of Ecology indicates that static groundwater levels throughout the site range from 18 to 79 feet below ground surface. Variations in ground water elevations likely reflect the changes in ground surface elevation, screened interval depth of these wells, and the presence of multiple aquifers and confining units.

the changes in ground surface elevation, screened interval depth of these wells, and the presence of multiple aquifers and confining units.

Groundwater levels are often subject to seasonal variance and may rise during extended periods of increased precipitation. Perched groundwater may also be present in localized areas. Seeps and springs may become evident during site grading, primarily along slopes or in areas cut below existing grade. Structures, roads, and drainage design should be planned accordingly.

5.0 GEOLOGIC HAZARD AREAS ASSESSMENT

City of La Center Municipal Code, Section 18.300 defines geologic hazard requirements for proposed development in areas subject to City of La Center jurisdiction. Three potential geologic hazards are identified: (1) erosion hazard areas, (2) slope and landslide hazard areas, and (3) seismic hazard areas.

Columbia West conducted geologic hazard review to assess whether these hazards are present at the subject property proposed for development, and if so, to provide appropriate development recommendations. The geologic hazard review was based upon physical and visual reconnaissance, subsurface exploration, laboratory analysis of collected soil samples, and review of maps and other published technical literature. The results of the geologic hazard review are discussed in the following sections.

5.1 Erosion Hazard Areas

According to *Clark County Maps Online*, the *Soil Survey of Clark County, Washington,* and field observations, the erosion hazard for site soils ranges from slight to severe depending upon slope grade. Therefore, according to the *City of La Center Municipal Code,* a soil erosion hazard area is present at the site. However, the soil erosion hazard can be successfully mitigated by preparation and adherence to a site-specific erosion control plan that identifies BMPs to reduce potential impacts on site soils during construction. Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Erosion control measures are discussed further in Section 6.12, *Erosion Control Measures*.

5.2 Slope and Landslide Hazard Areas

According to *City of La Center Municipal Code,* critical areas associated with slopes and landslide hazards are defined respectively as slopes with gradients meeting or exceeding 25 percent and areas subject to risk of mass movement due to a combination of geologic, topographic, and hydrologic factors.

Columbia West conducted review of available mapping, *Clark County GIS* data, and site reconnaissance to evaluate the potential presence of critical areas associated with slopes and landslide hazards on or near the subject site.

5.2.1 Geologic Literature Review

Columbia West reviewed *Slope Stability, Clark County, Washington* (Fiksdal, 1975) to assess site slope characteristics. The Fiksdal 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 (1) stable areas – no slides or unstable slopes.

Columbia West also reviewed the *Geologic Map of the Ridgefield Quadrangle, Clark County, Washington* (R.C. Evarts, Washington Division of Geology and Earth Resources, Scientific Investigations Map 2844, 2004), which indicates that no landslide deposits are mapped at the subject site or in the surrounding vicinity.

5.2.2 Slope Reconnaissance

To observe geomorphic conditions, Columbia West conducted visual and physical reconnaissance of drainage ravine slopes on the property. As previously described, test pit explorations conducted near drainage ravine slopes indicated the presence of fine-textured clay and sand underlain by a dense gravel formation (apparent sedimentary conglomerate). Groundwater was not observed within test pit explorations and soil horizons appeared firm and well developed. No landslide debris was observed within subsurface soils explored near the slopes.

Review of topographic mapping indicates that vertical slope heights for western drainage ravine slopes, as measured from toe to top-of-slope break, vary from approximately 25 to 50 feet with slope grades generally ranging from 30 to 55 percent. Southern site slopes were observed to be approximately 10 to 20 feet in total height with slope grades ranging from 10 to 25 percent. Slopes currently support vegetation consisting of deciduous and conifer trees, blackberry vines, grasses, and shrubs. Slopes are generally planar with no observed evidence of instability. There was no observed direct evidence of large-scale, mass slope movements or historic landslides.

5.2.3 Slope Stability Assessment

Based upon the results of literature review, subsurface exploration, and field reconnaissance, Columbia West did not observe a combination of geologic, topographic, or hydrologic features suggesting significant risk of mass slope movement. However, slope gradients along western and southern drainage ravines meet or exceed 25 percent in several locations and meet the definition of a critical area according to *City of La Center Municipal Code*. The location of the critical areas are indicated on Figure 2. Site development near the critical areas may be successfully achieved by following the engineering and planning recommendations presented in this report and by maintaining appropriate geotechnical buffers from the top-of-slope as presented in the following text sections.

5.2.4 Geotechnical Buffer

To reduce the risk of adverse impacts to slope stability within and near the critical areas, residential structures, stormwater facilities, and structural fill placement should be avoided within the geotechnical buffer identified on Figure 2, unless a case-by-case assessment as described in Section 5.2.6 is conducted. The geotechnical buffer alignment is based upon slope reconnaissance and slope stability assessment described above, and is defined by a 50-foot setback measured from the existing top of slope.

The buffer recommendations described above are intended to reduce potential for slope instability by limiting locations for large dynamic and static loads derived from earthwork, residential structures, retaining walls, roadways, stormwater facilities, and other significant developments.

5.2.5 Grading Recommendations within the Geotechnical Buffer

The geotechnical buffer is intended to minimize adverse impacts to slope stability due to dynamic and static loading. Placement of engineered structural fill or stockpiles of disturbed soil should be avoided inside the geotechnical buffer without case-by-case evaluation per Section 5.2.6, *Potential Encroachment within the Geotechnical Buffer*. Soil excavation may be acceptable within the buffer, as driving forces may be reduced by removing soil mass. Columbia West should review mass grading plans as they relate to the geotechnical buffer.

Areas within the geotechnical buffer are not intended to be do-not-disturb conservation areas. Small disturbances such as minor landscaping, fence building, or walk-path construction 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 within the buffer should be minimized. The text herein pertains only to the geotechnical aspect of construction within the recommended geotechnical buffer.

5.2.6 Potential Encroachment within the Geotechnical Buffer

Encroachment of some site improvements or structural facilities inside the geotechnical buffer 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 geotechnical buffer 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.

5.3 Seismic Hazard Areas

Seismic hazards 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.

5.3.1 Soil Liquefaction and Dynamic Settlement

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, lateral spreading, and differential compaction.

Soils most susceptible to liquefaction are recent geologic deposits, such as river and floodplain sediments. These soils are generally saturated, cohesionless, loose to medium dense sands within 50 feet of ground surface. Potentially liquefiable soils located above the existing, historic, or expected ground water levels do not generally pose a liquefaction hazard. It is important to note that changes in perched ground water elevation may occur due to project development or other factors not observed at the time of investigation.

According to *the Liquefaction Susceptibility Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), the site is mapped as very low susceptibility for liquefaction. Based upon the results of subsurface exploration and laboratory analysis, observed soils were generally consistent with the hazard designation mapped for the subject-site. Therefore, the potential for soil liquefaction is considered to be very low.

5.3.2 Ground Shaking Amplification

Review of the *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), indicates that site soils may be represented by Site Class C and Site Class D as defined in *2015 IBC Section 1613.3.2*. A designation of Site Class C or D indicates that minor amplification of seismic energy may occur during a seismic event due to subsurface conditions. However, this is typical for many areas within Clark County, does not constitute a geologic hazard in Columbia West's opinion, and will not prohibit development if properly accounted for during the design process.

5.3.3 Fault Rupture

Because there are no known geologic seismic faults within the site boundaries, fault rupture is unlikely.

6.0 DESIGN RECOMMENDATIONS

The geotechnical site investigation suggests the proposed development is generally compatible with surface and subsurface soils, provided the recommendations presented in this report are utilized and incorporated into the design and construction processes. The primary geotechnical concerns associated with the site are fine-textured soils and drainage. Design recommendations are presented in the following text sections.

6.1 Site Preparation and Grading

Vegetation, organic material, unsuitable fill, and deleterious material that may be encountered should be cleared from areas identified for structures and site grading. Vegetation, other organic material, and debris should be removed from the site. Stripped topsoil should also be removed, or used only as landscape fill in nonstructural areas with slopes less than 25 percent. The anticipated stripping depth for sod and highly organic topsoil is anticipated to be approximately 8 to 10 inches. The required stripping depth may increase in areas of heavy organics or previously existing structures. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed. The post-construction maximum depth of landscape fill placed or spread at any location onsite should not exceed one foot.

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

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

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 in loose lifts not exceeding 12 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within two percentage points of optimum conditions. A field density at least equal to 95 percent of the maximum dry density, obtained from the standard Proctor moisture-density relationship test (ASTM D698), is recommended for structural fill placement. 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 Columbia West.

Engineered structural fill placement activities should be performed during dry summer months if possible. Most clean native soils may be suitable for use as structural fill if adequately 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. Native clay soils with a plasticity index greater than 25 (Soil Type 2) should be evaluated and approved by Columbia West prior to re-use as structural fill. Because they are moisture-sensitive, fine-textured soils are often difficult to excavate and compact during wet weather conditions. If adequate compaction is not achievable with clean native soils, import structural fill consisting of well-graded granular material with a maximum particle size of three inches and no more than five percent passing the No. 200 sieve is recommended.

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

6.3 Cut and Fill Slopes

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

Final cut or fill slopes at the site should not exceed 2H:1V or 20 feet in 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 and documented by Columbia West.

6.4 Foundations

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

The existing ground surface should be prepared as described in Section 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 modulus of subgrade reaction is estimated to be 100 psi/inch. The estimated coefficient of friction between in situ compacted native soil or engineered structural fill and in-place poured concrete is 0.45. Lateral forces may also be resisted by an assumed passive soil equivalent fluid pressure of 250 psf/f against embedded footings. The upper six inches of soil should be neglected in passive pressure calculations.

Footings should extend to a depth at least 18 inches below lowest adjacent grade to provide adequate bearing capacity and protection against frost heave. Foundations constructed during wet weather conditions 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 Settlement

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

6.6 Excavation

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

Based upon laboratory analysis and field testing, near-surface soils may be Washington State Industrial Safety and Health Administration (WISHA) Type C. For temporary open-cut excavations deeper than four feet, but less than 20 feet in soils of these types, the maximum allowable slope is 1.5H:1V. WISHA soil type should be confirmed during field construction activities by the contractor. Soil is often anisotropic and heterogeneous, and it is possible that WISHA soil types determined in the field may differ from those described above.

The contractor should be held responsible for site safety, sloping, and shoring. Columbia West is not responsible for contractor activities and in no case should excavation be conducted in excess of all applicable local, state, and federal laws.

6.7 Lateral Earth Pressure

If retaining walls or below-grade structures are proposed, lateral earth pressures should be carefully considered in the design process. Hydrostatic pressure and additional surcharge loading should also be considered. Retained material may include engineered structural backfill or existing embankment fill. Structural wall backfill should consist of imported granular material meeting *Section 9-03.12(2)* of WSDOT Standard Specifications. Backfill should be prepared and compacted to at least 95 percent of maximum dry density as determined by the modified Proctor test (ASTM D1557). Recommended parameters for lateral earth pressures for in situ soils and engineered structural backfill consisting of imported granular fill meeting WSDOT specifications for *Gravel Backfill for Walls 9-03.12(2)* are presented in Table 1.

The design parameters presented in Table 1 are valid for static loading cases only and are based upon in situ existing soils or compacted granular fill. The recommended earth pressures do not include surcharge loads, dynamic loading, hydrostatic pressure, or seismic design.

Table 1. Lateral Earth Pressure Parameters for Level Backfill

Retained Soil	Equivalent Fluid Pressure for Level Backfill			Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive		
Undisturbed native lean clay (Soil Type 1)	60 pcf	41 pcf	293 pcf	110 pcf	27°
Undisturbed native fat clay with sand (Soil Type 2)	66 pcf	48 pcf	231 pcf	105 pcf	22°
Undisturbed native clayey gravel with sand and cobbles (Soil Type 3)	55 pcf	35 pcf	442 pcf	125 pcf	34°
Undisturbed native clayey sand (Soil Type 4)	56 pcf	37 pcf	391 pcf	120 pcf	32°
Approved Structural Backfill Material WSDOT 9-03.12(2) compacted aggregate backfill	52 pcf	32 pcf	568 pcf	135 pcf	38°

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

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

A continuous one-foot-thick zone of free-draining, washed, open-graded 1-inch by 2-inch drain rock and a 4-inch perforated gravity drain pipe is assumed behind retaining walls. Geotextile filter fabric should be placed between the drain rock and backfill soil. Specifications for drainpipe design are presented in Section 6.9, *Drainage*. If walls cannot be gravity drained, saturated base conditions and/or applicable hydrostatic pressures should be assumed.

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

6.8 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 2.

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

	2% Probability of Exceedance in 50 yrs
Peak Ground Acceleration	0.38 g
0.2 sec Spectral Acceleration	0.89 g
1.0 sec Spectral Acceleration	0.39 g

The listed probabilistic ground motion values are based upon "firm rock" sites with an assumed shear wave velocity of 2,500 ft/s in the upper 100 feet of soil profile. These values should be adjusted for site class effects by applying site coefficients Fa and Fv as defined in 2015 IBC Tables 1613.3.3(1) and (2); the PGA should be adjusted by applying the site coefficient F_{PGA} as defined by *ASCE 7, Chapter 11, Table 11.8-1*. The site coefficients are intended to more accurately characterize estimated peak ground and respective earthquake spectral response accelerations by considering site-specific soil characteristics and index properties.

The *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), indicates site soils may be represented by Site Class C and Site Class D in *2015 IBC Section 1613.3.2*. Based upon site-specific testing, site soils may be considered to be Site Class D. This site class designation indicates that some amplification of seismic energy may occur during a seismic event because of subsurface conditions.

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

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

6.9 Drainage

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

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

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

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

6.10 Bituminous Asphalt and Portland Cement Concrete

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

For dry weather construction, pavement surface sections should bear upon competent subgrade consisting of scarified and compacted native soil or engineered structural fill. Wet weather pavement construction is discussed in Section 6.11, *Wet Weather Construction Methods and Techniques*. Subgrade conditions should be evaluated and tested by Columbia West prior to placement of crushed aggregate base. Subgrade evaluation should include nuclear gauge density testing and wheel proof-roll observations conducted with a 12-cubic yard, double-axle dump truck or equivalent. Nuclear gauge density testing should be conducted at 150-foot intervals or as determined by the onsite geotechnical engineer. Subgrade soil should be compacted to at least 95 percent of the modified Proctor dry density, as determined by ASTM D1557. Areas of observed deflection or rutting during proof-roll evaluation should be excavated to a firm surface and replaced with compacted crushed aggregate.

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

Portland cement concrete curbs and sidewalks should be installed in accordance with City of La Center, Washington specifications. Curb and sidewalk aggregate base should be observed and proof-rolled by Columbia West. Soft areas that deflect or rut should be stabilized prior to pouring concrete. Concrete should be tested during installation in accordance with ASTM C171, C138, C231, C143, C1064, and C31. This includes casting of cylinder specimen at a frequency of four cylinders per 100 cubic yards of poured concrete. Recommended field and analytical laboratory concrete testing includes slump, air entrainment, temperature, and unit weight.

6.11 Wet Weather Construction Methods and Techniques

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

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

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

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

Crushed aggregate base should be compacted to at least 95 percent of maximum dry density according to the modified Proctor density test (ASTM D1557). Compaction should be verified by nuclear gauge density testing. Observation of a proof-roll with a loaded dump truck is also recommended as an indication of the compacted aggregate's performance.

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

6.12 Erosion Control Measures

According to *Clark County Maps Online (http://gis.clark.wa.gov/ccgis/mol/property.htm)* portions of the site are mapped as containing severe erosion hazard areas. The *Soil Survey of Clark County, Washington* also indicates potential erosion hazards for site soils. As previously discussed, near-surface soils generally consisted of fine-textured silts, sands, and clays at the locations explored.

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

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

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

6.13 Soil Shrink/Swell Potential

Based upon laboratory analysis, near-surface soils contain as much as 87 percent by weight passing the No. 200 sieve and exhibit a plasticity index ranging from 20 to 44 percent. This indicates the potential for soil shrinking or swelling and underscores the importance of proper moisture conditioning during fill placement. Medium to high plasticity soils should be placed and compacted at a moisture content approximately two percent above optimum as determined by laboratory analysis.

6.14 Utility Installation

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

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

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

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

Sincerely,

COLUMBIA WEST ENGINEERING, Inc.

Lance V. Lehto, PE, GE
President

LANCE V. LEHTO
STATE OF WASHINGTON
36301
REGISTERED
PROFESSIONAL ENGINEER

10-20-17

REFERENCES

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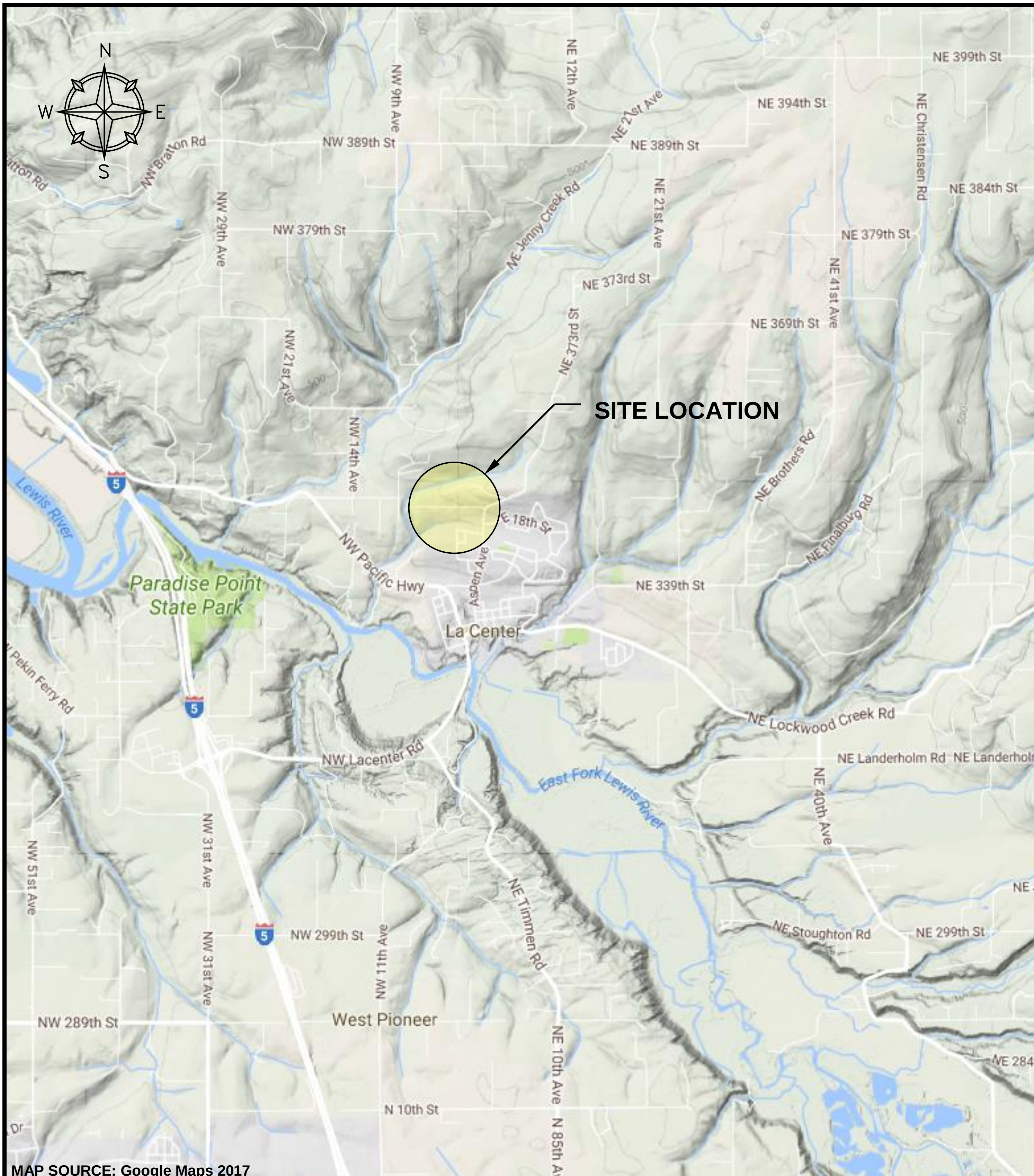
FIGURES



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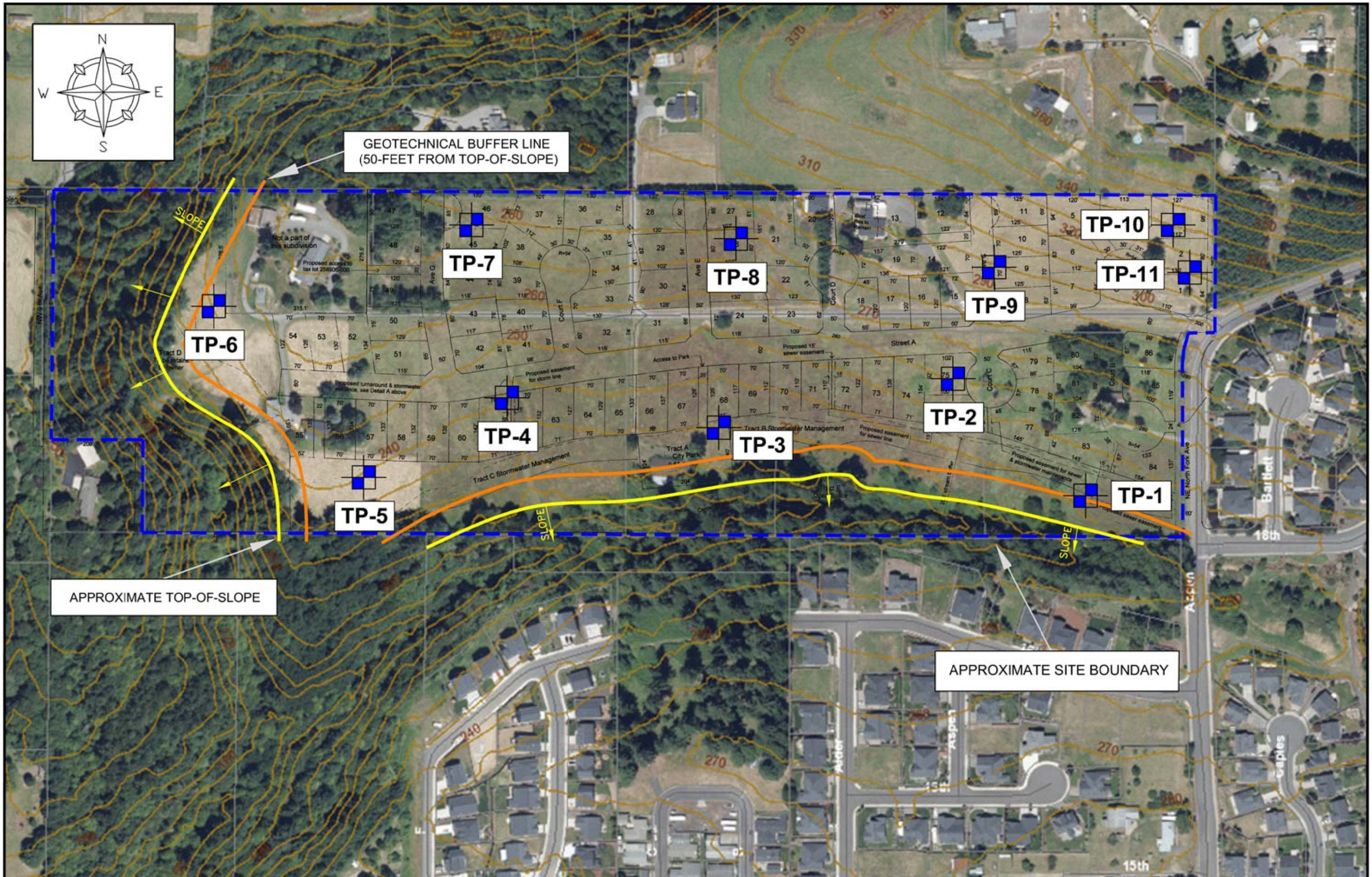
Design	Drawn: CWS			
Checked: GLW	Date: 10/13/2017			
Client: STEPHENS–	Rev	By	Date	
Job No.: 17248				
CAD File: FIGURE 1				
Scale: ~1:50,000				

SITE LOCATION MAP

STEPHENS PROPERTY
LA CENTER, WASHINGTON

FIGURE
1



NOTES:
1. SITE LOCATION: 34700 NORTHEAST NORTH FORK AVENUE, LA CENTER, WASHINGTON.
2. SITE CONSISTS OF PARCELS 258901000, 258919000, 258922000, 258971000, 258972000 TOTALLING APPROXIMATELY 42.03 ACRES.
3. DRAWING IS NOT TO SCALE.
4. BASE MAPS OBTAINED FROM CLARK COUNTY, WASHINGTON GIS AND THE WOLFE GROUP, LLC.
5. SOIL EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
6. TEST PITS LOOSELY BACKFILLED WITH ONSITE SOILS 09/08/17.

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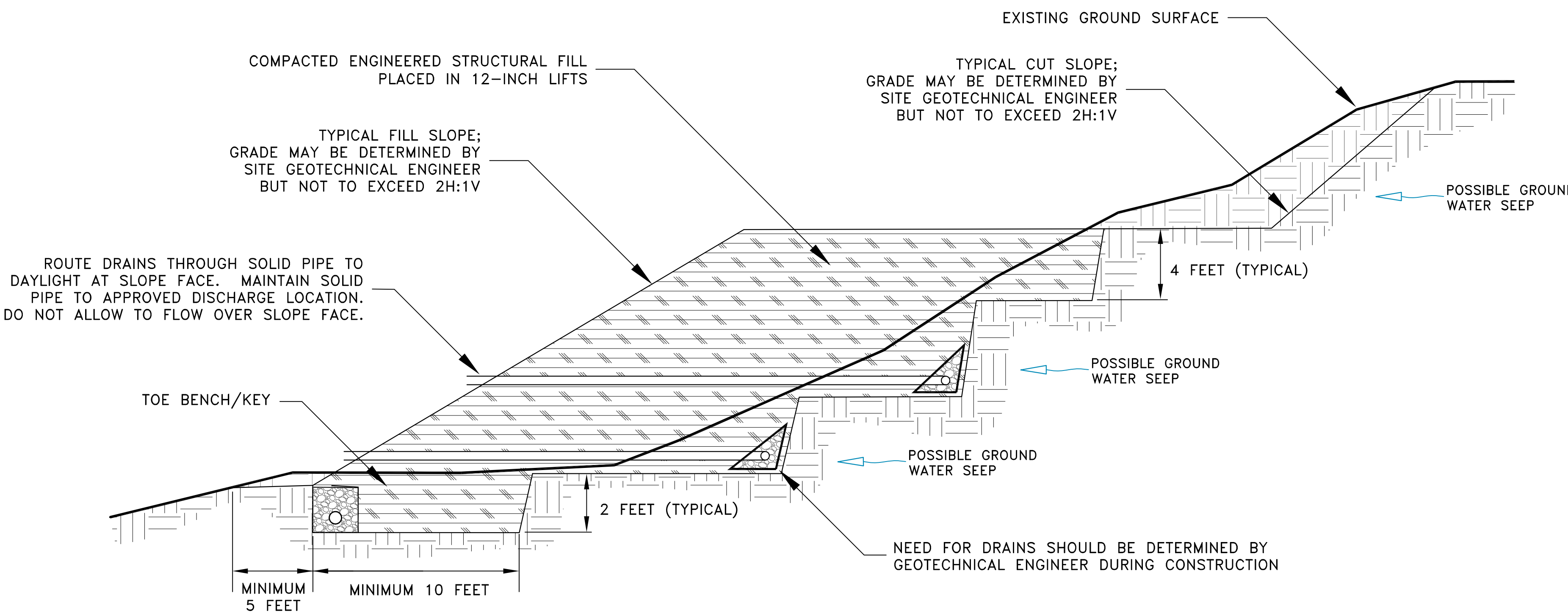
Client: STEPHENS-RERICK	Rev	By	Date
Job No: 17248			
CAD File: FIGURE 2			
Scale: NONE			

EXPLORATION LOCATION MAP

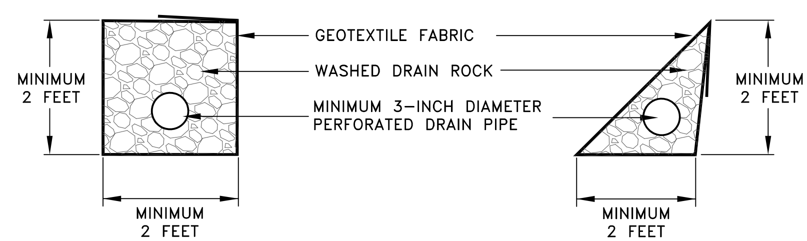
STEPHENS PROPERTY
LA CENTER, WASHINGTON

FIGURE 2

TYPICAL CUT AND FILL SLOPE CROSS-SECTION



TYPICAL DRAIN SECTION DETAIL



DRAIN SPECIFICATIONS

GEOTEXTILE FABRIC SHALL CONSIST OF MIRAFI 140N OR APPROVED EQUIVALENT WITH AOS BETWEEN No. 70 AND No. 100 SIEVE.

WASHED DRAIN ROCK SHALL BE OPEN-GRADED ANGULAR DRAIN ROCK WITH LESS THAN 2 PERCENT PASSING THE No. 200 SIEVE AND A MAXIMUM PARTICLE SIZE OF 3 INCHES.

NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FILL AND CUT SLOPE SECTION, AND MAY NOT BE SITE-SPECIFIC.

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Design:	Drawn: CWS			TYPICAL CUT AND FILL SLOPE CROSS-SECTION
Checked: GLW	Date: 09/25/17			
Client: STEPHEHS-RERICK	Rev	By	Date	
Job No: 17248				STEPHENS PROPERTY LA CENTER, WASHINGTON
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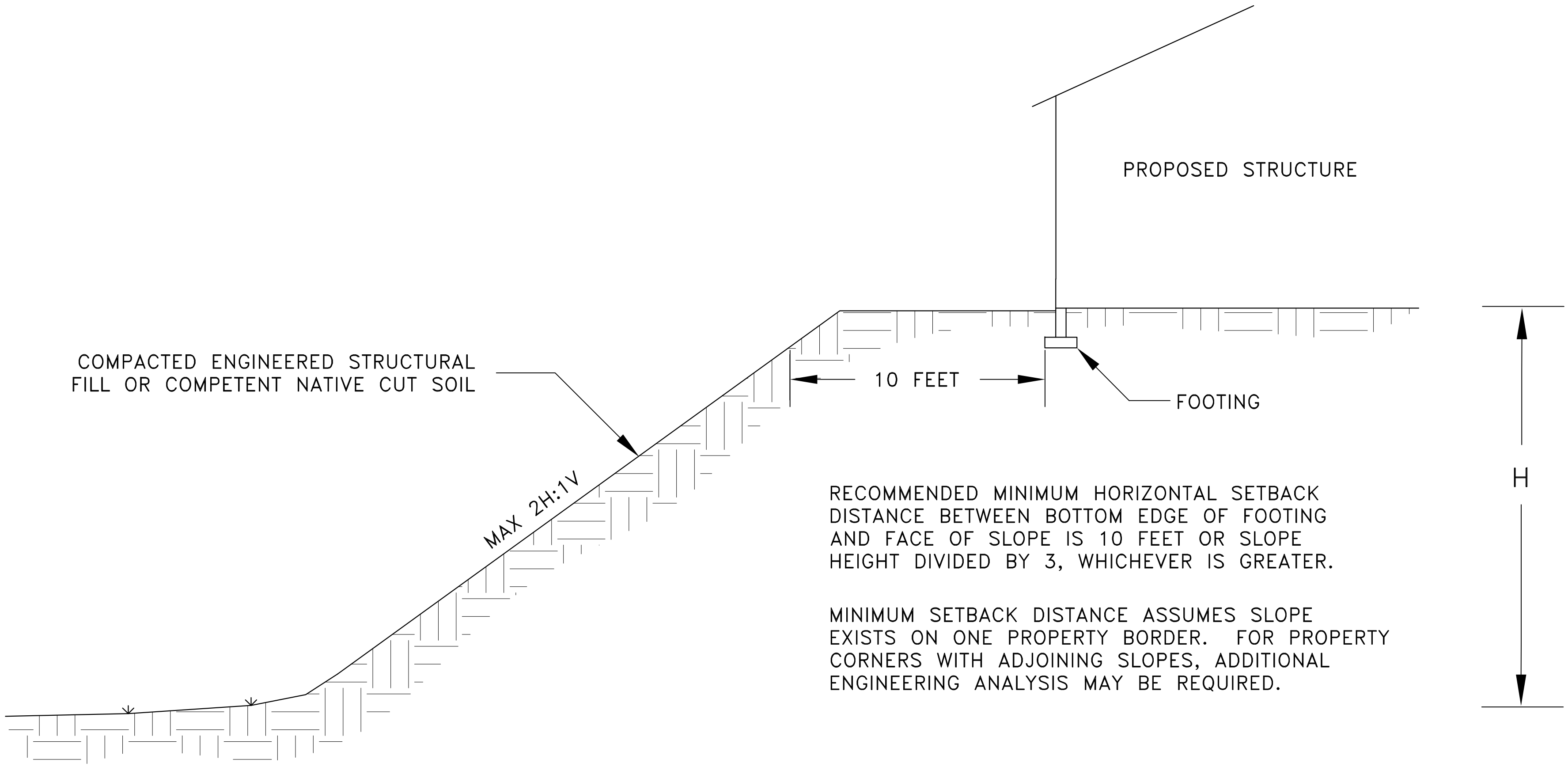
Scale: NONE				

FIGURE 3

MINIMUM FOUNDATION SLOPE SETBACK 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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Design:	Drawn: CWS			MINIMUM FOUNDATION SLOPE SETBACK DETAIL
Checked: GLW	Date: 09/25/17			
Client: STEPHENS-RERICK	Rev	By	Date	
Job No: 17248				STEPHENS PROPERTY LA CENTER, WASHINGTON
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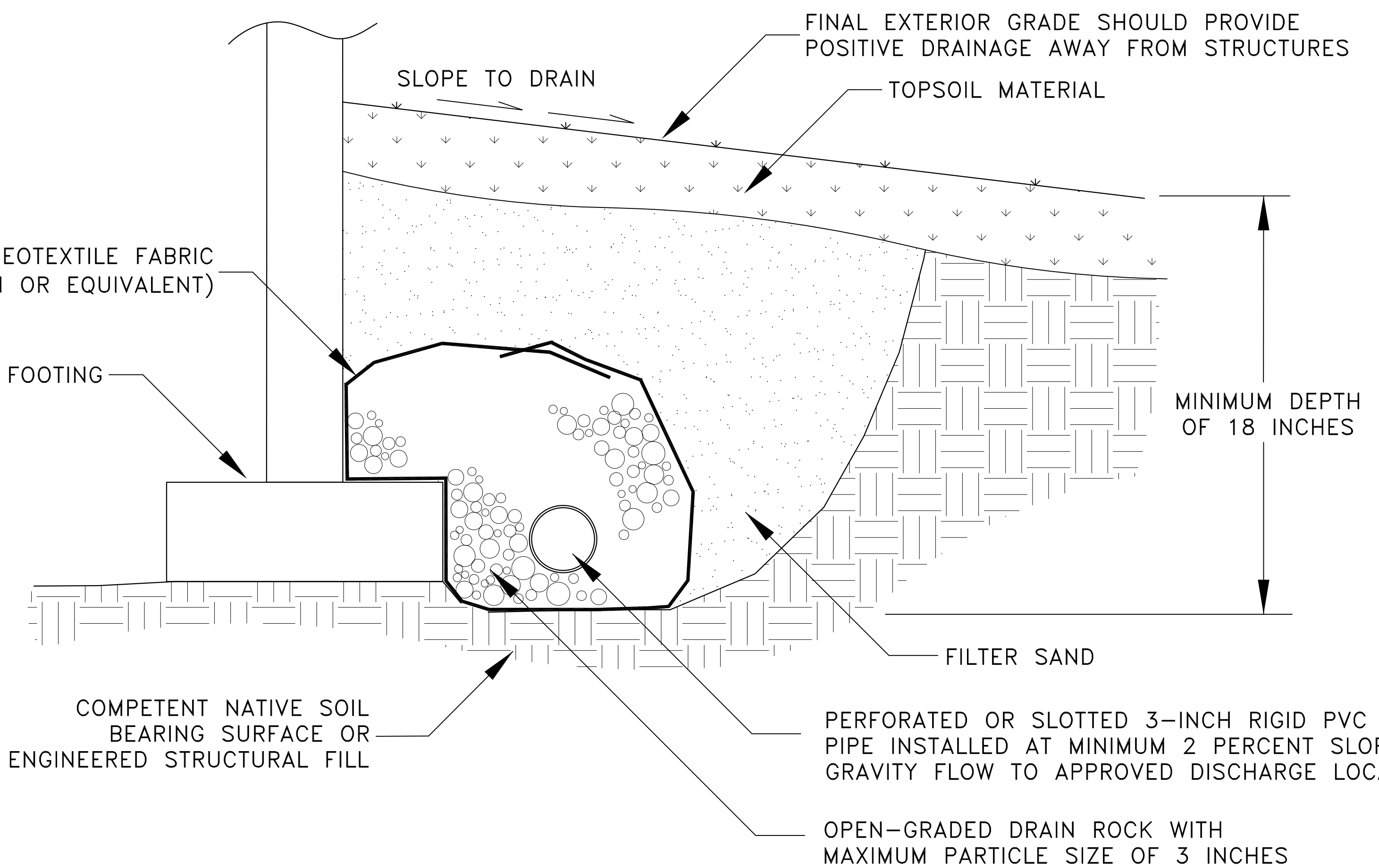
Scale: NONE				

FIGURE 4

TYPICAL PERIMETER FOOTING DRAIN DETAIL



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

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Job No: 17248				
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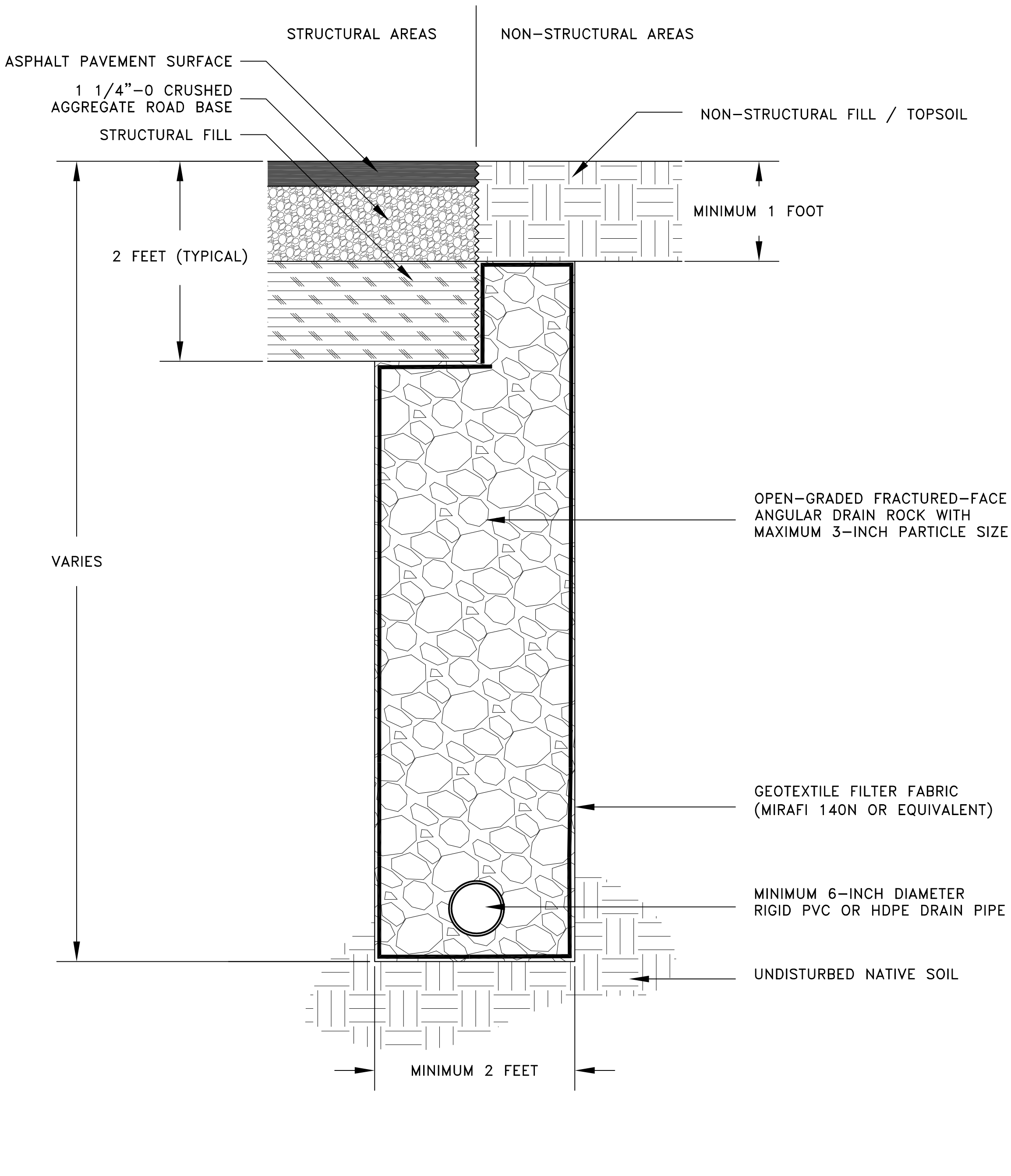
Scale: NONE				

TYPICAL PERIMETER FOOTING DRAIN DETAIL

STEPHENS PROPERTY
LA CENTER, WASHINGTON

FIGURE 5

TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL



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

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Design:	Drawn: CWS		
Checked: GLW	Date: 09/25/17		
Client: STEPHENS-RERICK	Rev	By	Date
Job No: 17248			
CAD File: FIGURE 6			
Scale: NONE			

TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL

STEPHENS PROPERTY
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FIGURE 6

APPENDIX A
LABORATORY TEST RESULTS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-707
		REPORT DATE: 09/27/17	FIELD ID: TP1.1
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-01 depth = 2 feet	CL, Lean Clay with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-6(15)

LABORATORY TEST DATA

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

ADDITIONAL DATA

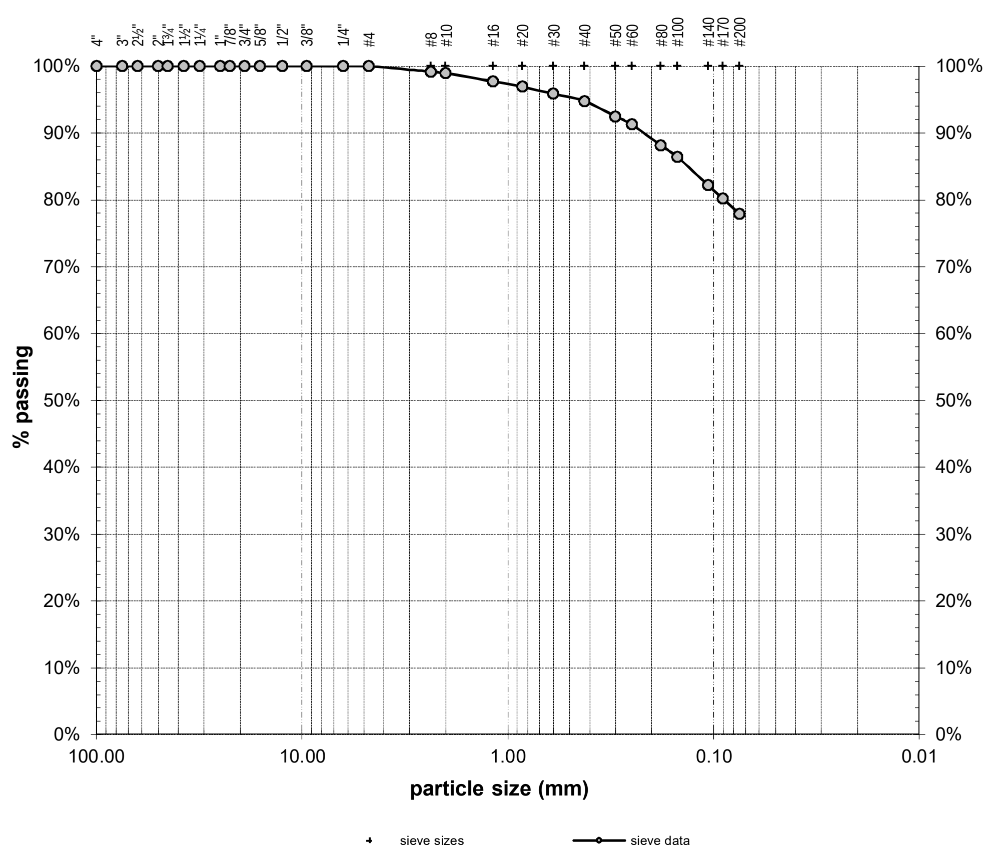
initial dry mass (g) =	467.18		
as-received moisture content =	16.8%	coefficient of curvature, C_C =	n/a
liquid limit =	39	coefficient of uniformity, C_U =	n/a
plastic limit =	19	effective size, $D_{(10)}$ =	n/a
plasticity index =	20	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 22.1%
% silt and clay = 77.9%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		99.2%		
	#10	2.00	99.0%			
	#16	1.18		97.7%		
	#20	0.850	96.9%			
	#30	0.600		95.9%		
	#40	0.425	94.8%			
	#50	0.300		92.5%		
	#60	0.250	91.3%			
	#80	0.180		88.1%		
	#100	0.150	86.4%			
	#140	0.106		82.2%		
	#170	0.090		80.2%		
	#200	0.075	77.9%			

GRAIN SIZE DISTRIBUTION



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-707
		REPORT DATE: 09/27/17	FIELD ID: TP1.1
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit TP-01 depth = 2 feet	CL, Lean Clay with Sand

LABORATORY TEST DATA

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

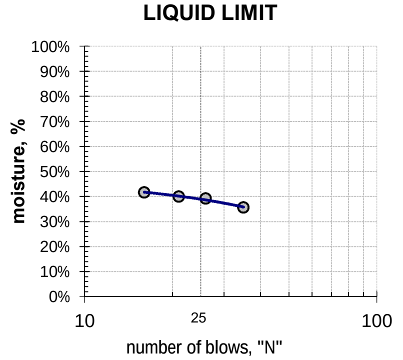
ATTERBERG LIMITS

liquid limit = 39
plastic limit = 19
plasticity index = 20

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.13	35.85	36.80	37.08
dry soil + pan weight, g =	30.63	31.65	32.18	32.31
pan weight, g =	20.78	20.93	20.61	20.84
N (blows) =	35	26	21	16
moisture, % =	35.5 %	39.2 %	39.9 %	41.6 %

LIQUID LIMIT



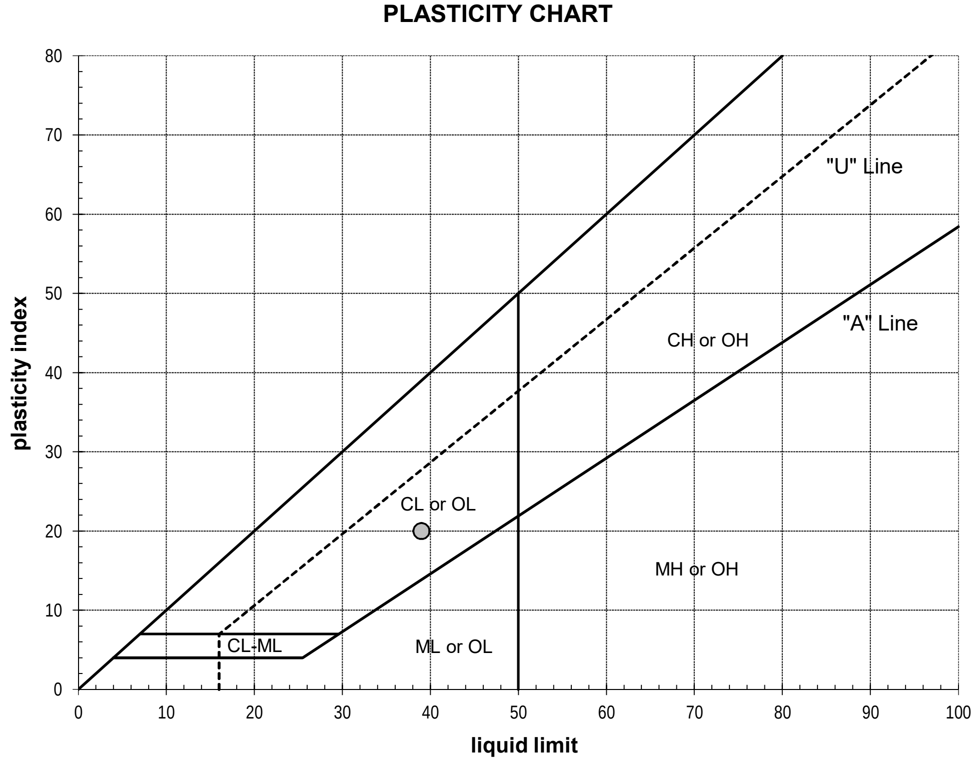
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	29.17	29.01		
dry soil + pan weight, g =	27.83	27.71		
pan weight, g =	20.82	20.85		
moisture, % =	19.1 %	19.0 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 22.1%
% silt and clay = 77.9%
% silt = n/a
% clay = n/a
moisture content = 16.8%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-708
		REPORT DATE: 09/27/17	FIELD ID: TP1.2
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY	Test Pit TP-01 depth = 10 feet	CH, Fat Clay

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(40)

LABORATORY TEST DATA

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

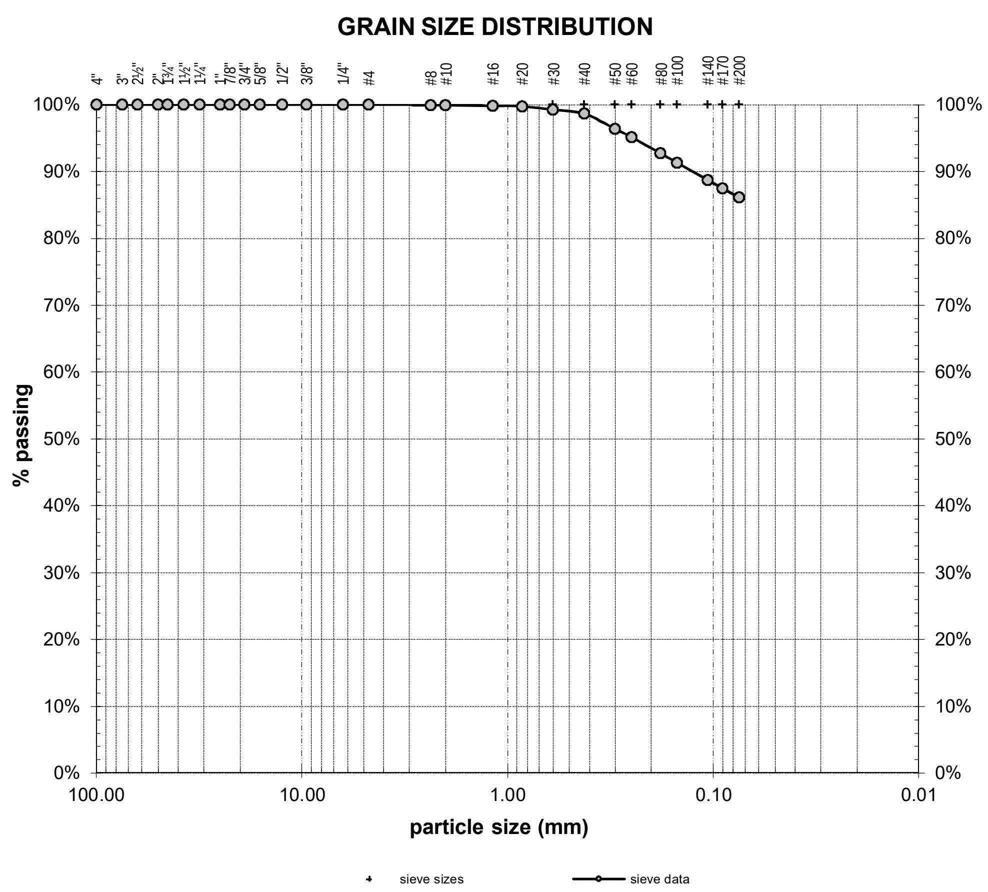
ADDITIONAL DATA

initial dry mass (g) =	290.72		
as-received moisture content =	30.8%	coefficient of curvature, C_C =	n/a
liquid limit =	62	coefficient of uniformity, C_U =	n/a
plastic limit =	18	effective size, $D_{(10)}$ =	n/a
plasticity index =	44	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 13.9%
% silt and clay = 86.1%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		100.0%		
	#10	2.00	99.9%			
	#16	1.18		99.8%		
	#20	0.850	99.8%			
	#30	0.600		99.2%		
	#40	0.425	98.7%			
	#50	0.300		96.4%		
	#60	0.250	95.2%			
	#80	0.180		92.7%		
	#100	0.150	91.3%			
	#140	0.106		88.7%		
	#170	0.090		87.5%		
	#200	0.075	86.1%			



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-708
		REPORT DATE: 09/27/17	FIELD ID: TP1.2
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY	Test Pit TP-01 depth = 10 feet	CH, Fat Clay

LABORATORY TEST DATA

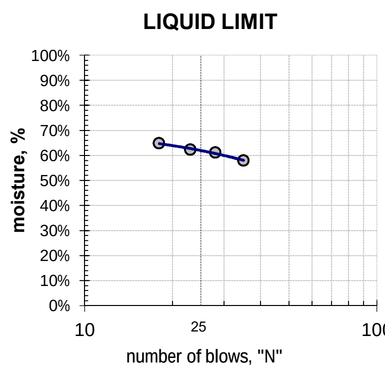
LABORATORY EOUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

ATTERBERG LIMITS

liquid limit =	62
plastic limit =	18
plasticity index =	44

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	36.70	34.61	35.38	34.74
dry soil + pan weight, g =	30.91	29.33	29.80	29.29
pan weight, g =	20.92	20.69	20.84	20.89
N (blows) =	35	28	23	18
moisture, % =	58.0 %	61.1 %	62.3 %	64.9 %

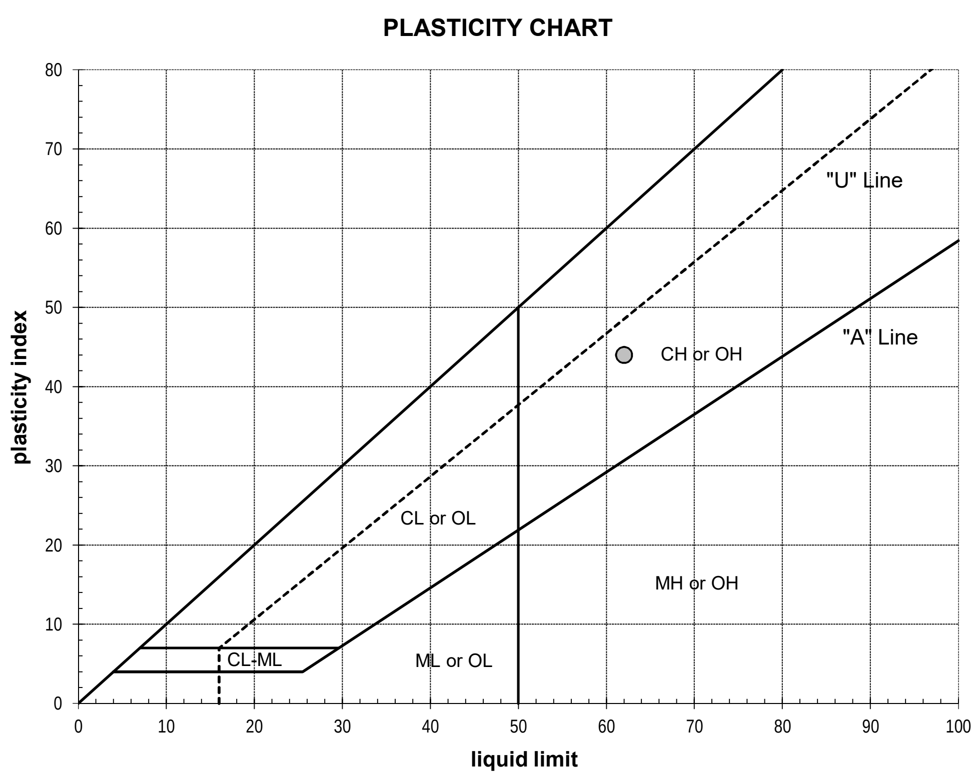


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.02	28.58		
dry soil + pan weight, g =	26.91	27.44		
pan weight, g =	20.70	20.85		
moisture, % =	17.9 %	17.3 %		



ADDITIONAL DATA

% gravel =	0.0%
% sand =	13.9%
% silt and clay =	86.1%
% silt =	n/a
% clay =	n/a
moisture content =	30.8%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-709
		REPORT DATE: 09/27/17	FIELD ID: TP2.1
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY with Sand	Test Pit TP-02 depth = 5 feet	CH, Fat Clay with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(36)

LABORATORY TEST DATA

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

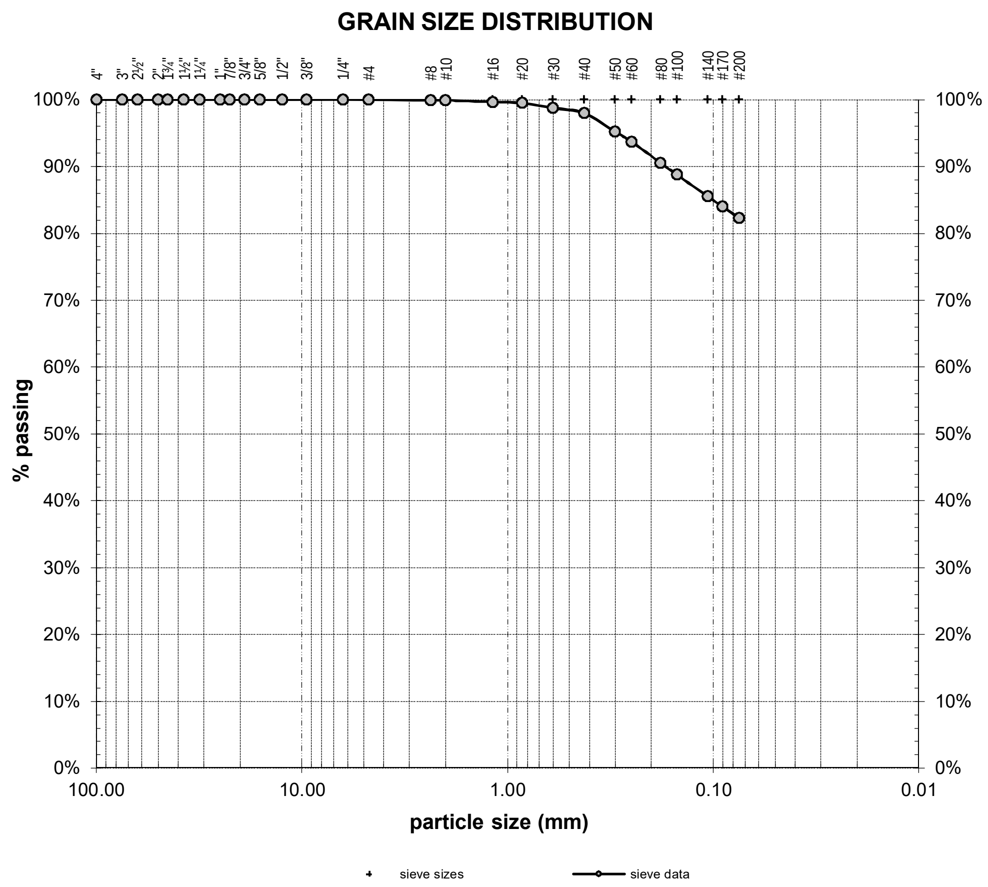
ADDITIONAL DATA

initial dry mass (g) =	461.72		
as-received moisture content =	26.7%	coefficient of curvature, C_C =	n/a
liquid limit =	60	coefficient of uniformity, C_U =	n/a
plastic limit =	17	effective size, $D_{(10)}$ =	n/a
plasticity index =	43	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 17.7%
% silt and clay = 82.3%

	SIEVE SIZE US	mm	PERCENT PASSING SIEVE act.	interp.	SPECS max	min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		99.9%		
	#10	2.00	99.9%			
	#16	1.18		99.7%		
	#20	0.850	99.5%			
	#30	0.600		98.8%		
	#40	0.425	98.0%			
	#50	0.300		95.2%		
	#60	0.250	93.7%			
	#80	0.180		90.5%		
	#100	0.150	88.8%			
	#140	0.106		85.5%		
	#170	0.090		84.0%		
	#200	0.075	82.3%			



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-709
		REPORT DATE: 09/27/17	FIELD ID: TP2.1
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY with Sand	Test Pit TP-02 depth = 5 feet	CH, Fat Clay with Sand

LABORATORY TEST DATA

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

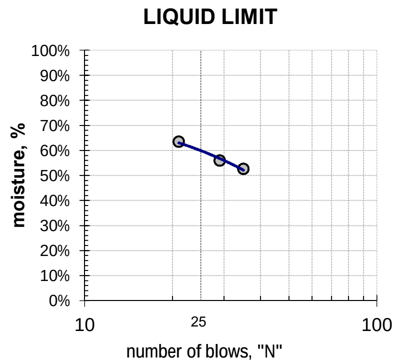
ATTERBERG LIMITS

liquid limit =	60
plastic limit =	17
plasticity index =	43

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.56	34.57	34.68	
dry soil + pan weight, g =	29.81	29.68	29.22	
pan weight, g =	20.78	20.93	20.61	
N (blows) =	35	29	21	
moisture, % =	52.6 %	55.9 %	63.4 %	

LIQUID LIMIT



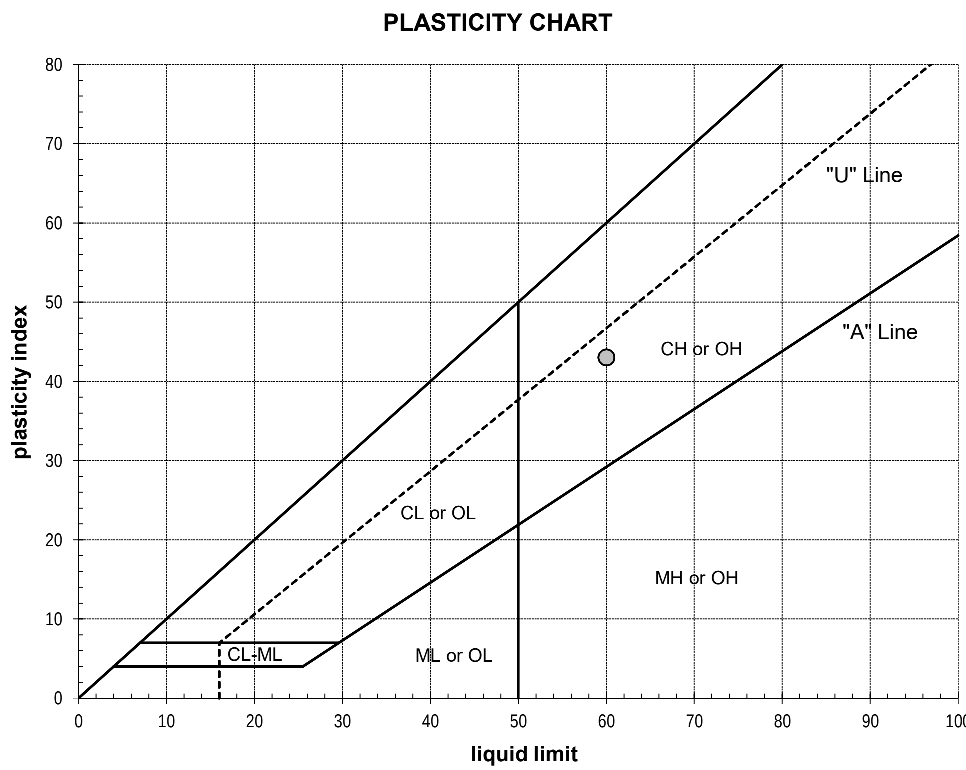
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.46	28.82		
dry soil + pan weight, g =	27.34	27.63		
pan weight, g =	20.82	20.85		
moisture, % =	17.2 %	17.6 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	17.7%
% silt and clay =	82.3%
% silt =	n/a
% clay =	n/a
moisture content =	26.7%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-710
		REPORT DATE: 09/27/17	FIELD ID: TP3.1
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY	Test Pit TP-03 depth = 7.5 feet	CH, Fat Clay

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(30)

LABORATORY TEST DATA

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

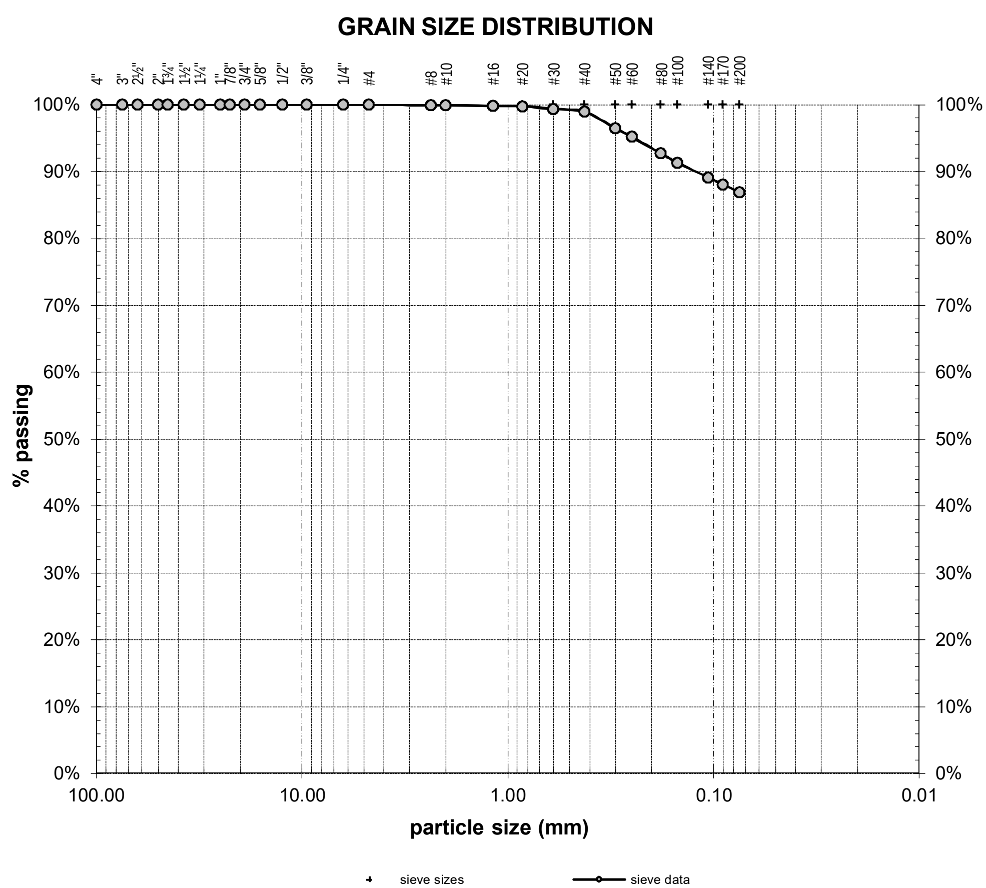
ADDITIONAL DATA

initial dry mass (g) =	478.30		
as-received moisture content =	23.0%	coefficient of curvature, C_C =	n/a
liquid limit =	50	coefficient of uniformity, C_U =	n/a
plastic limit =	17	effective size, $D_{(10)}$ =	n/a
plasticity index =	33	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 13.1%
% silt and clay = 86.9%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		99.9%		
	#10	2.00	99.9%			
	#16	1.18		99.8%		
	#20	0.850	99.8%			
	#30	0.600		99.4%		
	#40	0.425	98.9%			
	#50	0.300		96.5%		
	#60	0.250	95.2%			
	#80	0.180		92.7%		
	#100	0.150	91.3%			
	#140	0.106		89.1%		
	#170	0.090		88.1%		
	#200	0.075	86.9%			



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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-710
		REPORT DATE: 09/27/17	FIELD ID: TP3.1
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY	Test Pit TP-03 depth = 7.5 feet	CH, Fat Clay

LABORATORY TEST DATA

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

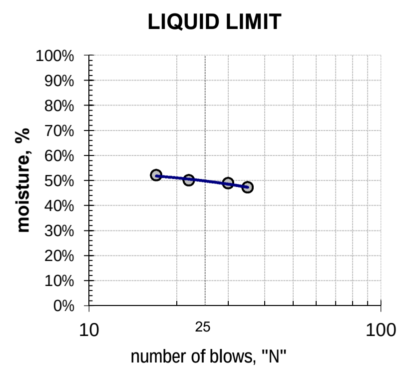
ATTERBERG LIMITS

liquid limit =	50
plastic limit =	17
plasticity index =	33

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	37.76	35.96	35.68	35.78
dry soil + pan weight, g =	32.36	30.95	30.75	30.64
pan weight, g =	20.92	20.69	20.89	20.77
N (blows) =	35	30	22	17
moisture, % =	47.2 %	48.8 %	50.0 %	52.1 %

LIQUID LIMIT



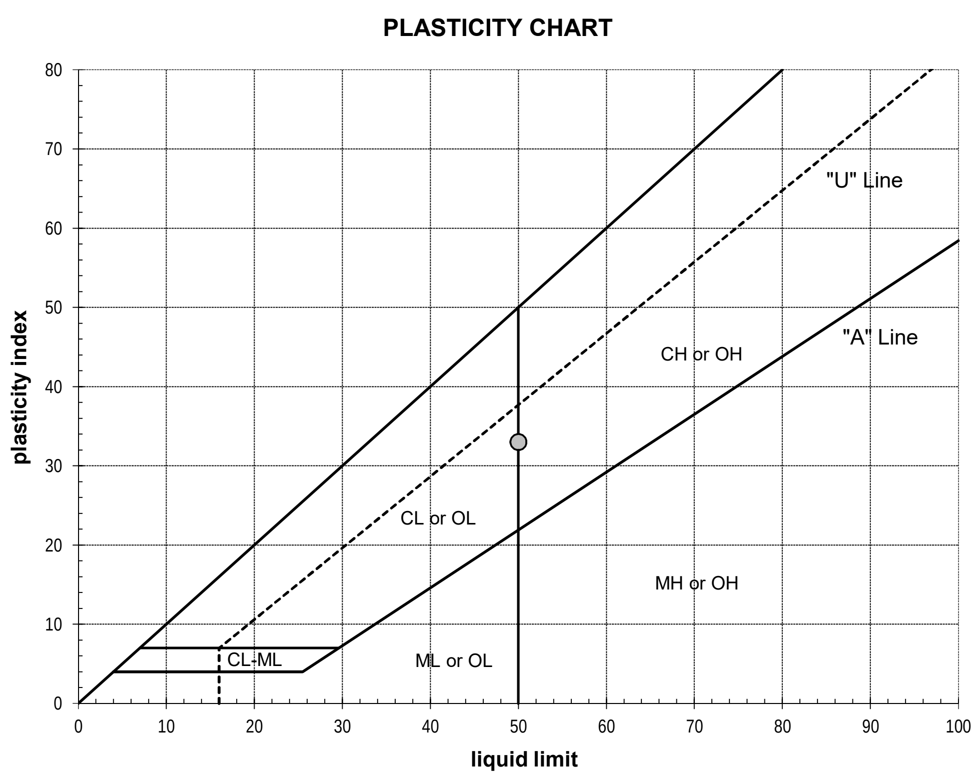
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.59	29.07		
dry soil + pan weight, g =	27.47	27.90		
pan weight, g =	20.70	20.85		
moisture, % =	16.5 %	16.6 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	13.1%
% silt and clay =	86.9%
% silt =	n/a
% clay =	n/a
moisture content =	23.0%

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-711
		REPORT DATE: 09/27/17	FIELD ID: TP4.2
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey GRAVEL with Sand and Cobbles	Test Pit TP-04 depth = 12 feet	GC, Clayey Gravel with Sand and cobbles

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

LABORATORY TEST DATA

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

ADDITIONAL DATA

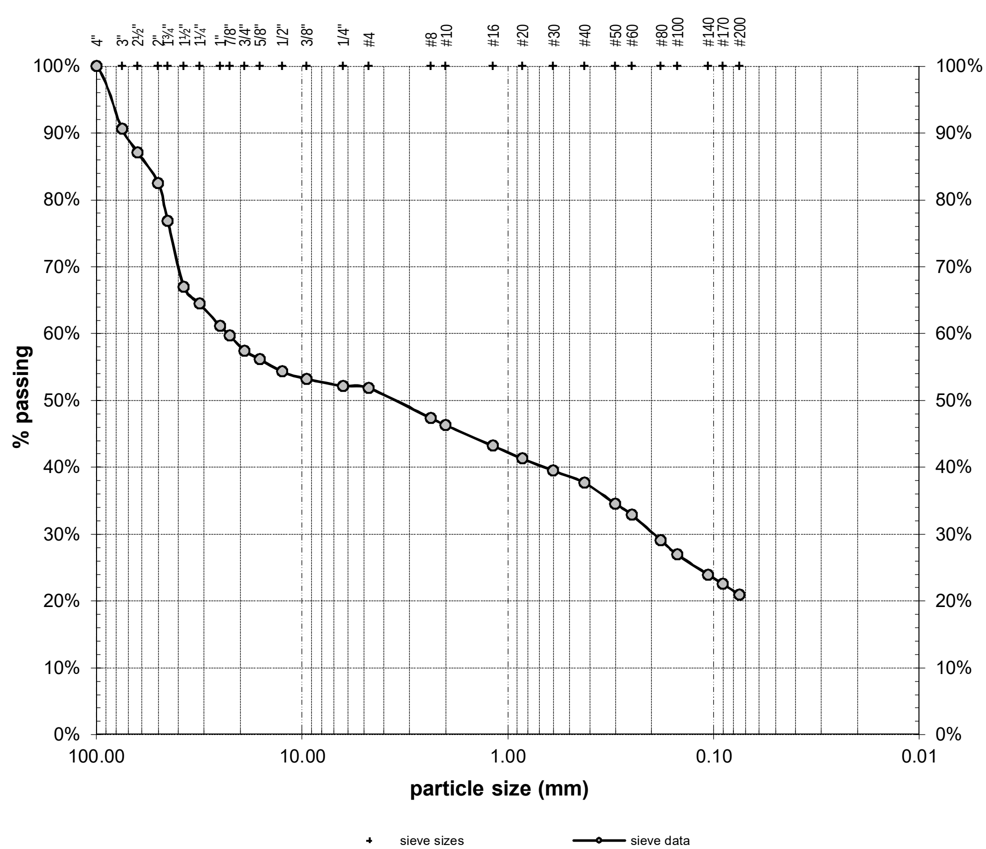
initial dry mass (g) =	19823.7		
as-received moisture content =	26.7%	coefficient of curvature, C_C =	n/a
liquid limit =	44	coefficient of uniformity, C_U =	n/a
plastic limit =	26	effective size, $D_{(10)}$ =	n/a
plasticity index =	18	$D_{(30)}$ =	0.195 mm
fineness modulus =	n/a	$D_{(60)}$ =	22.952 mm

SIEVE DATA

% gravel = 48.2%
% sand = 30.9%
% silt and clay = 20.9%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0	100.0%			
	3.00"	75.0	90.6%			
	2.50"	63.0		87.1%		
	2.00"	50.0	82.5%			
	1.75"	45.0		76.8%		
	1.50"	37.5	67.0%			
	1.25"	31.5		64.5%		
	1.00"	25.0	61.2%			
	7/8"	22.4		59.7%		
	3/4"	19.0	57.4%			
	5/8"	16.0		56.1%		
	1/2"	12.5	54.3%			
	3/8"	9.50	53.2%			
	1/4"	6.30	52.2%			
	#4	4.75	51.8%			
SAND	#8	2.36		47.3%		
	#10	2.00	46.3%			
	#16	1.18		43.2%		
	#20	0.850	41.3%			
	#30	0.600		39.5%		
	#40	0.425	37.7%			
	#50	0.300		34.5%		
	#60	0.250	32.9%			
	#80	0.180		29.1%		
	#100	0.150	26.9%			
	#140	0.106		23.9%		
	#170	0.090		22.5%		
	#200	0.075	20.9%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
09/21/17	BTT/RTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Stephens Property La Center, Washington	Mark, Perry, and Carleen Stephens Mary Rerick 24600 NE 98th Court Battle Ground, Washington 98604	17248	S17-711
		REPORT DATE: 09/27/17	FIELD ID: TP4.2
		DATE SAMPLED: 09/08/17	SAMPLED BY: CWS

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Clayey GRAVEL with Sand and Cobbles	Test Pit TP-04 depth = 12 feet	GC, Clayey Gravel with Sand and cobbles

LABORATORY TEST DATA

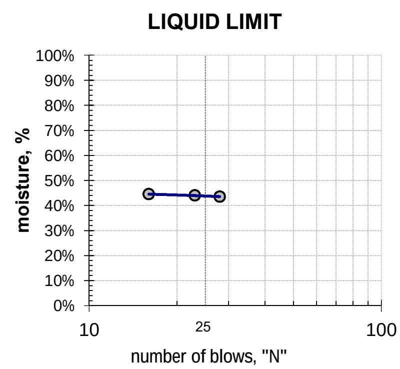
LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

ATTERBERG LIMITS

liquid limit = 44
plastic limit = 26
plasticity index = 18

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	35.68	39.31	38.18	
dry soil + pan weight, g =	31.16	33.69	32.84	
pan weight, g =	20.77	20.92	20.84	
N (blows) =	28	23	16	
moisture, % =	43.5 %	44.0 %	44.5 %	

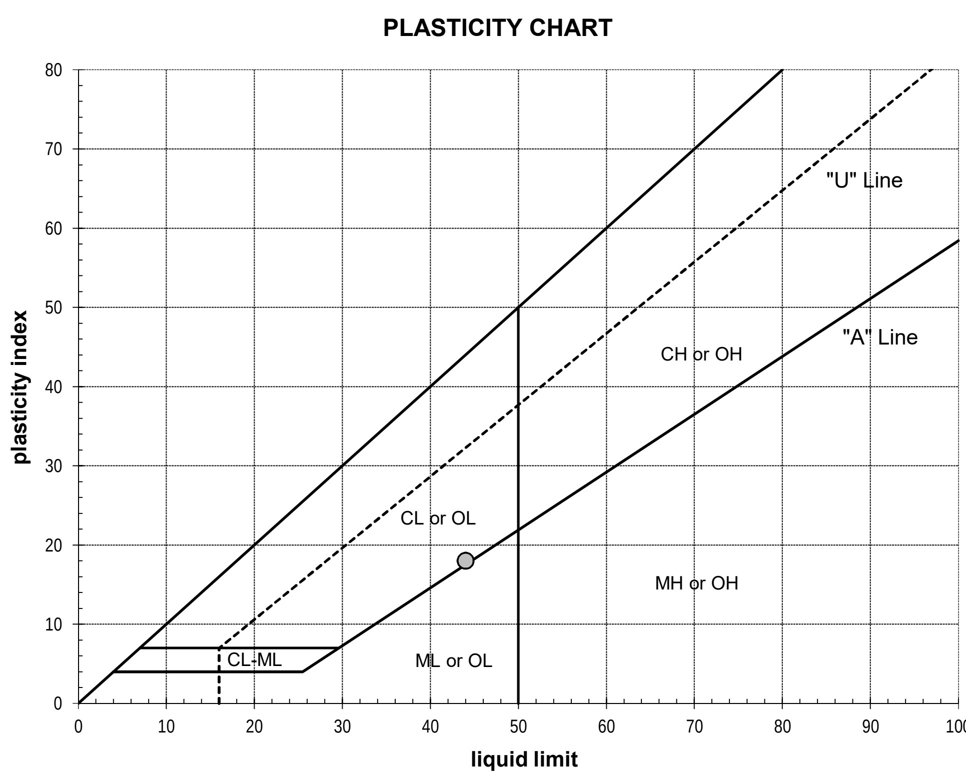


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.65	28.95		
dry soil + pan weight, g =	27.06	27.30		
pan weight, g =	20.82	20.85		
moisture, % =	25.5 %	25.6 %		



ADDITIONAL DATA

% gravel = 48.2%
% sand = 30.9%
% silt and clay = 20.9%
% silt = n/a
% clay = n/a
moisture content = 26.7%

DATE TESTED	TESTED BY
09/25/17	RTT

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-1
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	GEOLOGIST	DATE
La Center, Washington	L&S Contractors	Excavator	CWS	10/13/17
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	240 amsl	Not Encountered	0705	0755

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 8 to 10-inches of topsoil and grass.					
		Hillsboro silt loam		CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
	TP-1.1		A-6(15)				16.8	77.9	39	20	
5											
				CH		Brown with orange/grey mottling, moist, medium stiff, fat CLAY (Soil Type 2).					
10	TP-1.2		A-7-6(40)				30.8	86.1	62	44	
15						Bottom of test pit at 15 feet bgs. Groundwater not encountered.					

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-2
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	GEOLOGIST	DATE
La Center, Washington	L&S Contractors	Excavator	CWS	10/13/17
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	250 amsl	Not Encountered	0800	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 8 to 10-inches of topsoil and grass.					
		Hillsboro silt loam	A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
5	TP-2.1		A-7-6(36)	CH		Brown with orange/grey mottling, moist, medium stiff, fat CLAY with sand (Soil Type 2).	26.7	82.3	60	43	
10											
						Bottom of test pit at 13 feet bgs. Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-3
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	GEOLOGIST	DATE
La Center, Washington	L&S Contractors	Excavator	CWS	10/13/17
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	230 amsl	Not Encountered	0832	0900

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 8 to 10-inches of topsoil and grass.					
		Hillsboro silt loam	A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
5											
	TP-3.1		A-7-6(30)	CH		Brown with orange/grey mottling, moist, medium stiff, fat CLAY (Soil Type 2).	23.0	86.9	50	33	
10											
						Bottom of test pit at 13 feet bgs. Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-4
PROJECT LOCATION	**CONTRACTOR**	**EQUIPMENT**	**GEOLOGIST**	**DATE**
La Center, Washington	L&S Contractors	Excavator	CWS	10/13/17
TEST PIT LOCATION	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**START TIME**	**FINISH TIME**
See Figure 2	240 amsl	Not Encountered	0907	0930

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 8 to 10-inches of topsoil and grass.					
		Hillsboro silt loam	A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
5											
			A-7-6	CH		Brown with orange/grey mottling, moist, medium stiff, fat CLAY (Soil Type 2).					
				GC		Brown, moist, medium dense, clayey GRAVEL with sand and cobbles (Soil Type 3).					
10											
	TP-4.2		A-2-7(0)				26.7	20.9	44	18	
						Bottom of test pit at 13 feet bgs. Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-5
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	GEOLOGIST	DATE
La Center, Washington	L&S Contractors	Excavator	CWS	10/13/17
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	240 amsl	Not Encountered	0940	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 8 to 10-inches of topsoil and grass.					
		Hillsboro silt loam	A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
5											
			A-7-6	CH		Brown with orange/grey mottling, moist, medium stiff, fat CLAY (Soil Type 2).					
			A-2-7	GC		Brown, moist, medium dense, clayey GRAVEL with sand and cobbles (Soil Type 3).					
10											
						Bottom of test pit at 13 feet bgs. Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-6
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	GEOLOGIST	DATE
La Center, Washington	L&S Contractors	Excavator	CWS	10/13/17
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	230 amsl	Not Encountered	1010	1040

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 8 to 10-inches of topsoil and grass.					
		Gee silt loam	A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
5											
			A-7-6	CH		Brown with orange/grey mottling, moist, medium stiff, fat CLAY (Soil Type 2).					
			A-2-7	GC		Brown, moist, medium dense, clayey GRAVEL with sand and cobbles (Soil Type 3).					
10											
15						Bottom of test pit at 15 feet bgs. Groundwater not encountered.					

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-7
PROJECT LOCATION La Center, Washington	CONTRACTOR L&S Contractors	EQUIPMENT Excavator	GEOLOGIST CWS	DATE 10/13/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 275 amsl	GROUNDWATER DEPTH Not Encountered	START TIME 1050	FINISH TIME 1110

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 8 to 10-inches of topsoil and grass.					
		Gee silt loam	A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
5			A-7-6	CH		Brown with orange/grey mottling, moist, medium stiff, fat CLAY with trace gravels (Soil Type 2).					
10											
						Bottom of test pit at 13 feet bgs. Groundwater not encountered.					
15											

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PROJECT NAME Stephens Property	CLIENT Stephens-Rerick		PROJECT NO. 17248	TEST PIT NO. TP-8
PROJECT LOCATION La Center, Washington	CONTRACTOR L&S Contractors	EQUIPMENT Excavator	GEOLOGIST CWS	DATE 10/13/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 280 amsl	GROUNDWATER DEPTH Not Encountered	START TIME 1120	FINISH TIME 1150

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 8 to 10-inches of topsoil and grass.					
		Hillsboro silt loam	A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
5			A-7-6	CH		Brown with orange/grey mottling, moist, medium stiff, fat CLAY (Soil Type 2).					
10											
			A-2-7	GC		Brown, moist, medium dense, clayey GRAVEL with sand and cobbles (Soil Type 3).					
						Bottom of test pit at 13 feet bgs. Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-9
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	GEOLOGIST	DATE
La Center, Washington	L&S Contractors	Excavator	CWS	10/13/17
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	300 amsl	Not Encountered	1155	1222

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 8 to 10-inches of topsoil and grass.					
		Hillsboro silt loam	A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
5			A-2-7	GC		Brown, moist, medium dense, clayey GRAVEL with sand and cobbles (Soil Type 3).					
			A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
			A-7-6	CH		Brown with orange/grey mottling, moist, medium stiff, fat CLAY (Soil Type 2).					
10						Bottom of test pit at 10 feet bgs. Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-10
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	GEOLOGIST	DATE
La Center, Washington	L&S Contractors	Excavator	CWS	10/13/17
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	340 amsl	Not Encountered	1230	1305

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 8 to 10-inches of topsoil and grass.					
		Hesson gravelly clay loam	A-2-7	SC		Golden brown, moist, medium dense, clayey SAND (Soil Type 4).					
5											
10											
						Bottom of test pit at 12 feet bgs. Groundwater not encountered.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Stephens Property	Stephens-Rerick		17248	TP-11
PROJECT LOCATION La Center, Washington	CONTRACTOR L&S Contractors	EQUIPMENT Excavator	GEOLOGIST CWS	DATE 10/13/17
TEST PIT LOCATION See Figure 2	APPROX. SURFACE ELEVATION 320 amsl	GROUNDWATER DEPTH Not Encountered	START TIME 1320	FINISH TIME 1400

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 8 to 10-inches of topsoil and grass.					
		Hesson gravelly clay loam	A-2-7	GC		Brown, moist, medium dense, clayey GRAVEL with sand and cobbles (Soil Type 3).					
5			A-6	CL		Brown with slight orange/grey mottling, dry to moist, medium stiff, lean CLAY with sand (Soil Type 1).					
10											
15						Bottom of test pit at 12 feet bgs. 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.

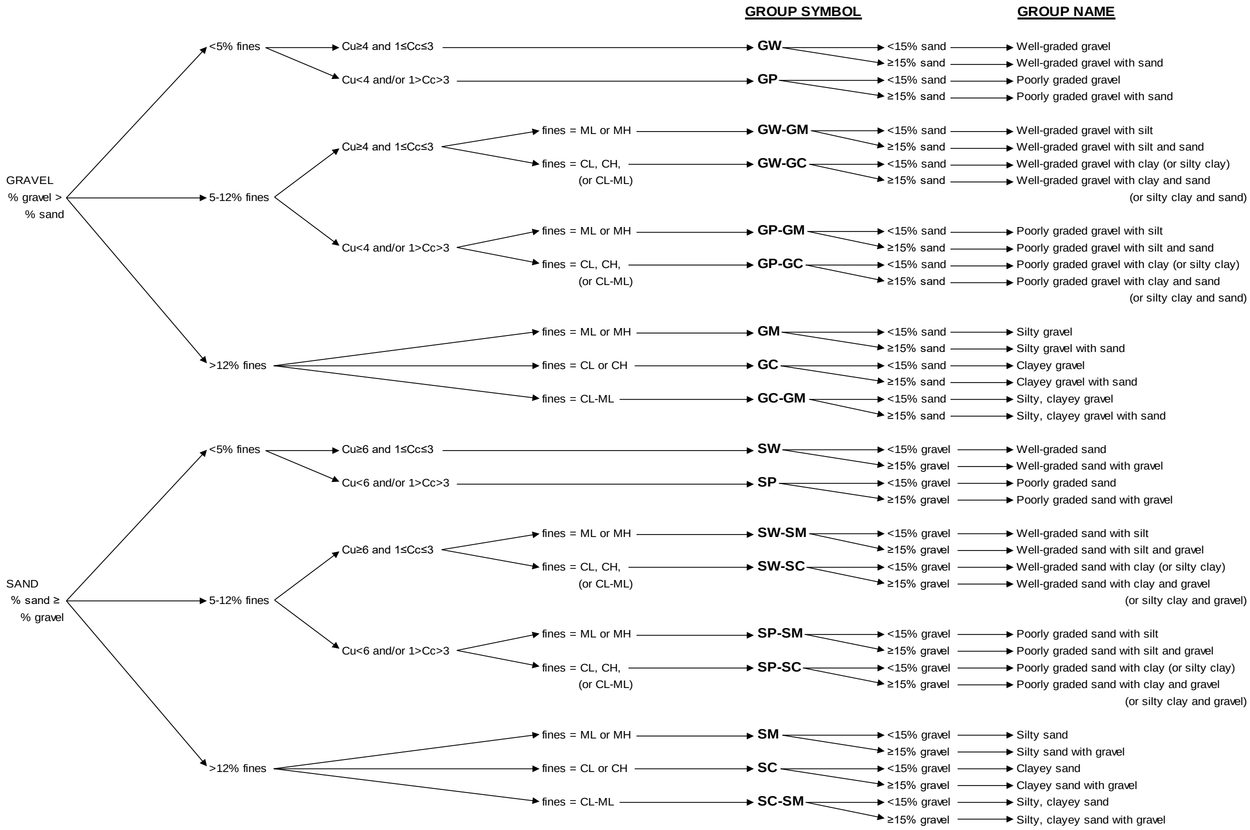
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

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			

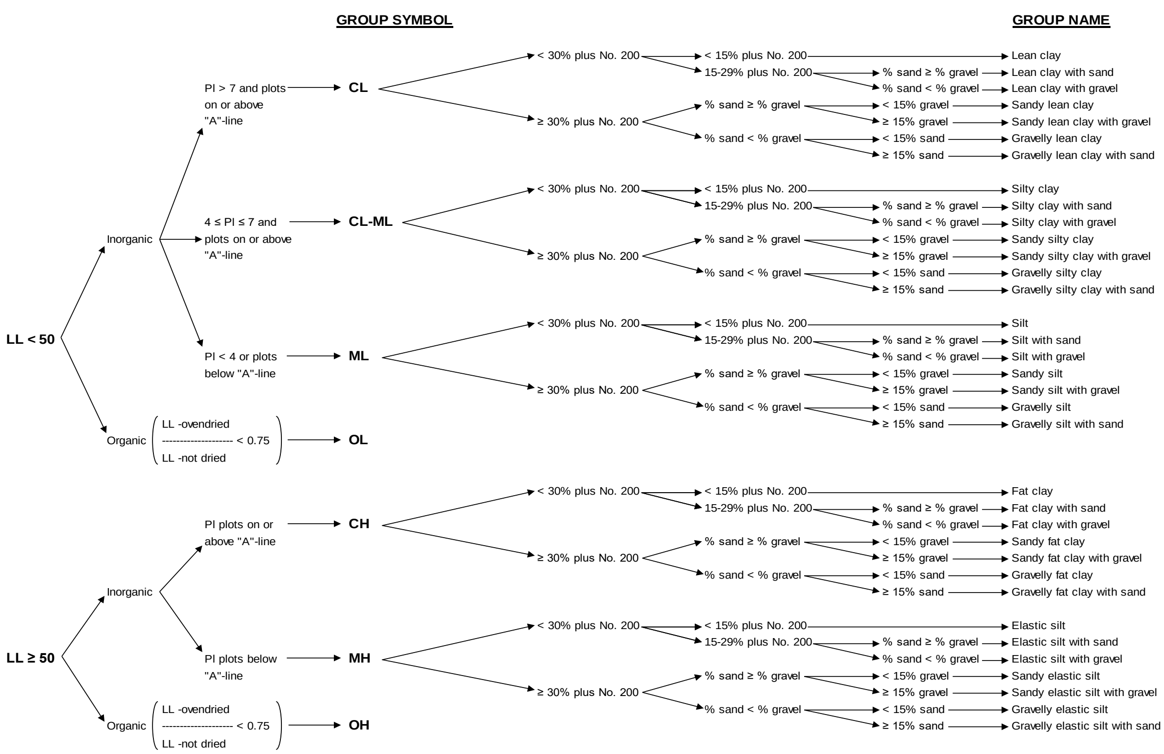
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

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
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

Date: October 20, 2017
Project: Stephens Property
La Center, Washington

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

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

Report Conclusions and Preliminary Nature

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

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

Additional Investigation and Construction QA/QC

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

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

Collected Samples

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

Report Contents

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

Report Limitations for Contractors

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

Report Ownership

Columbia West retains the ownership and copyright property rights to this entire report and its contents, which may include, but may not be limited to, figures, text, logs, electronic media, drawings, laboratory reports, and appendices. This report was prepared solely for the client, and other relevant approved users or parties, and its distribution must be contingent upon prior express written consent by Columbia West. Furthermore, client or approved users may not use, lend, sell, copy, or distribute this document without express written consent by Columbia West. Client does not own nor have rights to electronic media files that constitute this report, and under no circumstances should said electronic files be distributed or copied. Electronic media is susceptible to unauthorized manipulation or modification, and may not be reliable.

Consultant Responsibility

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

APPENDIX E
PHOTO LOG

STEPHENS PROPERTY
LA CENTER, WASHINGTON
PHOTO LOG

Site Terrain, Facing West from TP-8.

Site Terrain, Facing West From TP-11

STEPHENS PROPERTY
LA CENTER, WASHINGTON
PHOTO LOG

Site Terrain, Facing North From TP-1

Conducting Test Pits, TP-10

STEPHENS PROPERTY
LA CENTER, WASHINGTON
PHOTO LOG

Typical Soil Profile Observed on the Site, TP-1