Columbia West Engineering, Inc.
Geotechnical • Environmental • Materials Testing

# Geotechnical Site Investigation
# Goode Property
# La Center, Washington

January 31, 2008

11917 NE 95th Street
Vancouver, Washington 98682
phone: 360-823-2900, fax 360-823-2901
www.columbiawestengineering.com

# GEOTECHNICAL SITE INVESTIGATION
# GOODE PROPERTY
# LA CENTER, WASHINGTON

**Prepared For:** **Randy Goode**
**862 E 15th Circle**
**La Center, Washington 98629**

**Site Location:** **NW Pacific Highway and NW Larson Drive**
**La Center, Washington**

**Prepared By:** **Columbia West Engineering, Inc.**
**11917 NE 95th Street**
**Vancouver, Washington 98682**
**Phone: 360-823-2900**
**Fax: 360-823-2901**

**Date Prepared:** **January 31, 2008**

# TABLE OF CONTENTS

# LIST OF FIGURES

# LIST OF APPENDICES

# GEOTECHNICAL SITE INVESTIGATION
# GOODE PROPERTY
# LA CENTER, WASHINGTON

## 1.0 INTRODUCTION

Columbia West Engineering, Inc. was retained by Randy Goode to conduct a geotechnical site investigation for the Goode Property in La Center, Washington. The purpose of the investigation was to observe and assess subsurface soil conditions and provide geotechnical engineering analyses to support property development, planning, and design recommendations. The specific scope of services was outlined in a proposal contract dated November 20, 2007, and authorized by client signature on December 6, 2007. 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 G.

### 1.1 General Site Information

As indicated on Figures 1 and 2, the Goode Property is located south of Pacific Highway and west of NW Larsen Drive in La Center, Washington. The subject site is bordered by residential property to the west, north and east, and the East Fork Lewis River to the South. The regulatory jurisdictional agency is Clark County. The approximate latitude and longitude are N 45° 52' 5" and W 122° 41' 18" and the legal description is a portion of the SE ¼ of Section 33, T5N, R1E, Willamette Meridian and a portion of the NE ¼ of Section 4, T4N, R1E, Willamette Meridian. The subject site is approximately 52 acres in size.

### 1.2 Proposed Development

Correspondence with the client indicates the site is planned for low-density acreage-lot residential development. Based upon a preliminary site plan prepared by Moss & Associates, 10 residential lots are planned for development. 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 and Cowlitz Counties, Washington* (Evarts, USGS Scientific Investigations Map 2844, United States Geological Survey [USGS, 2004) near surface soils are expected to consist of Pleistocene cataclysmic-flood deposits of unconsolidated clay, silt, and fine to medium sand (Qfs) underlain by Pleistocene or Pliocene semi-consolidated pebble and cobble gravel conglomerate (QTc), Miocene Sentinal Bluffs basalt (Tgsb) and Eocene Andesite (Ta). The *Geologic Map of the Vancouver Quadrangle, Washington and Oregon* (Washington Division of Geology and Earth Resources, Open File Report 87-10, Revised November 1987), indicates similar conditions, with near-surface soils expected to consist of upper-Pleistocene, fine-textured, rhythmically bedded periglacial deposits derived from catastrophic outburst floods of Glacial Lake Missoula (Qs) underlain by Pleistocene-Miocene well indurated to weakly consolidated sedimentary deposits of the Troutdale Formation.

The *Soil Survey of Clark County, Washington* (United States Department of Agriculture, Soil Conservation Service [USDA SCS], November 1972) identifies surface soils as primarily Hillsboro silt loam, Odne silt loam, and Gee silt loam. Although soil conditions may vary from the broad USDA descriptions, Hillsboro, Odne and Gee soils are generally fine to medium-textured with low to moderate permeability and moderate to severe erosion hazard. The shrink/swell potential is moderate, the shear strength is low, and the soils are somewhat compressible and generally moisture sensitive. Odne soils are hydric and typically associated with existing or former wetland areas.

## 3.0 REGIONAL SEISMOLOGY

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

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

<u>*Portland Hills Fault Zone*</u>

The Portland Hills Fault Zone consists of several northwest-trending faults located along the eastern boundary of the Portland Hills. The fault zone is approximately 25 miles in length and is located approximately 16 miles southwest of the site. According to *Seismic Design Mapping, State of Oregon* (Geomatrix Consultants, 1995), there is no definitive consensus among geologists as to the zone fault type. Several alternate interpretations have been suggested, including various strike-slip and dipping thrust fault theories.

Evidence exists to suggest that fault movement has impacted shallow Holocene deposits and deeper Pleistocene sediments. Seismologists recorded 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 30 miles southwest of the site, the 50-mile long Gales Creek-Newberg-Mt. Angel Structural Zone consists of a series of discontinuous northwest-trending faults. Possible late-Quaternary geomorphic surface deformation may exist along the structural zone *(Geomatrix Consultants, 1995)*. Although no definitive evidence of impacts to Holocene sediments has reportedly been observed, a M5.6 earthquake occurred in March 1993 near Scotts Mills, approximately four miles south of the mapped extent of the Mt. Angel fault. It is unclear if the earthquake occurred along the fault zone or a parallel structure. Therefore, the Gales Creek-Newberg-Mt. Angel Structural Zone is considered potentially active.

*Lacamas Creek-Sandy River Fault Zone*

The northwest-trending Lacamas Creek Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 20 miles south of the site. According to *Geology and Groundwater Conditions of Clark County Washington* (USGS Water Supply Paper 1600, Mundorff, 1964*)* and *the Geologic Map of the Lake Oswego Quadrangle* (Oregon DOGAMI Series GMS-59, 1989), the Lacamas Creek fault zone consists of shear contact between the Troutdale Formation and underlying Oligocene andesite-basalt bedrock. Secondary shear contact associated with the fault zone may have produced a series of prominent northwest-southeast geomorphic lineaments in proximity to the site. Recorded mild seismic activity during the recent past indicates this area may be potentially seismogenic.

*Cascadia Subduction Zone*

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

## 4.0 GEOTECHNICAL FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance, four mechanically augered soil borings (SB-01 through SB-04), three hand-augered soil borings (HA-01 through HA-03) and three test pit explorations (TP-01 through TP-03) was conducted at the site. Soil borings SB-01 through SB-04 were advanced with a trailer-mounted auger drill rig. Test pit exploration was performed with a track-mounted excavator. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications. Standard penetration test (SPT) blow counts were obtained using split-spoon samplers at regular intervals in soil borings SB-01 through SB-03. Torvane shear, pocket penetrometer and nuclear density gauge tests were performed within the test pits. As indicated on the boring and test pit logs, field tests provided measurements and estimates of moisture content, relative density, shear strength, and in situ penetration resistance. Disturbed and undisturbed soil samples were collected from relevant soil horizons and submitted for laboratory analyses. Sample results are presented in Appendix A. The boring and test pit locations are indicated on Figure 2. The boring logs are presented in Appendix B and test pit logs are presented in Appendix C. Soil description and classification information are provided in Appendix D.

### 4.1 Surface Investigation and Site Description

Field reconnaissance and review of recent topographic survey indicate the subject property ranges in elevation from approximately 10 to 220 feet above mean sea level. The site generally occupies a broadly eroded terrace bordered on the south by the East Fork Lewis River. Most of the upland portions of the site generally slope down-gradient from northeast to southwest with grades ranging from four to 10 percent. Steep slopes approaching 50 to 75 percent exist primarily near the southern property boundary at the edge of the river floodplain. Exposed outcrops of bedrock were observed and evidence of past shallow slumps and ground movement was apparent in the western portion of this slope. The entire slope was heavily forested with native conifer and deciduous trees. An abandoned rock quarry and related limited access road grade exist along these steep southern slopes. A stream and drainage ditch flow from east to west in the central area of the property into a heavily forested ravine near the western property boundary. An existing residential structure, two outbuildings and a small pond are present on the northern boundary of the site adjacent to NW Pacific Highway. Except for the forested areas noted above, the site generally consists of open fields, vegetated with grasses.

#### 4.1.1 Subsurface Exploration and Investigation

Four soil borings and three hand-auger investigations were advanced to a maximum depth of 53 feet on December 13, 2007. Three test pits were advanced to a maximum depth of 21 feet on October 14, 2005. Test pit and boring locations were selected to observe subsurface soil characteristics near slopes and in areas proposed for development. Test pit and soil boring locations are indicated on Figure 2.

#### 4.1.2 Soil Type Description

The field investigation indicated the site is generally covered with a topsoil layer approximately 12 inches thick at the locations observed. The subsurface profile was somewhat similar for most test pits and may generally be described by soil types identified in the following text.

*Soil Type 1 – Lean CLAY / Lean CLAY with sand*

Soil Type 1 was observed to consist of reddish brown, moist to wet, medium-stiff to very stiff, plastic, lean CLAY and lean CLAY with sand. Soil Type 1 was encountered underlying the topsoil in all borings and test pits and extended to depths of 7.5 to 25 feet. Laboratory analysis indicated the in situ moisture content, expressed as the ratio of weight of water to weight of dry soil, varied from approximately 24 to 34 percent. SPT blow counts conducted upon Soil Type 1 varied from approximately 6 to 30 blows per foot. Analytical laboratory testing conducted on representative samples of Soil Type 1 indicated approximately 75 to 89 percent by weight passing the No. 200 sieve. Atterberg limits testing resulted in liquid limits that vary from 31 to 46 and plasticity indexes ranging from 13 to 26. Soil Type 1 is classified CL according to USCS specifications.

*Soil Type 1A – Lean CLAY with Sand*

Soil Type 1A was observed to consist of blueish gray, wet to saturated, medium-stiff to stiff, plastic, lean CLAY with sand. Soil Type 1A was encountered underlying Soil Type 1 in soil borings SB-01 and SB-04 and test pits TP-02 and TP-03 at depths ranging from 16 to 20 feet below ground surface. Soil Type 1A is similar in texture to Soil Type 1. Laboratory analysis indicated the in situ moisture content, expressed as the ratio of weight of water to weight of dry soil, varied from approximately 29 to 35 percent. Soil Type 1A is classified CL according to USCS specifications.

*Soil Type 2 – Severely weathered CONGLOMERATE*

Soil Type 2 was observed to consist of light gray to reddish-brown with varying mottles, sub-angular gravels and cobbles in a cemented silt, clay and sand matrix and likely represents severely weathered CONGLOMERATE formed from Pleistocene sedimentary deposits. Soil Type 2 was observed underlying Soil Type 1A in soil borings SB-01 and SB-04, underlying Soil Type 1 in soil boring SB-03 and underlying the topsoil in TP-01. Laboratory analysis indicated the in situ moisture content, expressed as the ratio of weight of water to weight of dry soil, was approximately 23 percent. SPT blow counts conducted upon Soil Type 2 varied from approximately 25 blows per foot to 50 for a six-inch penetration. Soil Type 2 most likely represents the Pleistocene or Pliocene semi-consolidated pebble and cobble gravel conglomerate (QTc) according to the *Geological Map of the Ridgefield Quadrangle, Clark and Cowlitz Counties, Washington* (Evarts, USGS Scientific Investigations Map 2844, USGS, 2004).

*Soil Type 3 – Basalt Bedrock*

Soil Type 3 was observed to consist of basalt bedrock or bedrock boulders. In general the upper few feet of the basalt appeared to be severely weathered and consisted of basalt boulders in a sandy clay matrix. Bedrock was encountered underlying Soil Types 1 and 2 at a depth of 8 feet in test pit TP-01 and soil boring SB-02. In addition, bedrock was exposed at the surface in several outcrops located on the steeply sloped terraces in the southern portion of the site. Excavator refusal during test pit excavation indicates drilling and explosive blasting may be required to install utilities and construct site improvements in dense bedrock areas. Soil Type 3 most likely represents the Miocene Sentinal Bluffs basalt (Tgsb) or Eocene Andesite (Ta) according to the *Geological Map of the Ridgefield Quadrangle, Clark and Cowlitz Counties, Washington* (Evarts, USGS Scientific Investigations Map 2844, USGS, 2004).

### 4.1.3 Ground Water

Ground water was observed at depths ranging from 2.5 to 24 feet below existing ground surface in the boring locations. According to *Ground Water Data for the Portland Basin, Oregon and Washington*, (USGS, Open-File Report 90-126), static ground water at nearby wells has been observed at depths of approximately 18 to 30 feet. Ground water elevation may vary depending upon the location, elevation, and screened interval of the well. Ground water levels are also often subject to seasonal variance and may rise during extended periods of increased precipitation. Perched ground water may also be present in localized areas. Seeps and springs may become evident during site grading, primarily in areas cut below existing grade. Structures and drainage design should be planned accordingly.

# 5.0 SLOPE STABILITY

To identify appropriate setback distances from possible slope hazards present at the site, Columbia West conducted a literature review, slope reconnaissance, and stability analysis. These items are discussed below.

## 5.1 Literature Review

Columbia West reviewed *Slope Stability, Clark County, Washington* (Fiksdal, 1975) to assess site slope characteristics. The report identifies four levels of potential slope instability within Clark County: (1) stable areas – no slides or unstable slopes, (2) areas of potential instability because of underlying geologic conditions and physical characteristics associated with steepness, (3) areas of historical or still active landslides, and (4) older landslide debris. The majority of the site is mapped as (1) stable areas-no slides or unstable slopes. The steep riverbank terrace slopes and drainage ravines in the southern portion of the site are mapped as (2) areas of potential instability because of underlying geologic conditions and physical characteristics associated with steepness.

## 5.2 Slope Reconnaissance

To observe geomorphic conditions, Columbia West personnel conducted visual and physical reconnaissance of the steeply sloped terraces adjacent to the East Fork Lewis River in the southern portion of the site. Test pits and soil borings were explored near the crest of the slopes. Subsurface soil conditions at the locations observed generally consisted of medium-stiff lean CLAY underlain by semi-consolidated, severely weathered conglomerate and slightly weathered basalt bedrock. Several rock outcrops of basalt were observed in river sideslopes. Based upon visual inclinometer assessment, the slope grades typically range from 20 to 50 percent, with some localized steeper areas associated with rock outcrops. The overall terrace slope height, measured vertically from the toe to crest, varied from approximately 100 to 130 feet. Slopes currently support heavy vegetation consisting of established deep-rooted conifer and deciduous trees and mixed understory bushes, grasses, ferns, and shrubs. Shallow ground water seeps and surface water were observed along the face of the slope.

Most portions of the slopes were planar with little evidence of instability. However, a few localized slumps were also observed and shallow soil bulges and several trees with inclined or rotated trunks were also present. These observations typically indicate the presence of soil creep or shallow soil movement along the slope face. Without drainage or other stabilizing measures, retrogressive slope activity is possible, particularly near active springs and seeps. The potential for soil creep and retrogressive slope movement illustrates the importance of proper site planning and drainage design.

## 5.3 Slope Stability Analysis

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

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

### 5.3.1 SLOPE/W Input

*Slope Geometry*

The slope profile location selected for slope stability analyses (A-A’) is indicated on Figure 2 and the slope cross-section is shown on Figure 3. The cross-section was derived from topographical data for the site based upon topographic contour maps provided by the civil site plan engineer. The approximate delineations of individual soil layers were determined based upon boring and test pit data and visual observation of the subject site and adjacent properties. Selection of the cross-section location was based upon several factors, including slope height, length, grade, and proximity to proposed residential structures.

*Soil Characteristics*

Soil strength parameters and unit weights selected for computer modeling were based upon in-situ soil testing, SPT blow counts, analytical laboratory analysis, research of existing soil mechanics data, and visual observation. Input parameters for the conglomerate were determined using the Hoek-Brown failure criterion, which evaluates observable characteristics of rock to provide equivalent friction angles and cohesion values for use in limit equilibrium and other analyses.

Input values were generally selected to provide for conservative analyses. SLOPE/W utilizes the individual soil layer moist unit weight, saturated unit weight, internal shear strength parameters, pore water pressure, and slope geometry to determine the location of the most critical failure plane. The soil and rock strength characteristics are summarized in Table 1. Determinations of strength parameters for rock are included in Appendix F.

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

| Soil Type | Moist Unit Weight (pcf) | Cohesion (psf) | Drained Friction Angle (degrees) |
|---|---|---|---|
| Lean Clay with sand (Soil Type 1) | 120 | 140 | 24 |
| Lean Clay with sand (Soil Type 1A) | 120 | 140 | 24 |
| Conglomerate (Soil Type 2) | 140 | 600* | 46* |
| Basalt Bedrock (Soil Type 3) | - | - | - |

**Equivalent friction angles and cohesive strength for rock mass are based upon Hoek-Brown failure criterion (see Appendix F).*

### *Ground Water*

A piezometric surface, reflecting the elevation of the ground water table observed during the field investigation, was included in the slope stability analyses. The estimated piezometric surface reflects local slope topography, soil layer geometry, visual observations of active ground water springs and surface water, and consideration of seasonal fluctuations. To compensate for these factors, the ground water table was assumed to be at the ground surface.

### *Seismic Considerations*

Seismic events can induce horizontal ground acceleration significantly in excess of static conditions, and should be adequately modeled to predict slope stability. A pseudostatic analysis represents the potential effects of a seismic event by using a horizontal acceleration that effectively increases inertial inter-slice forces during computation. Pseudostatic analyses were performed for the slope cross-section using a horizontal acceleration equal to 0.1g (10 percent of the force of gravity). The horizontal acceleration used in the analysis is slightly more than one-half times the peak ground acceleration (PGA) of 0.187g for an anticipated earthquake at the subject site with a 90-percent probability of not being exceeded within 50 years (i.e. a 475-year return seismic event). This meets current geotechnical state of the practice for slope stability analysis in similar environments.

### *Vegetation*

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

### 5.3.2 SLOPE/W Results

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

Based upon results of the analyses as indicated in Appendix E, factors of safety for the steep terrace slopes in the southern portion of the subject property were less than 1.5 for static conditions or 1.1 for pseudostatic conditions. This can be interpreted to indicate elevated potential instability risk. The location of the critical failure surface entry points behind the top of slope indicates a need for maintaining horizontal setback distances for future structures and loads. Recommended horizontal setback distances from the top of slope, as discussed below, were established to exclude loads from proposed development within anticipated critical entry surfaces for static and pseudostatic factors of safety.

## 5.4 Slope Setback Distance

To reduce the risk of slope instability, it is recommended that residential structures maintain a horizontal building setback distance of at least 140 feet from the top of slopes in the western portion of the property as indicated on Figure 2. The presence of shallow bedrock near the slopes in the eastern portion of the sloped area allows for a decreased horizontal setback distance. Structures in the eastern portion of the property should maintain a horizontal building setback distance of at least 30 feet from the top of slopes. Certain structural facilities which may be able to tolerate higher levels of risk for slope movement may encroach within the setback distance as described below in Section 5.4.2, *Potential Encroachment of Structures within Slope Setback Zone*.

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

### 5.4.1 Grading Recommendations within Slope Setback Zone

Major soil disturbance, grading, vegetation removal, logging, and other major construction activities should be prohibited within the slope setback zone. 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.

Major cuts or fills, mass grading, or site improvement construction activities are not recommended along the slopes or within the geotechnical setback zone. However, the geotechnical setback zone is not intended to be a do-not-disturb conservation area. Small disturbances such as minor landscaping, building fences, removing shrubs, or establishing a yard are acceptable. The text herein pertains only to the geotechnical aspect of construction within the recommended geotechnical setback zones.

### 5.4.2 Potential Encroachment within Slope Setback Zone

Based upon correspondence with the client and civil site plan engineer, a roadway is proposed within the geotechnical slope setback zone. Alternative roadway alignments outside of the geotechnical setback should be investigated. If roadway construction outside of the slope setback zone is not achievable, encroachment may be feasible, depending upon dimensions, locations, and specific design features of the proposed roadway. If necessary, grading performed within the setback zone should be limited to avoid disturbance of existing soil and increased risk of slope instability. Additional geotechnical assessment is recommended if the proposed road encroaches within the setback zone.

Correspondence with the client also indicated that existing overhead power utilities within the setback zone may be placed underground during site development. In the event of slope instability, underground utilities may become damaged or destroyed. Repair of utilities may require large temporary and permanent earthworks and significant cost. Such risk should be understood prior to investment of significant resources.

Existing septic percolation test pits were observed within the geotechnical slope setback zone at the time of site investigation. Although Columbia West recommends limiting development within the setback zone, encroachment may be feasible if septic systems are proposed at these locations. Anticipated volumes of water introduced to the slope by septic systems are not expected to be detrimental to overall slope stability.

Operation of storm water utilities or systems capable of introducing large volumes of water to site slopes will increase the risk of instability. Therefore, these facilities should be located outside of the setback zone if possible. Utilities or facilities within the setback zone should be monitored to ensure proper operation. Specific recommendations for storm water management near site slopes are presented in Section 6.10, *Drainage*.

Elevated risk of settlements beyond tolerable and serviceable limits exists for facilities constructed within the established slope setback zones. Proposed encroachment within the setback zone should be evaluated by the geotechnical engineer on a case-by-case basis.

## 5.5 Slope Stability Limitations and Risk

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

Due to multiple unknowns inherent in slope stability analysis it is often difficult or impossible to definitively predict stability. This slope stability analysis is based upon information gathered from research of existing data, subsurface soil explorations, and visual site observations as described in the text herein. This slope stability analysis applies only to the proposed and identified lots in the southern portion of the site and may not be valid if building locations or other site plans are altered. Columbia West should review proposed drainage, building, and grading plans prior to final approval.

# 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 process. The primary geotechnical concerns associated with project development are shallow ground water and flowing surface water areas, shallow bedrock and steep slopes. The following text sections present design recommendations for the site.

## 6.1 Site Preparation and Grading

Vegetation should be cleared and topsoil stripped from areas identified for structural facilities 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 stripping depth is anticipated to be approximately 12 inches. The required stripping depth may increase in proposed demolition areas containing existing structures and paved surfaces. Stripped topsoil should be stockpiled prior to removal or placed in a separate designated location away from other material. The post-construction maximum depth of topsoil or landscaped fill placed or spread at any location onsite should not exceed one foot.

Previously disturbed soil, debris, or undocumented fill encountered during grading or construction activities should be removed completely and thoroughly. Existing structures to be demolished should be removed entirely. This includes old foundations, utilities, and associated unconsolidated soils. Abandoned septic systems, including tanks and drainfields, should be removed completely. Excavation areas should be backfilled with engineered structural fill. Wells should be properly abandoned and filled with bentonite, cement grout, or other suitable means in accordance with applicable state and federal regulations. Additional geotechnical assessment is recommended if structures are proposed in proximity to abandoned wells.

Trees and stumps should be removed from structural areas, individually and carefully. Roots should be completely removed, and the root cavity backfilled with competent engineered structural fill.

Test pits excavated during site exploration activities were backfilled loosely with onsite soils. These test pits should be located and properly backfilled with structural fill during site improvements construction.

Site grading activities should be performed in accordance with requirements specified in the *2006 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 an experienced geotechnical engineer or designated representative.

### 6.2 Engineered Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in the preceding text. Surface soils should then be scarified and re-compacted prior to placement of additional fill. 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 90 percent of the maximum dry density, obtained from the modified Proctor moisture-density relationship test (ASTM D1557), is recommended for structural fill placement. For engineered structural fill placed on sloped grades, the area should be benched to provide a horizontal surface for compaction.

Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D2922-91 and ASTM D3017-88 (93). Field compaction testing should be performed for each vertical foot of engineered fill placed. Engineered fill placement should be observed by an experienced geotechnical engineer or designated representative.

Engineered structural fill placement activities should be performed during dry summer months if possible. If fill placement occurs during dry weather conditions, clean fine-textured soils may be suitable for use as structural fill if adequately moisture-conditioned to achieve recommended compaction specifications. Because they are moisture-sensitive, fine-textured soils are nearly impossible to compact during wet weather conditions. If adequate compaction is not achievable with fine-textured 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 the geotechnical engineer prior to placement. Laboratory analyses should include particle-size gradation and modified 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. For fill slopes greater than six feet in height, the toe of the slope should be vertically keyed into existing subsurface soil. A typical fill slope cross-section is shown in Figure 4. Drainage implementations, including subdrains or perforated drain pipe trenches, may also be necessary in proximity to cut and fill slopes if seeps or springs are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by the geotechnical engineer during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion. Drainage recommendations are presented in Section 6.10, *Drainage*.

Final cut or fill slopes at the site should not exceed 2H:1V or 20 feet in 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. Figure 4 presents a minimum slope setback detail for structures. Please note that the minimum setback distance applies to graded cut or fill slopes. Specific setback distances for steep river terrace slopes were presented in Section 5.4, *Slope Setback Distance.*

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

## 6.4 Foundations

Foundations for proposed residential structures are anticipated to consist of shallow continuous perimeter footings or column spread footings. Typical building loads are not expected to exceed approximately 2 to 4 kips per foot for perimeter footings or 10 to 20 kips per column. Footing design should conform to requirements specified in the *2006 IBC, Table 1805.4.2, Footings Supporting Walls of Light-Frame Construction*, with exceptions as noted. Footings should bear upon firm native soil, engineered structural fill, or bedrock.

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 residential foundations placed upon firm competent native soil or compacted engineered structural fill is 1,500 psf. Bearing capacity may be increased by one-third for transient lateral forces such as seismic or wind. The modulus of subgrade reaction is estimated to be 250 psi/inch. The estimated coefficient of friction between recommended subgrade and in-place poured concrete is 0.35. Lateral forces may also be resisted by an assumed passive soil equivalent fluid pressure of 250 psf/f against embedded footings. The upper six inches of soil should be neglected in passive pressure calculations.

Footings should extend to a depth at least 18 inches below lowest adjacent exterior grade to provide adequate bearing capacity and protection against frost heave. If foundations are constructed during wet weather conditions, over-excavation and granular structural backfill is recommended. 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 undocumented fill or disturbed soil. Because soil is often heterogeneous and anisotropic, it is recommended that an experienced geotechnical engineer or designated representative observe foundation excavation and compaction of structural fill prior to placing forms or reinforcing bar to verify subgrade support conditions are as anticipated in this report.

### 6.4.1 Foundations on Bedrock and Soil

It is anticipated that proposed building foundations may extend across areas of bedrock and native soils or structural fill. Because settlement is not expected in areas where foundations are supported on bedrock, foundations spanning both bedrock and soil may be subjected to higher stresses and differential settlement. To mitigate potential adverse impacts of differential settlement at the soil and bedrock interface, it is recommended that foundation excavations in these areas be over-excavated and backfilled with a minimum of 18 inches of compacted crushed aggregate base placed and compacted in lifts in accordance with the specifications identified in Section 6.2, *Engineered Structural Fill*.

## 6.5 Settlement

Some total and differential footing displacement due to underlying soil settlement may be expected. For deep fill areas, total footing settlements may increase due to consolidation of fill material and underlying native soil. The resulting vertical displacement after loading may be due to elastic distortion, dissipation of excess pore pressure, or soil creep. Expansion of subgrade may also occur due to uplift rebound forces after unloading of native soils in deep cut areas. Increased potential for differential settlement may also be expected in proposed lots where the difference in fill depth between opposite building pad corners exceeds 10 feet.

## 6.6 Excavation

To install utilities and construct site improvements, subsurface excavation is anticipated. Due to the presence of basalt bedrock, drilling and blasting or specialized rock excavation techniques may be required. As indicated on the descriptive logs provided in Appendices B and C, difficult excavation was observed in TP-01 and SB-02. Refusal was encountered at a depth of approximately 9 feet in TP-01 and bedrock was encountered at a depth of 9 feet in SB-02. Bedrock is also exposed at the surface in some areas.

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

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

If significant excavation depths are proposed, Columbia West recommends a blasting contractor review information presented in this report and design a drill pattern. A pre-blast survey of adjacent properties and review of Clark County specifications for explosive blasting should also be conducted. It should be noted that excavation of fractured material may be difficult even after blasting.

Based upon laboratory analysis and in situ penetrometer testing, near-surface residual 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. This includes blasting and specialized rock excavation. Columbia West is not responsible for contractor activities and in no case should excavation or blasting be conducted in excess of all applicable local, state, and federal laws. This includes *WAC Chapter 296-155 Part N*.

### 6.7 Lateral Earth Pressure

Lateral earth pressure should be carefully considered for design of retaining walls. Hydrostatic pressure and additional surcharge loading should also be considered. Retained material may include engineered structural backfill or relatively undisturbed native soil. Structural wall backfill may consist of recompacted native soils or imported granular material. Backfill should be prepared and compacted to at least 90 percent of maximum dry density as determined by the modified Proctor test (ASTM D1557). Recommended parameters for lateral earth pressures for engineered structural fill should be determined after wall locations and dimensions are finalized.

Seismic forces for unrestrained walls may be calculated by superimposing a uniform later 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. Base coefficient of friction and bearing capacity for retaining wall design may be estimated based upon the values identified previously in Section 6.4, *Foundations*.

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 should be installed behind retaining walls. Geotextile filter fabric should be placed between the drain rock and backfill soil. Specifications for drainpipe design are presented in Section 6.10, *Drainage*. If walls cannot be gravity drained, saturated base conditions and/or applicable hydrostatic pressures should be assumed.

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

## 6.8 Seismic Design Considerations

According to the *National Seismic Hazard Maps, Open-File 02-420*, United States Geologic Survey (USGS), October 2002, the anticipated peak ground and maximum considered earthquake spectral response accelerations resulting from seismic activity for the subject site are summarized below in Table 2.

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

| | 10% Probability of Exceedance in 50 yrs | 2% Probability of Exceedance in 50 yrs |
|---|---|---|
| Peak Ground Acceleration | 0.19 g | 0.37 g |
| 0.2 sec Spectral Acceleration | 0.44 g | 0.87 g |
| 1.0 sec Spectral Acceleration | 0.15 g | 0.32g |

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 $F_a$ and $F_v$ as defined in 2006 IBC Tables 1615.5.3(1) and (2). The site coefficients are intended to more accurately characterize estimated peak ground and respective earthquake spectral response accelerations by considering site-specific soil characteristics and index properties. Based upon observed subsurface soil properties and review of the *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004) site soils may be represented by Site Class C as defined in *2006 IBC Table 1613.5.2*. This assessment is preliminary and based upon limited field exploration and research of existing published literature.

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

Identification of specific seismic response spectra for the site is beyond the scope of this investigation. If site structures are designed in accordance with recommendations specified in the *2006 IBC*, the potential for peak ground accelerations in excess of the adjusted and amplified values should be understood.

## 6.9 Liquefaction

Under certain conditions, a seismic event may induce soil liquefaction. Liquefaction, defined as the transformation of the behavior of a granular material from a solid to a liquid due to increased pore-water pressure and reduced effective stress, may occur when granular materials quickly compact under cyclic stresses caused by a seismic event. The effects of liquefaction may include immediate ground settlement, 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.

As defined by *Seed and Idriss (1982)*, potential for liquefaction is greatest if the following conditions are present:

- Fines content (material passing the no. 200 sieve) is less than 15 percent by weight.
- Liquid limit is less than 35 percent.
- Natural moisture content is greater than 0.9 times the liquid limit.

Based upon the results of the field investigation and laboratory analysis, soils at the site are generally medium stiff to stiff, contain a significant percentage of fines, and generally do no meet the criteria outlined above for soils susceptible to liquefaction. Therefore, the potential for liquefaction at the site is considered to be low.

### 6.10 Drainage

Shallow ground water, fine-textured soils, and areas of shallow bedrock indicate potential for reduced soil permeability and underscore the importance of proper drainage. At a minimum, site drainage should include surface water collection and conveyance to properly designed storm water management structures and facilities. Drainage design in general should conform to Clark County 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 the proposed residential structures. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from the foundation into the storm water system or approved discharge location.

Concentrated stormwater from roof drains should not be discharged near the top of the slopes. Stormwater from roof drains should not be allowed to collect and flow directly across the slopes. If concentrated storm or roof drain water must be conveyed toward the slopes, it should be collected and discharged by solid pipe to the base of the slopes. If discharge at the base of the slopes is not feasible, other mitigative design measures may be implemented to control erosion and limit instability associated with stormwater discharge. Such measures may include level spreaders or rip-rap channels. These methods of stormwater management and disposal will require additional geotechnical analysis and design. Therefore, if stormwater cannot be discharged at the base of the slopes, additional geotechnical assessment should be conducted.

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 Amoco 4545 or approved equivalent, with AOS between No. 70 and No. 100 sieve. The water permittivity should be greater than 1.5/sec. Figure 6 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 for portions of the site cut below surrounding grades. Shallow ground water, springs, or seeps should be conveyed via drainage channel or perforated pipe into the storm water management system. 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. Figure 7 presents a typical perforated drain pipe trench detail.

### 6.11 Storm Water Facility Construction

A small storm water management facility utilizing detain and release is proposed near the southeastern property corner. If earthen berms are utilized for above-ground storm water facility construction, a base key should be installed with minimum depth of H/3 and minimum width of W/2, where H is the total berm height and W is the total berm width. Figure 8 presents a typical storm water facility berm cross-section. The interior berm slope grade should not exceed 3H:1V and the top width of the berm in feet should be at least 10 + H/5. The berm should be constructed and compacted in lifts according to specifications identified in Section 6.2, *Engineered Structural Fill*. It is anticipated that onsite non-organic native soil will be suitable for use as structural berm fill, provided it is appropriately moisture-conditioned to achieve recommended compaction specifications. Composite samples of structural berm fill should be submitted to the geotechnical engineer for approval prior to construction.

Due to the anticipated limited magnitude of proposed site impervious surface, modified recommendations for small-scale storm water facilities may be necessary. A licensed geotechnical engineer should review final grading and earthwork plans for the storm water facility and discharge system prior to final design approval. The geotechnical engineer should also observe, test, and document earthwork and construction activities associated with the storm water facility.

### 6.12 Bituminous Asphalt and Portland Cement Concrete

Based upon preliminary correspondence with the client, the site may include new or improved asphalt concrete residential streets. Based upon analytical laboratory test results and field exploration, Columbia West recommends the general pavement design consist of a minimum of 8 inches of compacted crushed aggregate base overlain with a minimum of 3 inches of asphalt concrete pavement. This is a preliminary estimate based upon assumed subgrade modulus values and a traffic index representative of typical residential applications. If precise traffic data become available in the future, Columbia West can perform a specific flexible pavement design.

Columbia West has reviewed Clark County standards for public works construction and recommends adherence to identified pavement thickness sections if improvements to public roads are proposed for the site.

For dry weather road construction, road surface sections should bear upon competent subgrade consisting of scarified and compacted native soil or engineered structural fill. Wet weather road construction is discussed later in Section 6.13, *Wet Weather Construction Methods and Techniques*. Subgrade conditions should be evaluated and tested by a licensed geotechnical engineer or designated representative prior to placement of crushed aggregate base. Subgrade evaluation should include nuclear gauge density testing and wheel proof-roll observations conducted with a 12-cubic yard, double-axle dump truck or equivalent. Nuclear gauge density testing should be conducted at 250-foot intervals or as determined by the onsite geotechnical engineer. Subgrade soil should be compacted to at least 95 percent of 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 specifications.

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

### 6.13 Wet Weather Construction Methods and Techniques

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

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.

Pavement construction during wet weather conditions may require increased base thickness. Over-excavation may be necessary to provide a firm base upon which to place crushed aggregate. Geotextile filter fabric is also recommended. Crushed aggregate base should be installed in a single lift with trucks end-dumping from an advancing pad of granular fill. During extended wet periods, stripping activities may also need to be conducted from an advancing pad of granular fill. Once installed, the crushed aggregate base should be compacted with several passes from a static drum roller. A vibratory compactor is not recommended because it may further disturb the subgrade. Subdrains may also be necessary to provide subgrade drainage and maintain structural integrity.

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

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

### 6.14 Soil Erosion Potential

According to *Clark County Maps Online (http://gis.clark.wa.gov/ccgis/mol/property.htm),* the *Soil Survey of Clark County, Washington,* and field observations, the erosion hazard for most site soils is slight. However, near-surface soils in sloped areas along the southern and western property boundary may erode 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 environments may be controlled by application of strategically placed channels and small detention depressions with overflow pipes.

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

### 6.15 Soil Shrink/Swell Potential

The *Soil Survey of Clark County, Washington* (United States Department of Agriculture, Soil Conservation Service [USDA SCS], November 1972) indicates moderate potential for shrinking and swelling of native site soils. Based upon laboratory analysis, near-surface residual soils have a plasticity index of ranging from 13 to 26 and contain approximately 75 to 90 percent by weight passing the No. 200 sieve. This indicates moderate potential for soil shrinking or swelling. An experienced geotechnical engineer or designated representative should closely monitor placement and compaction activities if onsite soils are used as engineered structural fill. The potential for soil expansion can be minimized by properly controlling moisture content during fill placement.

### 6.16 Utility Installation

Utility installation at the site 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 ground water may result in accumulation of water within excavation zones and trenches. These areas should be dewatered in accordance with appropriate discharge regulations.

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 Clark County and the site geotechnical engineer. Native soils may be suitable for use as trench backfill in non-structural areas and should be evaluated on a case-by-case basis by the site geotechnical engineer. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). The remaining backfill should be compacted to at least 90 percent of maximum dry density as determined by the modified Proctor moisture-density test (ASTM D1557). 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 D2922-91 and ASTM D3017-88 (93). It is recommended that field compaction testing be performed at 250-foot intervals along the utility trench centerline at the surface and midpoint depth of the trench. Compaction frequency and specifications may be modified for non-structural areas in accordance with recommendations of the site geotechnical engineer.

## 7.0 CONCLUSION AND LIMITATIONS

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

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

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

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

Sincerely,

**COLUMBIA WEST ENGINEERING, Inc.**

Jason L. Ordway
Project Engineer

Lance V. Lehto, PE, MS
President

LANCE V. LEHTO
STATE OF WASHINGTON
36301
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES: 9-10-08

1-31-08

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

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

Evarts, Russell C., *Geologic Map of the Ridgefield Quadrangle, Clark and Cowlitz Counties, Washington*, Scientific Investigations Map MF-2844, U.S. Geological Survey, 2004.

Fiksdal, Allen, *Slope Stability of Clark County, Washington*, Open File Report 75-10, Washington State Department of Natural Resources, Division of Geology and Earth Resources, 1975.

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

*GeoStudio 2004*, Version 6.17, GEO-SLOPE International, Ltd., Calgary, Alberta, Canada, 1991-2005.

International Building Code: 2006 International Building Code, 2006 edition, International Code Council, 2006.

McCarthy, Kathleen A., and Anderson, Donald B., *Ground Water Data for the Portland Basin, Oregon and Washington*, Open File Report 90-126, United States Geological Survey, 1990.

McGee, Dale A., *Soil Survey of Clark County, Washington*, Soil Conservation Service, United States Department of Agriculture, November, 1972.

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

NAVFAC Design Manual 7.02, *Foundations and Earth Structures*, Naval Facilities Engineering Command, September, 1986.

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

National Seismic-Hazard Maps, Open File Report 02-420, United States Geological Survey, October, 2002.

NAVFAC Design Manual 7.02, *Foundations and Earth Structures*, Naval Facilities Engineering Command, September, 1986.

Palmer, Stephen P., et al, *Site Class Map of Clark County, Washington and Oregon*, Washington State Department of Natural Resources, Division of Geology and Earth Resources, September 2004.

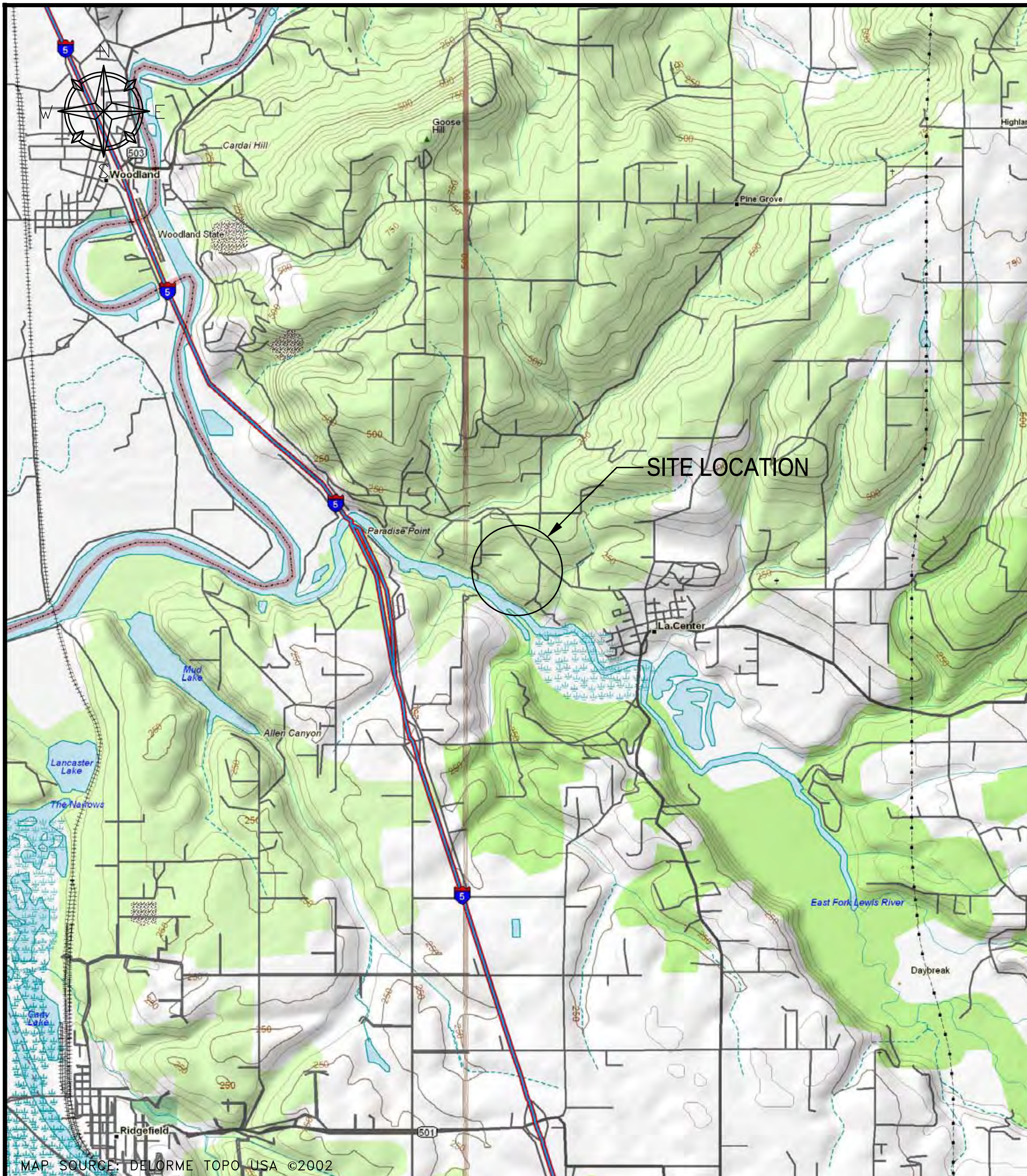
*Safety and Health Regulations for Construction*, 29 CFR Part 1926, Occupational Safety and Health Administration (OSHA), revised July 1, 2001.

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

Seed, H.B, and Idriss, I.M., *Ground Motions and Soil Liquefaction During Earthquakes*, Earthquake Engineering Research Institute, Berkeley, California, 1982.

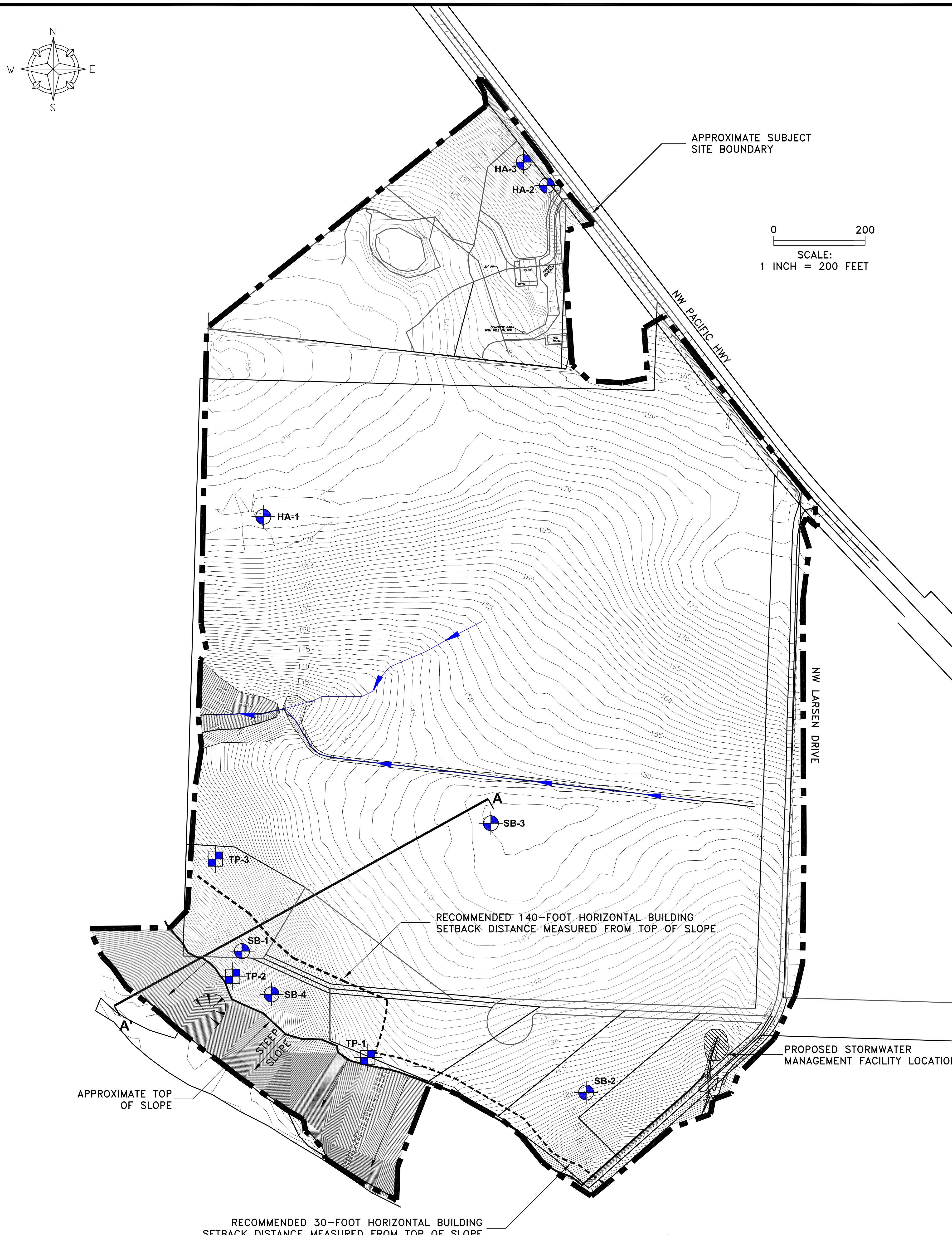
Wong, Ivan, et al, *Earthquake Scenario and Probabilistic Earthquake Ground Shaking Maps for the Portland, Oregon, Metropolitan Area*, IMS-16, Oregon Department of Geology and Mineral Industries, 2000.

# FIGURES




Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901

| | | | | | |
|---|---|---|---|---|---|
| Design: | Drawn: BCF | | | SITE LOCATION MAP | FIGURE 1 |
| Checked: LVL | Date: 12/07/07 | | | | |
| Client: RANDY GOODE | Rev | By | Date | GOODE PROPERTY<br>LA CENTER, WASHINGTON | |
| Job No.: 04133 | | | | | |
| CAD File: FIGURE 1 | | | | | |
| Scale: 1:50,000 | | | | | |




APPROXIMATE LOCATION OF TEST PITS

APPROXIMATE LOCATION OF SOIL BORINGS AND HAND-AUGERED EXPLORATIONS

APPROXIMATE LOCATION OF OBSERVED SURFACE WATER FLOW

FIELD VERIFIED SOIL SLUMP/ SLOPE FAILURE

NOTES:
1. SITE LOCATION: WEST OF NW LARSEN DRIVE AND NW PACIFIC HIGHWAY.
2. SITE IS APPROXIMATELY 52 ACRES IN SIZE.
3. DRAWING IS NOT TO SCALE.
4. BASE MAP PROVIDED BY MOSS.
5. TEST PIT LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
6. TEST PITS BACKFILLED LOOSELY WITH ON-SITE SOIL ON 10/14/05.
7. SOIL BORINGS BACKFILLED WITH BENTONITE ON 12/13/07.

Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901

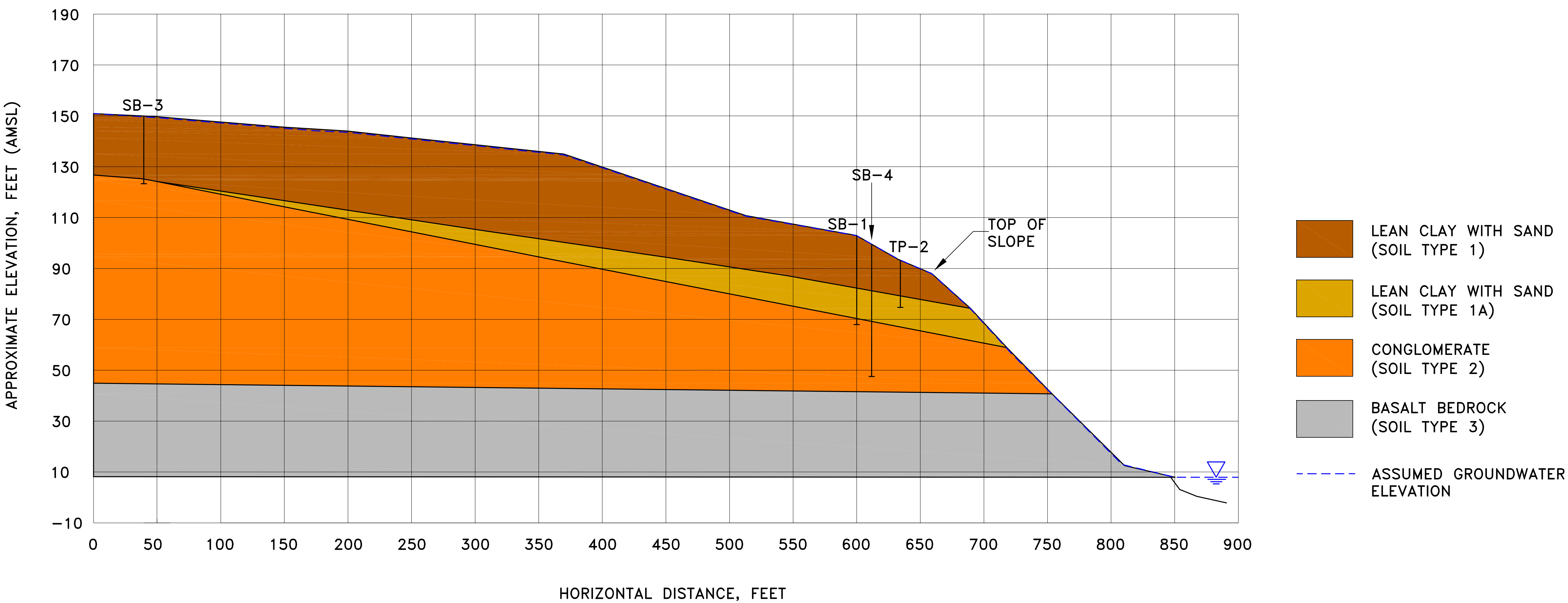
| | | | |
|---|---|---|---|
| Design: | Drawn: AJL | | |
| Checked: LVL | Date: 01/22/08 | | |
| Client: RANDY GOODE | Rev | By | Date |
| Job No: 04133 | | | |
| CAD File: FIGURE 2 | | | |
| Scale: NONE | | | |

SOIL BORING AND TEST PIT LOCATION MAP

GOODE PROPERTY
LA CENTER, WASHINGTON

FIGURE 2

# 04133, GOODE PROPERTY, CROSS SECTION A–A'




NOTE:
SUBSURFACE LITHOLOGY IS PROJECTED AND ASSUMED BASED ON BORING AND TEST PIT DATA AND MAY VARY SIGNIFICANTLY.


Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901


| | | | | |
|---|---|---|---|---|
| Design: | Drawn: AJL | | | |
| Checked: LVL | Date: 01/25/08 | | | |
| Client: RANDY GOODE | Rev | By | Date | |
| Job No: 04133 | | | | |
| CAD File: FIGURE 3 | | | | |
| Scale: NONE | | | | |

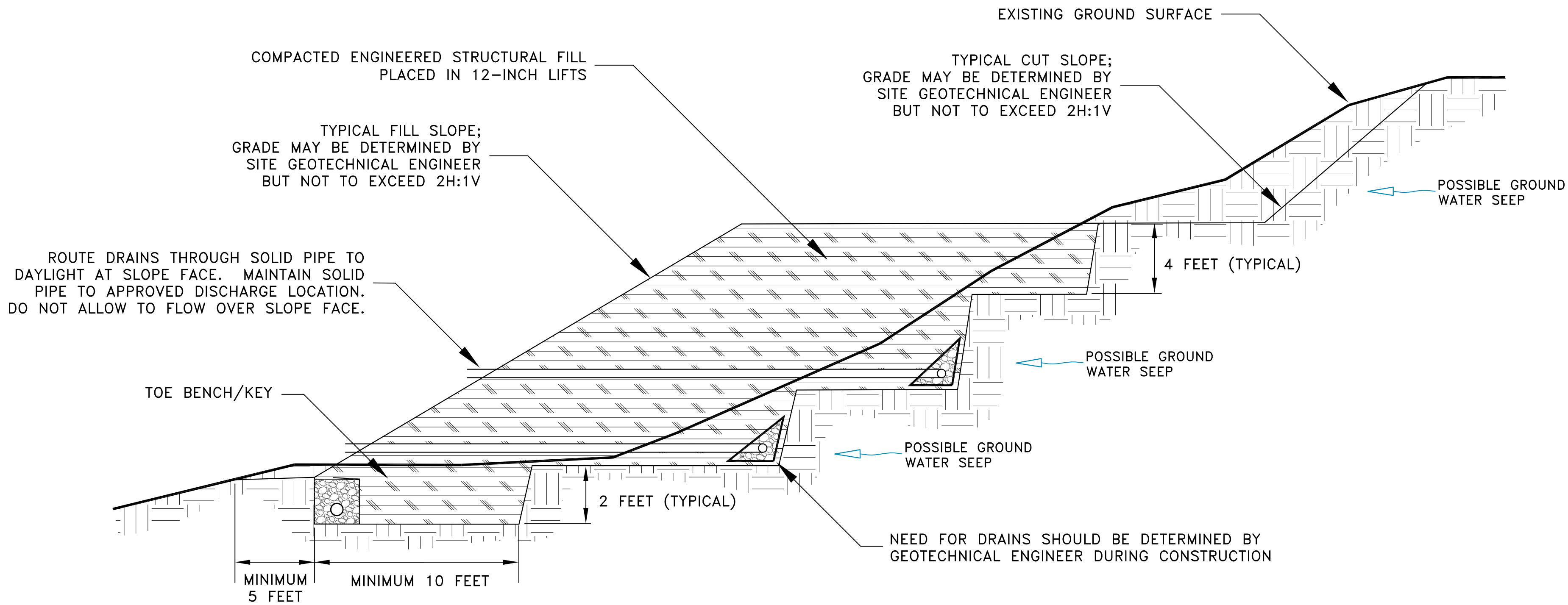
CROSS SECTION A–A'

GOODE PROPERTY

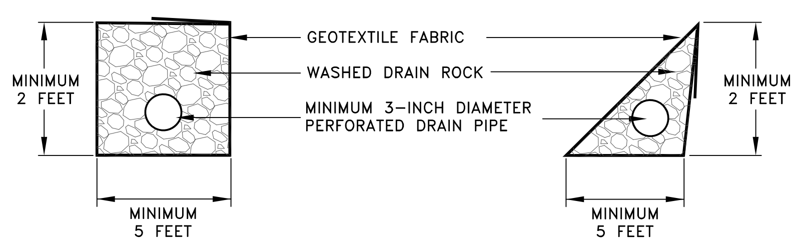
LA CENTER, WASHINGTON

FIGURE 3

# TYPICAL CUT AND FILL SLOPE CROSS-SECTION




TYPICAL DRAIN SECTION DETAIL




## DRAIN SPECIFICATIONS

GEOTEXTILE FABRIC SHALL CONSIST OF AMOCO 4545 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.

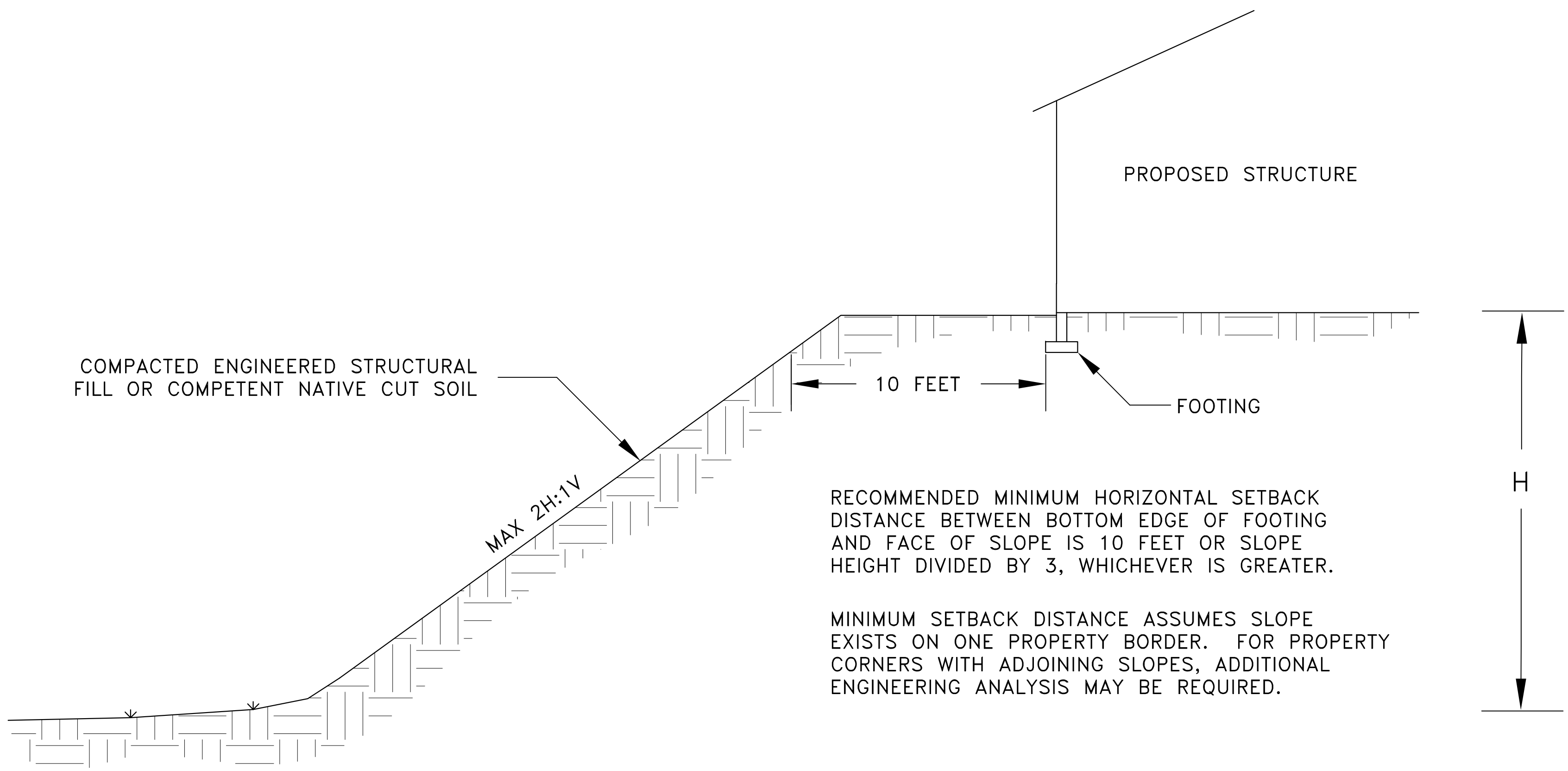
NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FILL AND CUT SLOPE SECTION, AND MAY NOT BE SITE-SPECIFIC.


Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901


| | | | | |
|---|---|---|---|---|
| Design: | Drawn: AJL | | | TYPICAL CUT AND FILL SLOPE CROSS-SECTION |
| Checked: LVL | Date: 01/25/08 | | | |
| Client: RANDY GOODE | Rev | By | Date | GOODE PROPERTY<br>LA CENTER, WASHINGTON |
| Job No: 04133 | | | | |
| CAD File: FIGURE 4 | | | | |
| Scale: NONE | | | | |

FIGURE 4

# MINIMUM FOUNDATION SLOPE SETBACK DETAIL FROM SMALL CUT OR FILL SLOPES

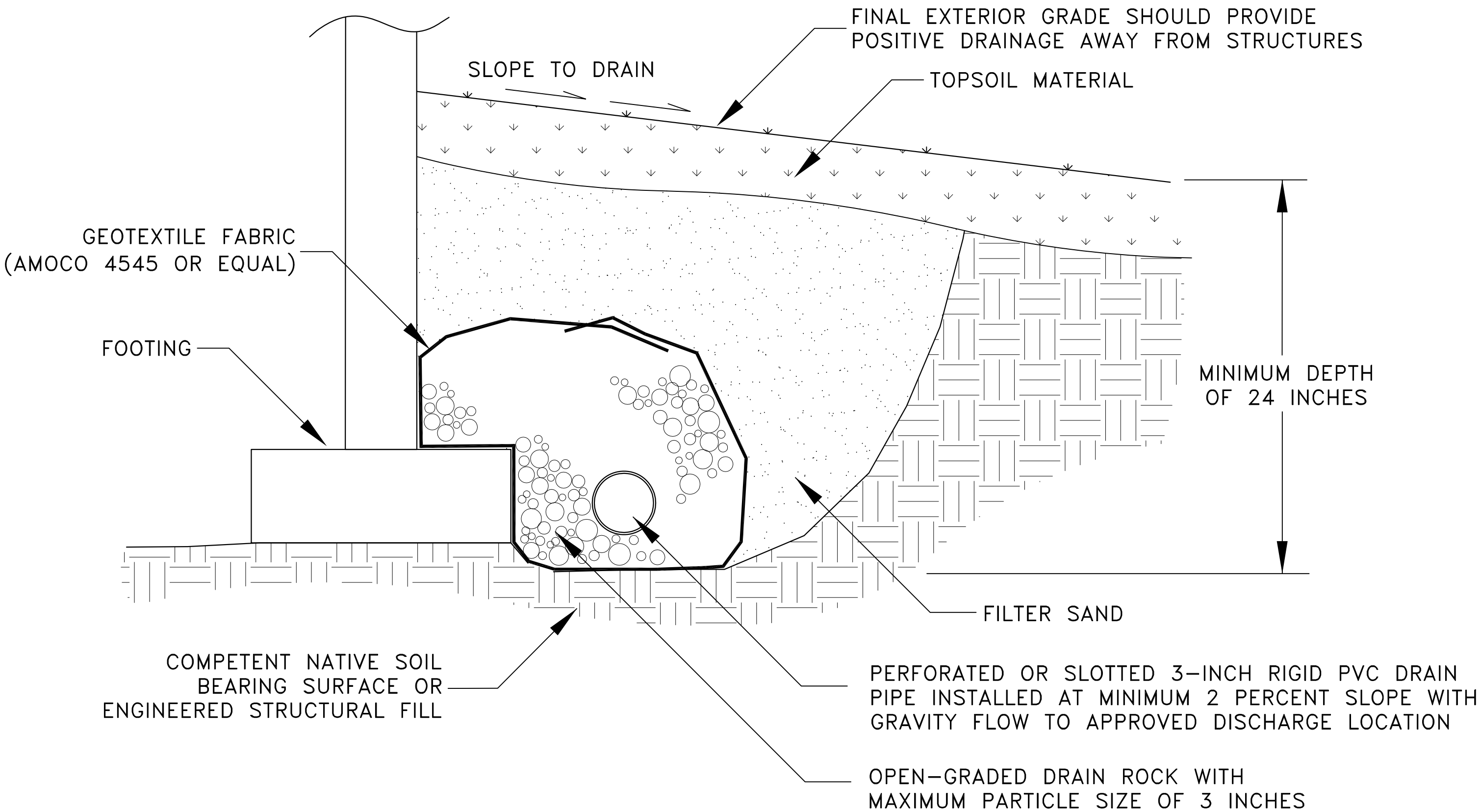



NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FOUNDATION SETBACK DETAIL FOR SMALL CUT OR FILL SLOPES, AND MAY NOT BE SITE-SPECIFIC.
4. ALTERNATE SLOPE SETBACK DISTANCES ARE APPLICABLE ADJACENT TO HIGH, STEEP NATIVE SLOPES ONSITE. REFER TO MAIN REPORT TEXT.


Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901


| | | | |
|---|---|---|---|
| Design: | Drawn: AJL | MINIMUM FOUNDATION SLOPE SETBACK DETAIL | FIGURE 5 |
| Checked: LVL | Date: 01/25/08 | | |
| Client: RANDY GOODE | Rev / By / Date | GOODE PROPERTY | |
| Job No: 04133 | | LA CENTER, WASHINGTON | |
| CAD File: FIGURE 5 | | | |
| Scale: NONE | | | |

# TYPICAL PERIMETER FOOTING DRAIN DETAIL




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


Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901

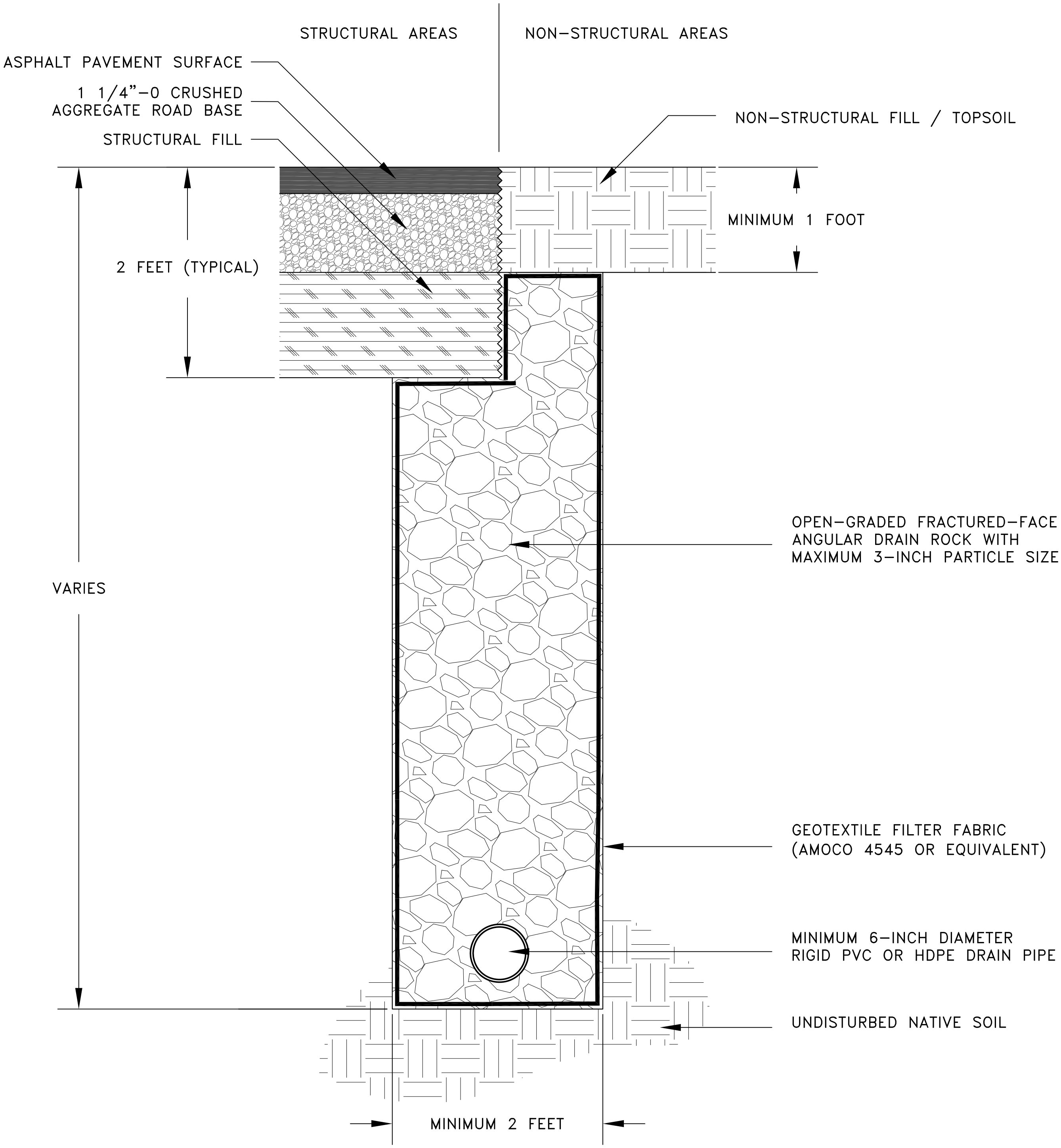

| | | | | |
|---|---|---|---|---|
| Design: | Drawn: JGH | | | |
| Checked: LVL | Date: 01/25/08 | | | |
| Client: RANDY GOODE | Rev | By | Date | |
| Job No: 04133 | | | | |
| CAD File: FIGURE 6 | | | | |
| Scale: NONE | | | | |

TYPICAL PERIMETER FOOTING DRAIN DETAIL

GOODE PROPERTY
LA CENTER, WASHINGTON

FIGURE
6

# TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL

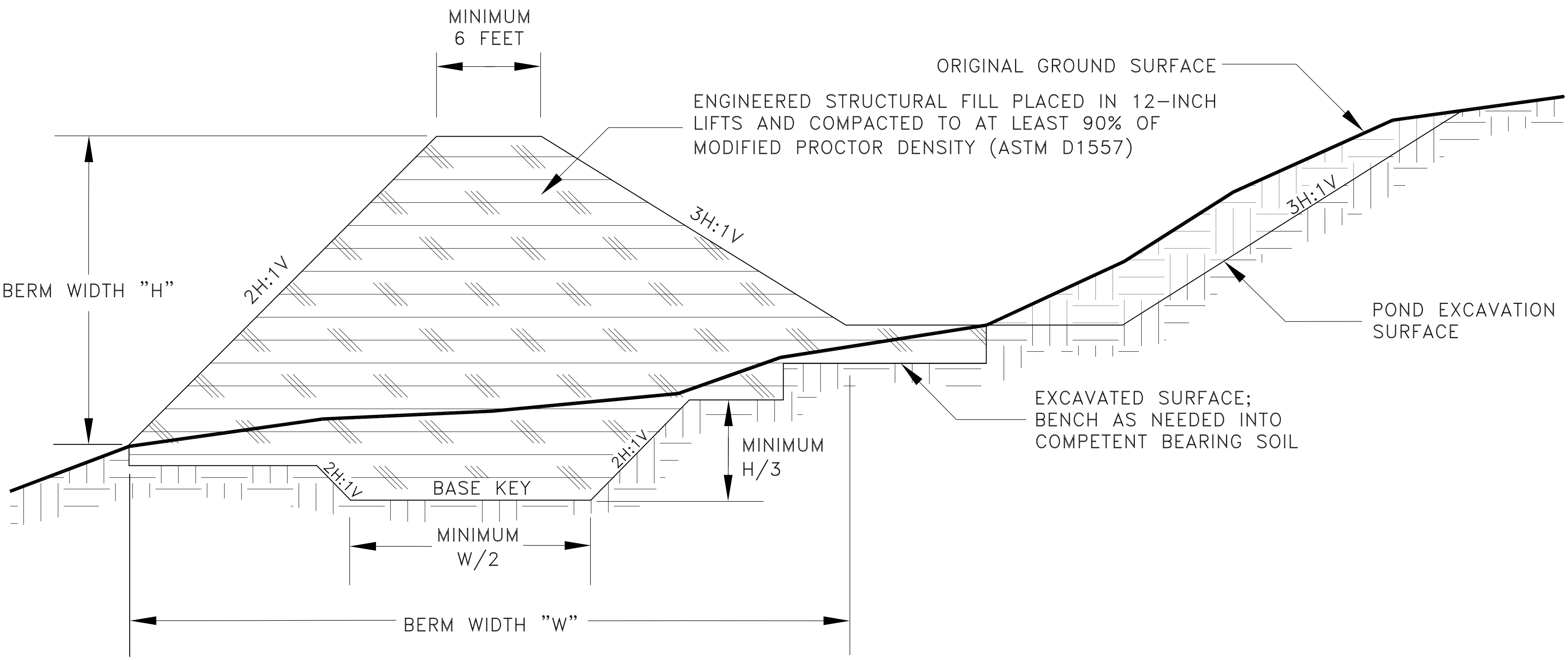



NOTE: LOCATION, INVERT ELEVATION, DEPTH OF TRENCH, AND EXTENT OF PERFORATED PIPE REQUIRED MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER DURING CONSTRUCTION BASED UPON FIELD OBSERVATION AND SITE-SPECIFIC SOIL CONDITIONS.


Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901


| | | | |
|---|---|---|---|
| Design: | Drawn: AJL | TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL | FIGURE 7 |
| Checked: LVL | Date: 01/25/08 | | |
| Client: RANDY GOODE | Rev / By / Date | GOODE PROPERTY | |
| Job No: 04133 | | LA CENTER, WASHINGTON | |
| CAD File: FIGURE 7 | | | |
| Scale: NONE | | | |

TYPICAL STORMWATER FACILITY CROSS-SECTION




NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL, GENERAL STORM WATER BERM SECTION, AND MAY NOT BE SITE-SPECIFIC.

Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901

| | | | | |
|---|---|---|---|---|
| Design: | Drawn: JGH | | | TYPICAL STORMWATER FACILITY CROSS-SECTION |
| Checked: LVL | Date: 10/02/08 | | | |
| Client: GOODE | Rev | By | Date | GOODE PROPERTY |
| Job No: 04133 | | | | LA CENTER, WASHINGTON |
| CAD File: FIGURE 8 | | | | |
| Scale: NONE | | | | |

FIGURE 8

# APPENDIX A
## ANALYTICAL LABORATORY TEST REPORTS

**Columbia West Engineering, Inc.**
Geotechnical Engineering and Environmental Consulting Services
11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# PARTICLE-SIZE ANALYSIS REPORT

| PROJECT | CLIENT | PROJECT NO. | LAB ID |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | S07-829 |
| | | REPORT DATE: 01/02/08 | FIELD ID: SB1.1 |
| | | DATE SAMPLED: 12/13/07 | SAMPLED BY: JLO |

## MATERIAL DATA

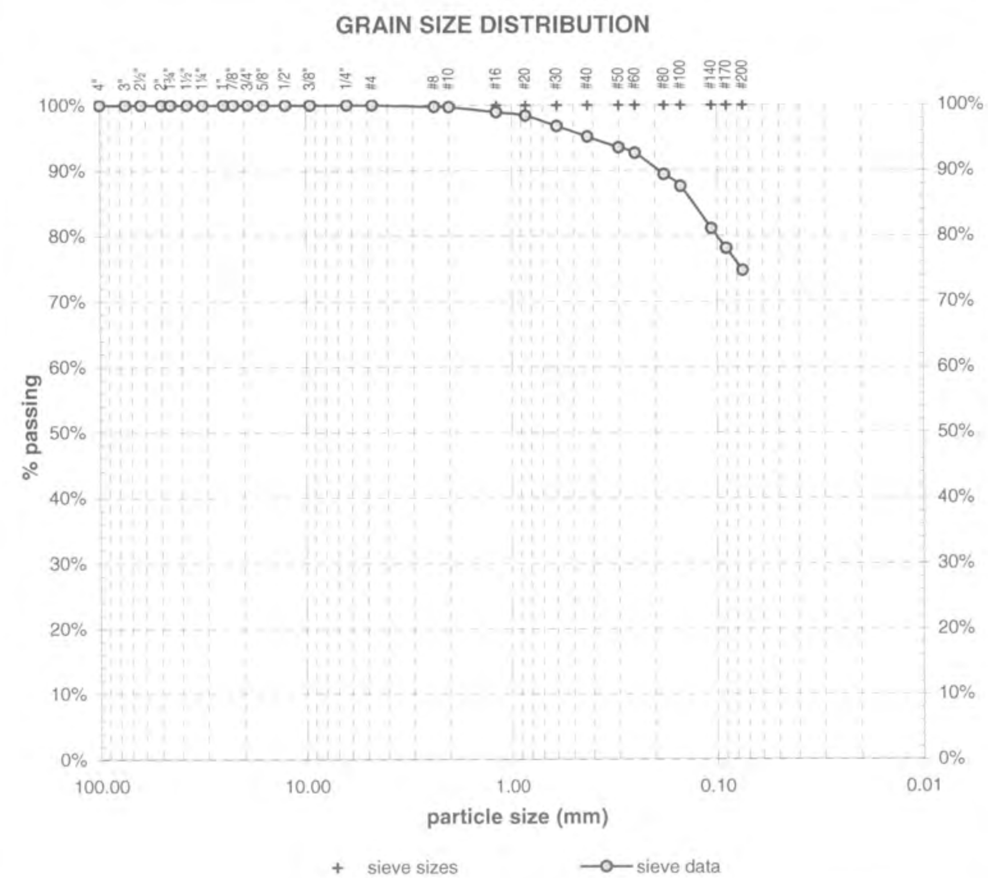
| MATERIAL SAMPLED | MATERIAL SOURCE | USCS SOIL TYPE |
|---|---|---|
| brown Lean CLAY with Sand | Soil Boring SB-01<br>depth = 2.5 feet | CL, Lean Clay with Sand |
| SPECIFICATIONS: none | | AASHTO SOIL TYPE: A-6(8) |

## LABORATORY TEST DATA

| LABORATORY EQUIPMENT | TEST PROCEDURE |
|---|---|
| Rainhart "Mary Ann" Sifter 637 | ASTM D6913, D2487 |

### ADDITIONAL DATA

| | | | |
|---|---|---|---|
| natural moisture content = | 24.0% | coefficient of curvature, $C_C$ = | n/a |
| liquid limit = | 31 | coefficient of uniformity, $C_U$ = | n/a |
| plastic limit = | 18 | effective size, $D_{(10)}$ = | n/a |
| plasticity index = | 13 | $D_{(30)}$ = | n/a |
| fineness modulus = | n/a | $D_{(60)}$ = | n/a |




### SIEVE DATA

% gravel = 0.0%
% sand = 25.2%
% silt and clay = 74.8%

| | SIEVE SIZE US | mm | PERCENT PASSING SIEVE act. | 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 | | 99.8% | | |
| | #10 | 2.00 | 99.7% | | | |
| | #16 | 1.18 | | 98.9% | | |
| | #20 | 0.850 | 98.4% | | | |
| | #30 | 0.600 | | 96.8% | | |
| | #40 | 0.425 | 95.1% | | | |
| | #50 | 0.300 | | 93.5% | | |
| | #60 | 0.250 | 92.7% | | | |
| | #80 | 0.180 | | 89.4% | | |
| | #100 | 0.150 | 87.6% | | | |
| | #140 | 0.106 | | 81.2% | | |
| | #170 | 0.090 | | 78.1% | | |
| | #200 | 0.075 | 74.8% | | | |

| DATE TESTED | TESTED BY |
|---|---|
| 12/19/07 | SMJ |

COLUMBIA WEST ENGINEERING, INC. authorized signature

Columbia West Engineering, Inc.
Geotechnical Engineering and Environmental Consulting Services

11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# ATTERBERG LIMITS REPORT

| PROJECT | CLIENT | PROJECT NO. | LAB ID |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | S07-829 |
| | | REPORT DATE<br>01/02/08 | FIELD ID<br>SB1.1 |
| | | DATE SAMPLED<br>12/13/07 | SAMPLED BY<br>JLO |

## MATERIAL DATA

| MATERIAL SAMPLED | MATERIAL SOURCE | USCS SOIL TYPE |
|---|---|---|
| brown Lean CLAY with Sand | Soil Boring SB-01<br>depth = 2.5 feet | CL, Lean Clay with Sand |

## LABORATORY TEST DATA

| LABORATORY EQUIPMENT | TEST PROCEDURE |
|---|---|
| Liquid Limit Machine, Plastic Limit Roller | ASTM D4318 |

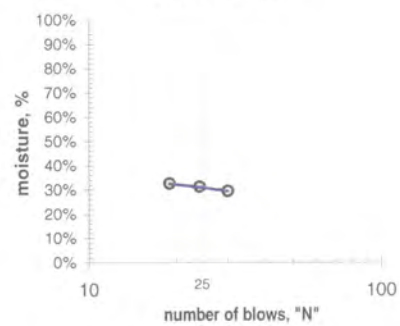
### ATTERBERG LIMITS

| | |
|---|---|
| liquid limit = | 31 |
| plastic limit = | 18 |
| plasticity index = | 13 |

### LIQUID LIMIT DETERMINATION

| | ❶ | ❷ | ❸ | ❹ |
|---|---|---|---|---|
| wet soil + pan weight, g = | 38.54 | 38.46 | 35.26 | |
| dry soil + pan weight, g = | 34.47 | 34.25 | 31.64 | |
| pan weight, g = | 20.67 | 20.82 | 20.54 | |
| N (blows) = | 30 | 24 | 19 | |
| moisture, % = | 29.5 % | 31.4 % | 32.6 % | |

### LIQUID LIMIT



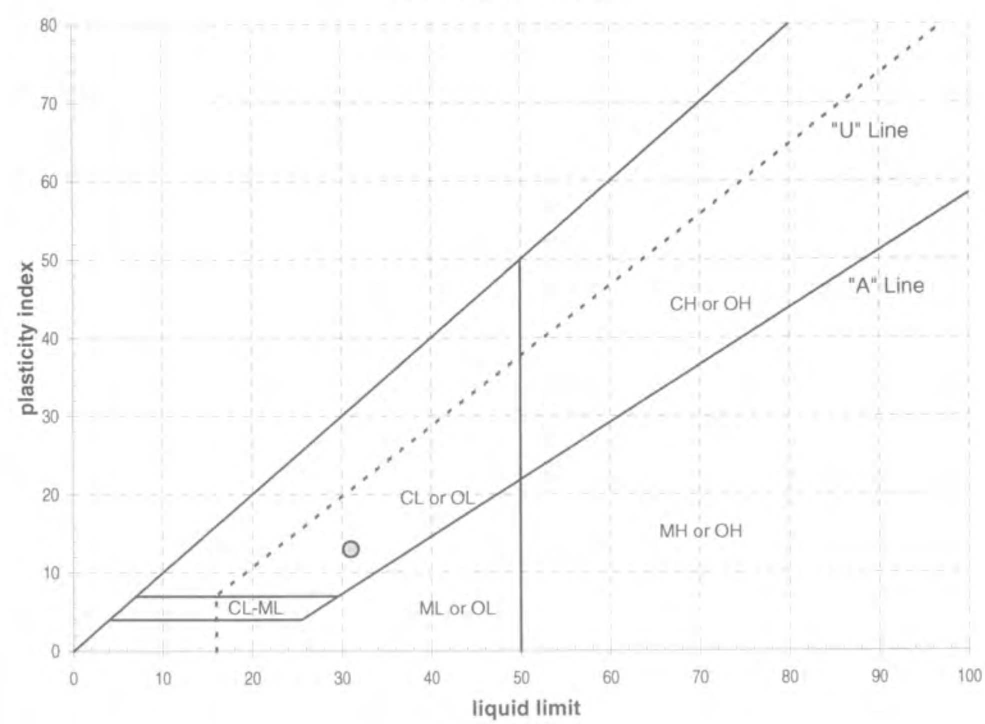

### SHRINKAGE

| | |
|---|---|
| shrinkage limit = | n/a |
| shrinkage ratio = | n/a |

### PLASTIC LIMIT DETERMINATION

| | ❶ | ❷ | ❸ | ❹ |
|---|---|---|---|---|
| wet soil + pan weight, g = | 29.63 | 30.64 | | |
| dry soil + pan weight, g = | 28.30 | 29.15 | | |
| pan weight, g = | 20.80 | 20.76 | | |
| moisture, % = | 17.7 % | 17.8 % | | |

### PLASTICITY CHART




### ADDITIONAL DATA

| | |
|---|---|
| % gravel = | 0.0% |
| % sand = | 25.2% |
| % silt and clay = | 74.8% |
| % silt = | n/a |
| % clay = | n/a |
| moisture content = | 24.0% |

| DATE TESTED | TESTED BY |
|---|---|
| 12/19/07 | SMJ |

COLUMBIA WEST ENGINEERING, INC. authorized signature

Columbia West Engineering, Inc.
Geotechnical Engineering and Environmental Consulting Services
11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# PARTICLE-SIZE ANALYSIS REPORT

| PROJECT | CLIENT | PROJECT NO. | LAB ID |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | S07-830 |
| | | REPORT DATE: 01/14/08 | FIELD ID: SB1.8 |
| | | DATE SAMPLED: 12/13/07 | SAMPLED BY: JLO |

## MATERIAL DATA

| MATERIAL SAMPLED | MATERIAL SOURCE | USCS SOIL TYPE |
|---|---|---|
| gray with bluish gray and brown mottling<br>Lean CLAY with Sand | Soil Boring SB-01<br>depth = 30 to 31 feet | CL, Lean Clay with Sand |
| SPECIFICATIONS: none | | AASHTO SOIL TYPE: A-6(10) |

## LABORATORY TEST DATA

| LABORATORY EQUIPMENT | TEST PROCEDURE |
|---|---|
| Rainhart "Mary Ann" Sifter 637 | ASTM D6913, D2487 |

### ADDITIONAL DATA

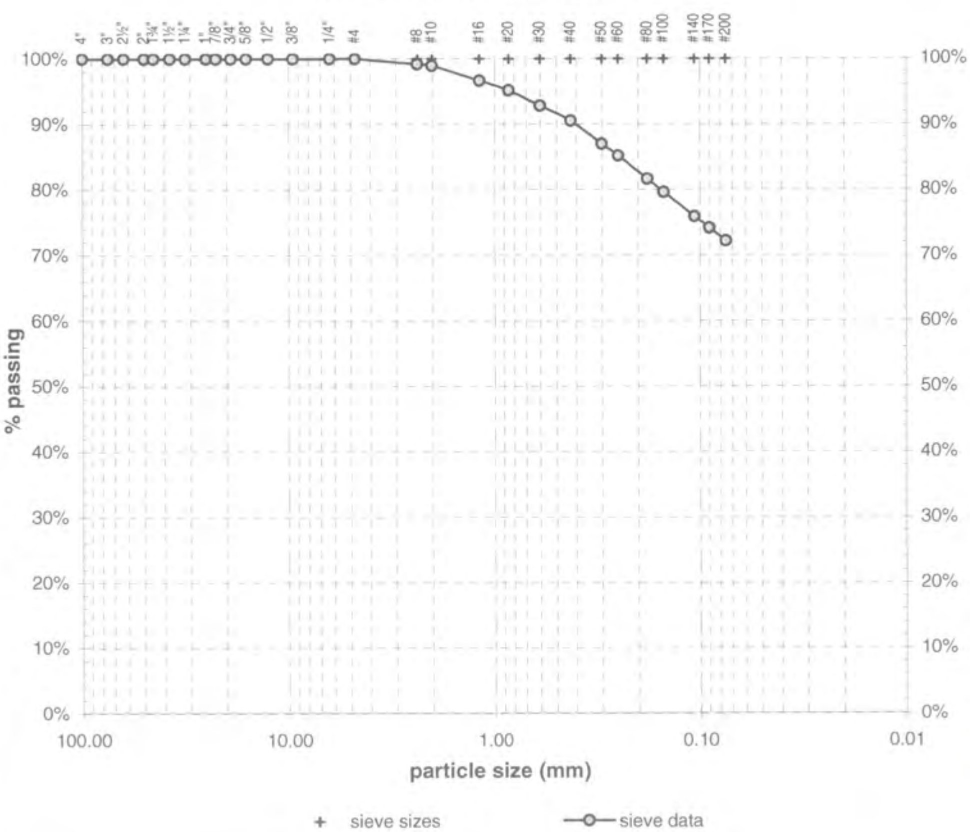
| | | | |
|---|---|---|---|
| natural moisture content = | 29.0% | coefficient of curvature, $C_C$ = | n/a |
| liquid limit = | 33 | coefficient of uniformity, $C_U$ = | n/a |
| plastic limit = | 17 | effective size, $D_{(10)}$ = | n/a |
| plasticity index = | 16 | $D_{(30)}$ = | n/a |
| fineness modulus = | n/a | $D_{(60)}$ = | n/a |

### SIEVE DATA

% gravel = 0.0%
% sand = 27.8%
% silt and clay = 72.2%

| | SIEVE SIZE US | SIEVE SIZE mm | PERCENT PASSING SIEVE act. | PERCENT PASSING SIEVE interp. | SPECS max | SPECS min |
|---|---|---|---|---|---|---|
| GRAVEL | 6.00" | 150.0 | | 100.0% | | |
| | 4.00" | 100.0 | | 100.0% | | |
| | 3.00" | 75.0 | | 100.0% | | |
| | 2.50" | 63.0 | | 100.0% | | |
| | 2.00" | 50.0 | | 100.0% | | |
| | 1.75" | 45.0 | | 100.0% | | |
| | 1.50" | 37.5 | | 100.0% | | |
| | 1.25" | 31.5 | | 100.0% | | |
| | 1.00" | 25.0 | | 100.0% | | |
| | 7/8" | 22.4 | | 100.0% | | |
| | 3/4" | 19.0 | | 100.0% | | |
| | 5/8" | 16.0 | | 100.0% | | |
| | 1/2" | 12.5 | | 100.0% | | |
| | 3/8" | 9.50 | | 100.0% | | |
| | 1/4" | 6.30 | | 100.0% | | |
| | #4 | 4.75 | 100.0% | | | |
| SAND | #8 | 2.36 | | 99.1% | | |
| | #10 | 2.00 | 98.9% | | | |
| | #16 | 1.18 | | 96.6% | | |
| | #20 | 0.850 | 95.2% | | | |
| | #30 | 0.600 | | 92.8% | | |
| | #40 | 0.425 | 90.5% | | | |
| | #50 | 0.300 | | 87.0% | | |
| | #60 | 0.250 | 85.2% | | | |
| | #80 | 0.180 | | 81.6% | | |
| | #100 | 0.150 | 79.7% | | | |
| | #140 | 0.106 | | 75.9% | | |
| | #170 | 0.090 | | 74.2% | | |
| | #200 | 0.075 | 72.2% | | | |

| DATE TESTED | TESTED BY |
|---|---|
| 01/11/07 | EJC |




COLUMBIA WEST ENGINEERING, INC. authorized signature

**Columbia West Engineering, Inc.**
Geotechnical Engineering and Environmental Consulting Services

11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# ATTERBERG LIMITS REPORT

| PROJECT | CLIENT | PROJECT NO. | LAB ID |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | S07-830 |
| | | REPORT DATE: 01/14/08 | FIELD ID: SB1.8 |
| | | DATE SAMPLED: 12/13/07 | SAMPLED BY: JLO |

## MATERIAL DATA

| MATERIAL SAMPLED | MATERIAL SOURCE | USCS SOIL TYPE |
|---|---|---|
| gray with bluish gray and brown mottling<br>Lean CLAY with Sand | Soil Boring SB-01<br>depth = 30 to 31 feet | CL, Lean Clay with Sand |

## LABORATORY TEST DATA

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

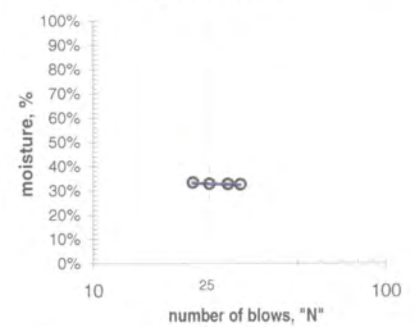
### ATTERBERG LIMITS

liquid limit = 33
plastic limit = 17
plasticity index = 16

### LIQUID LIMIT DETERMINATION

| | ❶ | ❷ | ❸ | ❹ |
|---|---|---|---|---|
| wet soil + pan weight, g = | 54.58 | 57.35 | 54.60 | 56.46 |
| dry soil + pan weight, g = | 46.24 | 48.32 | 46.22 | 47.53 |
| pan weight, g = | 20.70 | 20.77 | 20.73 | 20.82 |
| N (blows) = | 32 | 29 | 25 | 22 |
| moisture, % = | 32.7 % | 32.8 % | 32.9 % | 33.4 % |

### LIQUID LIMIT



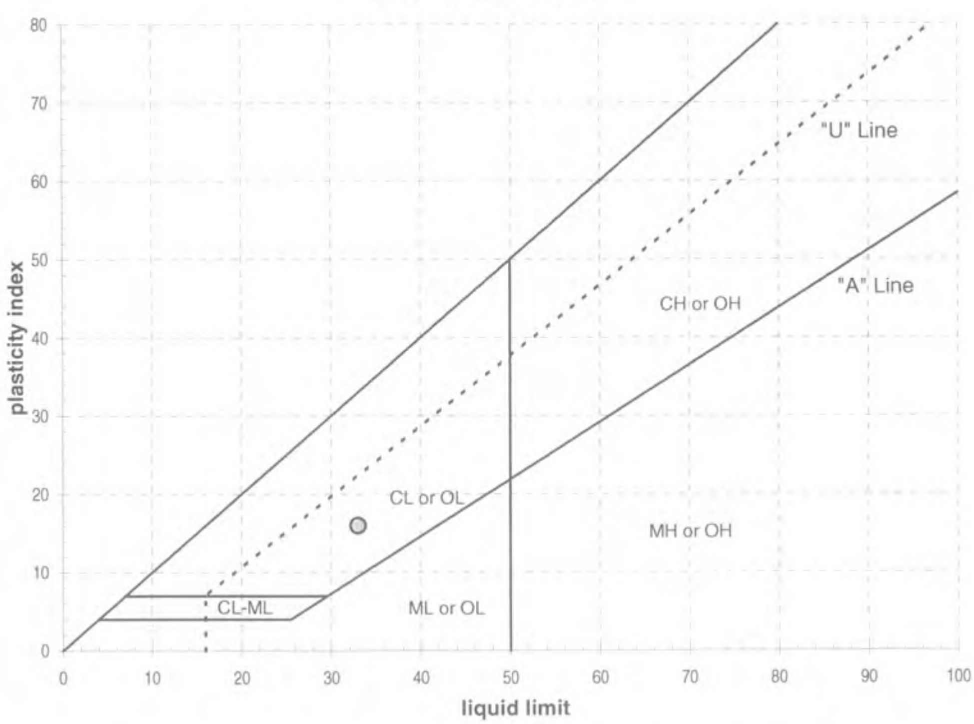

### SHRINKAGE

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

### PLASTIC LIMIT DETERMINATION

| | ❶ | ❷ | ❸ | ❹ |
|---|---|---|---|---|
| wet soil + pan weight, g = | 30.97 | 30.82 | | |
| dry soil + pan weight, g = | 29.49 | 29.37 | | |
| pan weight, g = | 20.57 | 20.68 | | |
| moisture, % = | 16.6 % | 16.7 % | | |

### PLASTICITY CHART




### ADDITIONAL DATA

| | |
|---|---|
| % gravel = | 0.0% |
| % sand = | 27.8% |
| % silt and clay = | 72.2% |
| % silt = | n/a |
| % clay = | n/a |
| moisture content = | 29.0% |

| DATE TESTED | TESTED BY |
|---|---|
| 01/11/08 | EJC |

COLUMBIA WEST ENGINEERING, INC. authorized signature

CWE-s14-r11/07

**Columbia West Engineering, Inc.**
Geotechnical Engineering and Environmental Consulting Services

11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# DIRECT SHEAR REPORT

| PROJECT | CLIENT | PROJECT NO. | LAB ID |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | S07-830 |
| | | **REPORT DATE** | **FIELD ID** |
| | | 01/14/08 | SB1.8 |
| | | **DATE SAMPLED** | **SAMPLED BY** |
| | | 12/13/07 | JLO |

## MATERIAL DATA

| MATERIAL SAMPLED | MATERIAL SOURCE | USCS SOIL TYPE |
|---|---|---|
| gray with bluish gray and brown mottling<br>Lean CLAY with Sand | Soil Boring SB-01<br>depth = 30 to 31 feet | CL, Lean Clay with Sand |

## LABORATORY TEST DATA

| LABORATORY EQUIPMENT | TEST PROCEDURE |
|---|---|
| ELE International Pneumatic Direct/Residual Shear Apparatus 26-2121/02 | ASTM D3080 |

### SAMPLE AND TEST DATA

| | PARAMETER | LOAD 1 | LOAD 2 | LOAD 3 |
|---|---|---|---|---|
| initial | initial thickness, in | 1.139 | 1.206 | 1.246 |
| | initial diameter, in | 2.399 | 2.397 | 2.392 |
| | dry weight, g | 127.9 | 132.9 | 135.3 |
| | initial wet density, pcf | 120.7 | 119.8 | 119.8 |
| | initial dry density, pcf | 94.6 | 92.9 | 91.9 |
| | initial moisture content, % | 27.7% | 29.0% | 30.3% |
| | initial void ratio | 0.748 | 0.780 | 0.799 |
| | initial saturation | 0.980 | 0.985 | 1.005 |
| test | displacement rate, in/min | 0.001 | 0.001 | 0.001 |
| | normal stress, tsf | 1.00 | 2.00 | 4.00 |
| | peak shear stress, tsf | 0.95 | 1.47 | 2.41 |
| | vertical deformation at peak shear, in | 0.749 | 0.010 | 0.034 |
| | horizontal deformation at peak shear, in | 0.084 | 0.093 | 0.225 |
| final | final wet density, pcf | 121.8 | 121.3 | 121.5 |
| | final dry density, pcf | 94.7 | 94.1 | 95.0 |
| | final moisture content, % | 28.6% | 29.0% | 27.9% |
| | final void ratio | 0.745 | 0.757 | 0.741 |
| | final saturation | 1.017 | 1.013 | 0.999 |

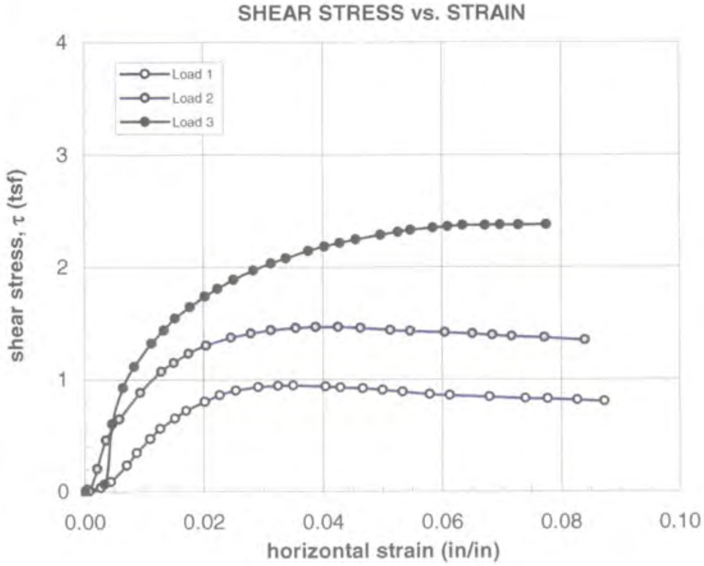
### SUMMARY

| load | applied normal stress, $\sigma$ (tsf) | shear stress at failure, $\tau$ (tsf) |
|---|---|---|
| 1 | 1.0 | 0.95 |
| 2 | 2.0 | 1.47 |
| 3 | 4.0 | 2.41 |

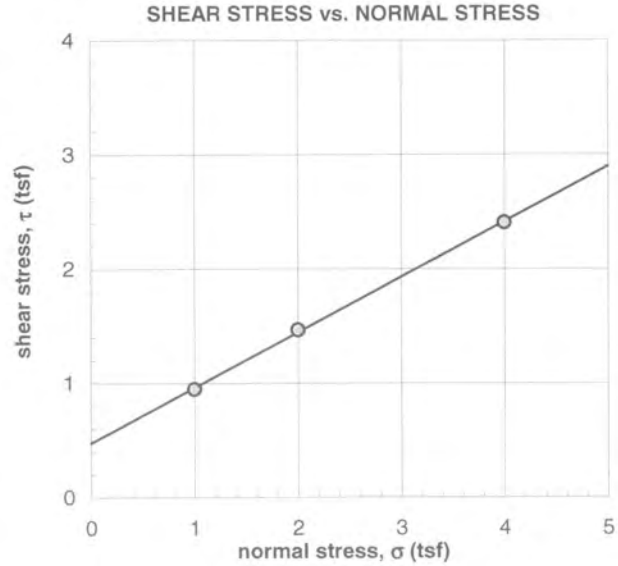
friction angle, $\phi$ = 25.8 °

cohesion, c = 956 psf

**SHEAR STRESS vs. STRAIN**




**SHEAR STRESS vs. NORMAL STRESS**




| DATE TESTED | TESTED BY |
|---|---|
| 01/09/08 | JJC |

COLUMBIA WEST ENGINEERING, INC. authorized signature

CWE-s16-r11/07

**Columbia West Engineering, Inc.**
Geotechnical Engineering and Environmental Consulting Services

11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# PARTICLE-SIZE ANALYSIS REPORT

| PROJECT | CLIENT | PROJECT NO. | LAB ID |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | S07-831 |
| | | REPORT DATE: 01/02/08 | FIELD ID: SB3.4 |
| | | DATE SAMPLED: 12/13/07 | SAMPLED BY: JLO |

## MATERIAL DATA

| MATERIAL SAMPLED | MATERIAL SOURCE | USCS SOIL TYPE |
|---|---|---|
| light brown Lean CLAY | Soil Boring SB-03<br>depth = 10 feet | CL, Lean Clay |
| SPECIFICATIONS: none | | AASHTO SOIL TYPE: A-7-6(20) |

## LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter 637

TEST PROCEDURE
ASTM D6913, D2487

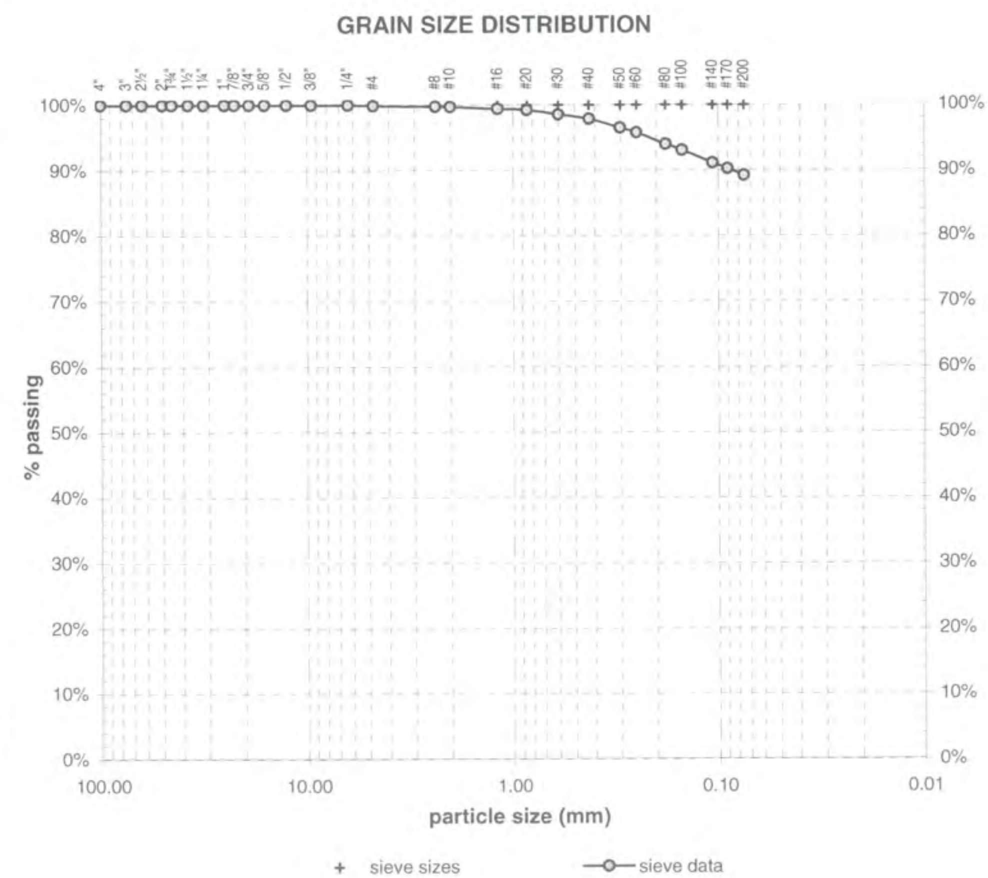
### ADDITIONAL DATA

| | | | |
|---|---|---|---|
| natural moisture content = | 33.9% | coefficient of curvature, $C_C$ = | n/a |
| liquid limit = | 41 | coefficient of uniformity, $C_U$ = | n/a |
| plastic limit = | 19 | effective size, $D_{(10)}$ = | n/a |
| plasticity index = | 22 | $D_{(30)}$ = | n/a |
| fineness modulus = | n/a | $D_{(60)}$ = | n/a |

### SIEVE DATA

% gravel = 0.1%
% sand = 10.7%
% silt and clay = 89.2%

| | SIEVE SIZE US | SIEVE SIZE mm | PERCENT PASSING SIEVE act. | PERCENT PASSING SIEVE interp. | SPECS max | SPECS min |
|---|---|---|---|---|---|---|
| GRAVEL | 6.00" | 150.0 | | 100.0% | | |
| | 4.00" | 100.0 | | 100.0% | | |
| | 3.00" | 75.0 | | 100.0% | | |
| | 2.50" | 63.0 | | 100.0% | | |
| | 2.00" | 50.0 | | 100.0% | | |
| | 1.75" | 45.0 | | 100.0% | | |
| | 1.50" | 37.5 | | 100.0% | | |
| | 1.25" | 31.5 | | 100.0% | | |
| | 1.00" | 25.0 | | 100.0% | | |
| | 7/8" | 22.4 | | 100.0% | | |
| | 3/4" | 19.0 | | 100.0% | | |
| | 5/8" | 16.0 | | 100.0% | | |
| | 1/2" | 12.5 | | 100.0% | | |
| | 3/8" | 9.50 | | 100.0% | | |
| | 1/4" | 6.30 | 100.0% | | | |
| | #4 | 4.75 | 99.9% | | | |
| SAND | #8 | 2.36 | | 99.7% | | |
| | #10 | 2.00 | 99.7% | | | |
| | #16 | 1.18 | | 99.4% | | |
| | #20 | 0.850 | 99.2% | | | |
| | #30 | 0.600 | | 98.5% | | |
| | #40 | 0.425 | 97.9% | | | |
| | #50 | 0.300 | | 96.5% | | |
| | #60 | 0.250 | 95.7% | | | |
| | #80 | 0.180 | | 94.0% | | |
| | #100 | 0.150 | 93.1% | | | |
| | #140 | 0.106 | | 91.1% | | |
| | #170 | 0.090 | | 90.2% | | |
| | #200 | 0.075 | 89.2% | | | |




| DATE TESTED | TESTED BY |
|---|---|
| 12/19/07 | SMJ |

COLUMBIA WEST ENGINEERING, INC. authorized signature

Columbia West Engineering, Inc.
Geotechnical Engineering and Environmental Consulting Services
11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# ATTERBERG LIMITS REPORT

| PROJECT | CLIENT | PROJECT NO. | LAB ID |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | S07-831 |
| | | REPORT DATE: 01/02/08 | FIELD ID: SB3.4 |
| | | DATE SAMPLED: 12/13/07 | SAMPLED BY: JLO |

## MATERIAL DATA

| MATERIAL SAMPLED | MATERIAL SOURCE | USCS SOIL TYPE |
|---|---|---|
| light brown Lean CLAY | Soil Boring SB-03<br>depth = 10 feet | CL, Lean Clay |

## LABORATORY TEST DATA

| LABORATORY EQUIPMENT | TEST PROCEDURE |
|---|---|
| Liquid Limit Machine, Plastic Limit Roller | ASTM D4318 |

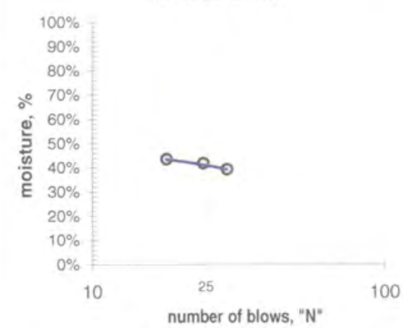
### ATTERBERG LIMITS

| | |
|---|---|
| liquid limit = | 41 |
| plastic limit = | 19 |
| plasticity index = | 22 |

### LIQUID LIMIT DETERMINATION

| | ❶ | ❷ | ❸ | ❹ |
|---|---|---|---|---|
| wet soil + pan weight, g = | 34.54 | 30.50 | 32.36 | |
| dry soil + pan weight, g = | 30.59 | 27.61 | 28.86 | |
| pan weight, g = | 20.55 | 20.71 | 20.82 | |
| N (blows) = | 29 | 24 | 18 | |
| moisture, % = | 39.3 % | 41.9 % | 43.5 % | |

### LIQUID LIMIT



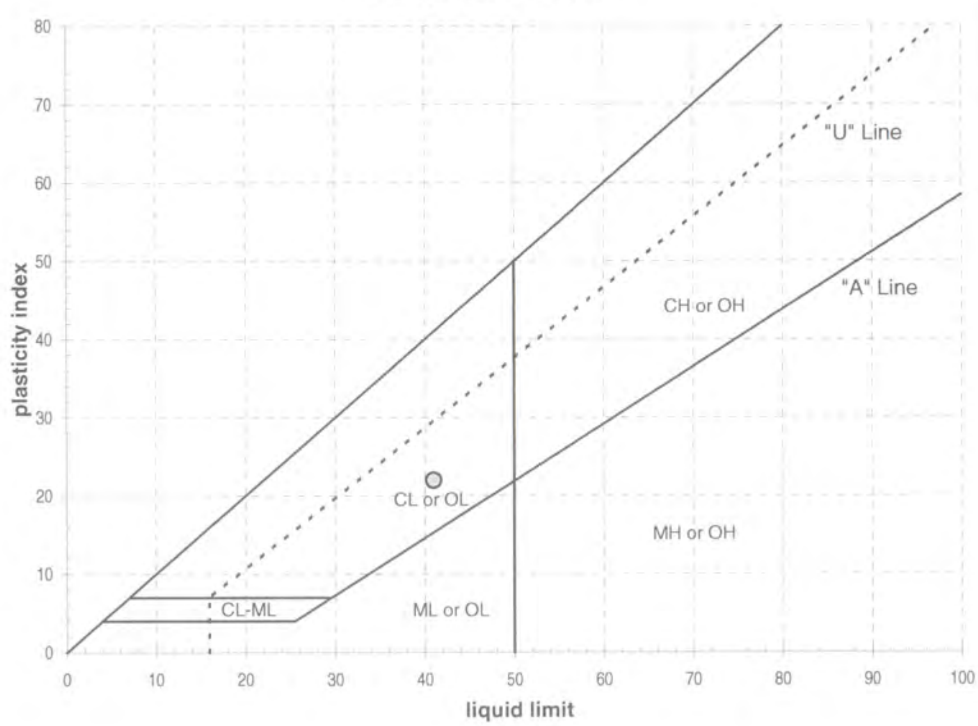

### SHRINKAGE

| | |
|---|---|
| shrinkage limit = | n/a |
| shrinkage ratio = | n/a |

### PLASTIC LIMIT DETERMINATION

| | ❶ | ❷ | ❸ | ❹ |
|---|---|---|---|---|
| wet soil + pan weight, g = | 29.10 | 31.37 | | |
| dry soil + pan weight, g = | 27.77 | 29.69 | | |
| pan weight, g = | 20.68 | 20.82 | | |
| moisture, % = | 18.8 % | 18.9 % | | |

### PLASTICITY CHART




### ADDITIONAL DATA

| | |
|---|---|
| % gravel = | 0.1% |
| % sand = | 10.7% |
| % silt and clay = | 89.2% |
| % silt = | n/a |
| % clay = | n/a |
| moisture content = | 33.9% |

| DATE TESTED | TESTED BY |
|---|---|
| 12/20/07 | SMJ |

COLUMBIA WEST ENGINEERING, INC. authorized signature

Columbia West Engineering, Inc.
Geotechnical Engineering and Environmental Consulting Services
11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# PARTICLE-SIZE ANALYSIS REPORT

| PROJECT | CLIENT | PROJECT NO. | LAB ID |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | S07-832 |
| | | REPORT DATE: 01/02/08 | FIELD ID: HA3.1 |
| | | DATE SAMPLED: 12/13/07 | SAMPLED BY: JLO |

## MATERIAL DATA

| MATERIAL SAMPLED | MATERIAL SOURCE | USCS SOIL TYPE |
|---|---|---|
| brown Lean CLAY with Sand | Hand Auger HA-03<br>depth = 6.5 feet | CL, Lean Clay with Sand |
| SPECIFICATIONS: none | | AASHTO SOIL TYPE: A-7-6(19) |

## LABORATORY TEST DATA

| LABORATORY EQUIPMENT | TEST PROCEDURE |
|---|---|
| Rainhart "Mary Ann" Sifter 637 | ASTM D6913, D2487 |

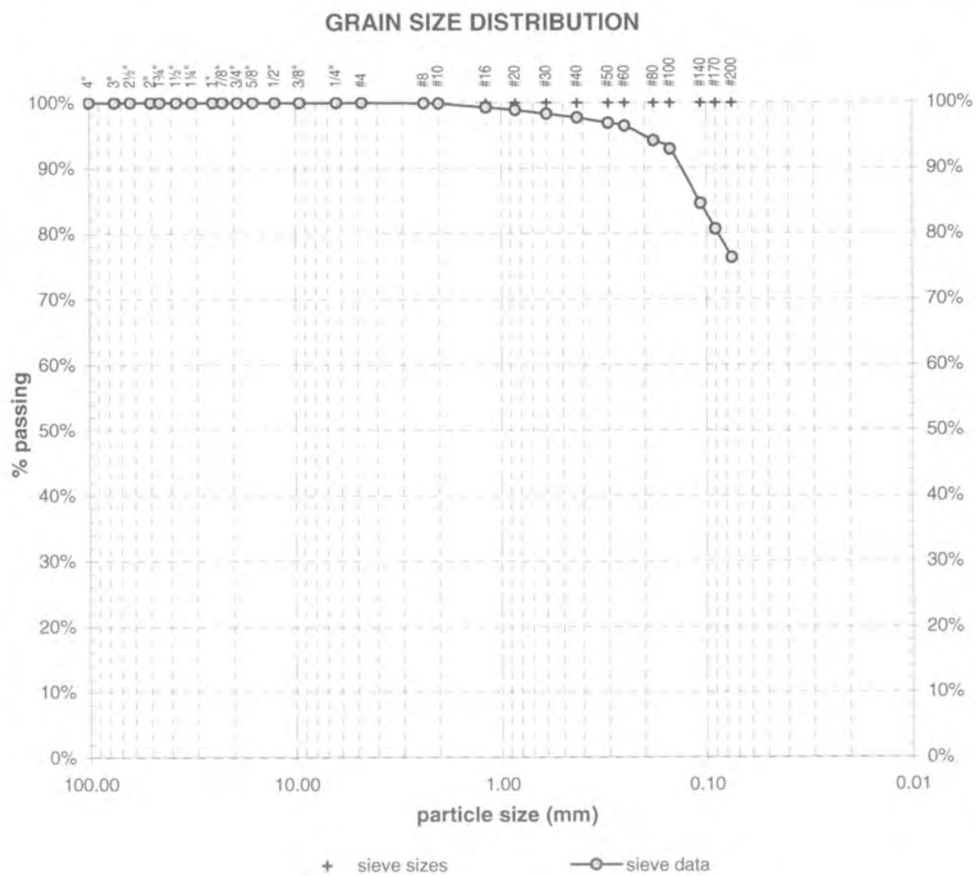
### ADDITIONAL DATA

| | | | |
|---|---|---|---|
| natural moisture content = | 26.7% | coefficient of curvature, $C_C$ = | n/a |
| liquid limit = | 46 | coefficient of uniformity, $C_U$ = | n/a |
| plastic limit = | 20 | effective size, $D_{(10)}$ = | n/a |
| plasticity index = | 26 | $D_{(30)}$ = | n/a |
| fineness modulus = | n/a | $D_{(60)}$ = | n/a |

### SIEVE DATA

% gravel = 0.0%
% sand = 23.7%
% silt and clay = 76.3%

| | 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 | | 99.3% | | |
| | #20 | 0.850 | 98.9% | | | |
| | #30 | 0.600 | | 98.3% | | |
| | #40 | 0.425 | 97.7% | | | |
| | #50 | 0.300 | | 96.9% | | |
| | #60 | 0.250 | 96.4% | | | |
| | #80 | 0.180 | | 94.2% | | |
| | #100 | 0.150 | 92.9% | | | |
| | #140 | 0.106 | | 84.6% | | |
| | #170 | 0.090 | | 80.7% | | |
| | #200 | 0.075 | 76.3% | | | |




| DATE TESTED | TESTED BY |
|---|---|
| 12/19/07 | SMJ |

COLUMBIA WEST ENGINEERING, INC. authorized signature

**Columbia West Engineering, Inc.**
Geotechnical Engineering and Environmental Consulting Services

11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# ATTERBERG LIMITS REPORT

| PROJECT | CLIENT | PROJECT NO. | LAB ID |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | S07-832 |
| | | REPORT DATE: 01/02/08 | FIELD ID: HA3.1 |
| | | DATE SAMPLED: 12/13/07 | SAMPLED BY: JLO |

## MATERIAL DATA

| MATERIAL SAMPLED | MATERIAL SOURCE | USCS SOIL TYPE |
|---|---|---|
| brown Lean CLAY with Sand | Hand Auger HA-03<br>depth = 6.5 feet | CL, Lean Clay with Sand |

## LABORATORY TEST DATA

| LABORATORY EQUIPMENT | TEST PROCEDURE |
|---|---|
| Liquid Limit Machine, Plastic Limit Roller | ASTM D4318 |

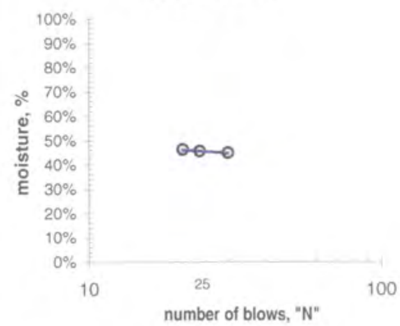
### ATTERBERG LIMITS

liquid limit = 46
plastic limit = 20
plasticity index = 26

### LIQUID LIMIT DETERMINATION

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| wet soil + pan weight, g = | 36.35 | 33.23 | 35.77 | |
| dry soil + pan weight, g = | 31.52 | 29.35 | 31.00 | |
| pan weight, g = | 20.80 | 20.86 | 20.72 | |
| N (blows) = | 30 | 24 | 21 | |
| moisture, % = | 45.1 % | 45.7 % | 46.4 % | |

### LIQUID LIMIT



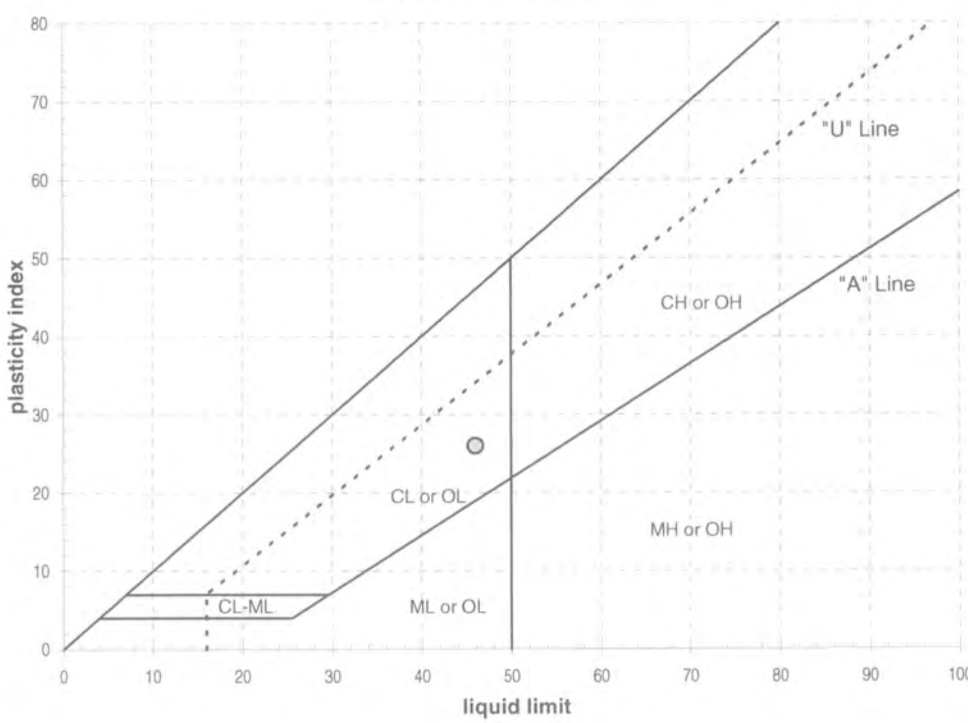

### SHRINKAGE

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

### PLASTIC LIMIT DETERMINATION

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| wet soil + pan weight, g = | 30.07 | 30.28 | | |
| dry soil + pan weight, g = | 28.53 | 28.73 | | |
| pan weight, g = | 20.77 | 20.75 | | |
| moisture, % = | 19.9 % | 19.4 % | | |

### PLASTICITY CHART




### ADDITIONAL DATA

% gravel = 0.0%
% sand = 23.7%
% silt and clay = 76.3%
% silt = n/a
% clay = n/a
moisture content = 26.7%

| DATE TESTED | TESTED BY |
|---|---|
| 12/20/07 | SMJ |

COLUMBIA WEST ENGINEERING, INC. authorized signature

Columbia West Engineering, Inc.
Geotechnical Engineering and Environmental Consulting Services
11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# MOISTURE CONTENT

| PROJECT | CLIENT | PROJECT NO. | |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | |
| | | REPORT DATE: 01/02/08 | |
| | | DATE SAMPLED: 12/13/07 | SAMPLED BY: JLO |

## LABORATORY TEST DATA

LABORATORY EQUIPMENT: Despatch LEB2

TEST PROCEDURE: ASTM D2216, Method B

| LAB ID | FIELD ID | MATERIAL DESCRIPTION AND SOURCE | MOIST MASS | DRY MASS | CONTAINER MASS | MOISTURE CONTENT | OVEN TEMP. |
|---|---|---|---|---|---|---|---|
| S07-829 | SB1.1 | brown Lean CLAY with Sand<br>Soil Boring SB-01, depth = 2.5 feet | 208.8 | 170.5 | 10.9 | 24.0% | 110 ± 5°C |
| S07-833 | SB1.2 | brown clay with sand<br>Soil Boring SB-01, depth = 5 feet | 431.3 | 333.8 | 10.5 | 30.2% | 110 ± 5°C |
| S07-834 | SB1.3 | brown clay with sand<br>Soil Boring SB-01, depth = 7.5 feet | 450.1 | 356.2 | 10.5 | 27.2% | 110 ± 5°C |
| S07-835 | SB1.4 | brown clay with sand<br>Soil Boring SB-01, depth = 10 feet | 439.5 | 346.7 | 10.4 | 27.6% | 110 ± 5°C |
| S07-836 | SB1.5 | grayish brown clay<br>Soil Boring SB-01, depth = 15 feet | 463.3 | 365.0 | 10.6 | 27.7% | 110 ± 5°C |
| S07-837 | SB1.6 | bluish gray clay<br>Soil Boring SB-01, depth = 20 feet | 394.7 | 306.5 | 10.4 | 29.8% | 110 ± 5°C |
| S07-838 | SB1.7 | grayish brown clay<br>Soil Boring SB-01, depth = 25 feet | 401.8 | 300.8 | 10.5 | 34.8% | 110 ± 5°C |
| S07-830 | SB1.8 | brownish gray Sandy Lean CLAY<br>Soil Boring SB-01, depth = 30 to 32 ft | 105.72 | 82.83 | 3.86 | 29.0% | 110 ± 5°C |
| S07-839 | SB1.9 | brown clay with sand<br>Soil Boring SB-01, depth = 32 feet | 401.7 | 309.2 | 10.5 | 31.0% | 110 ± 5°C |
| S07-840 | SB2.1 | brown clay with sand<br>Soil Boring SB-02, depth = 2.5 feet | 349.5 | 266.1 | 10.5 | 32.6% | 110 ± 5°C |

NOTES:

| DATE TESTED | TESTED BY |
|---|---|
| 12/17/07 | SMJ |

COLUMBIA WEST ENGINEERING, INC. authorized signature

Columbia West Engineering, Inc.
Geotechnical Engineering and Environmental Consulting Services
11917 NE 95th Street, Vancouver, WA 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# MOISTURE CONTENT

| PROJECT | CLIENT | PROJECT NO. | |
|---|---|---|---|
| Goode Cluster Subdivision<br>La Center, Washington | Mr. Randy Goode<br>862 E 15th Circle<br>La Center, Washington 98629 | 04133 | |
| | | REPORT DATE: 01/02/08 | |
| | | DATE SAMPLED: 12/13/07 | SAMPLED BY: JLO |

## LABORATORY TEST DATA

LABORATORY EQUIPMENT: Despatch LEB2

TEST PROCEDURE: ASTM D2216, Method B

| LAB ID | FIELD ID | MATERIAL DESCRIPTION AND SOURCE | MOIST MASS | DRY MASS | CONTAINER MASS | MOISTURE CONTENT | OVEN TEMP. |
|---|---|---|---|---|---|---|---|
| S07-841 | SB2.2 | brown clay with sand<br>Soil Boring SB-02, depth = 5 feet | 448.7 | 351.8 | 10.4 | 28.4% | 110 ± 5°C |
| S07-842 | SB2.3 | brown clay<br>Soil Boring SB-02, depth = 7.5 feet | 286.3 | 229.1 | 10.5 | 26.2% | 110 ± 5°C |
| S07-843 | SB2.4 | brown clayey gravel<br>Soil Boring SB-02, depth = 10 feet | 302.9 | 224.1 | 10.6 | 36.9% | 110 ± 5°C |
| S07-844 | SB3.1 | brown clay with sand<br>Soil Boring SB-03, depth = 2.5 feet | 393.4 | 302.8 | 10.7 | 31.0% | 110 ± 5°C |
| S07-845 | SB3.2 | brown clay<br>Soil Boring SB-03, depth = 5 feet | 362.8 | 283.7 | 10.4 | 28.9% | 110 ± 5°C |
| S07-846 | SB3.3 | brown clay<br>Soil Boring SB-03, depth = 7.5 feet | 338.7 | 263.2 | 10.6 | 29.9% | 110 ± 5°C |
| S07-831 | SB3.4 | light brown Lean CLAY<br>Soil Boring SB-03, depth = 10 feet | 213.3 | 162.0 | 10.6 | 33.9% | 110 ± 5°C |
| S07-847 | SB3.5 | grayish brown clay<br>Soil Boring SB-03, depth = 15 feet | 343.4 | 271.4 | 10.8 | 27.6% | 110 ± 5°C |
| S07-848 | SB3.6 | grayish brown clay<br>Soil Boring SB-03, depth = 20 feet | 343.1 | 261.0 | 10.6 | 32.8% | 110 ± 5°C |
| S07-849 | SB3.7 | brown clayey sand with gravel<br>Soil Boring SB-03, depth = 25 feet | 334.3 | 273.0 | 10.7 | 23.4% | 110 ± 5°C |
| | | | | | | | |

NOTES:

| DATE TESTED | TESTED BY |
|---|---|
| 12/17/07 | SMJ |

COLUMBIA WEST ENGINEERING, INC. authorized signature

# APPENDIX B
## SOIL BORING LOGS

# Columbia West Engineering, Inc.

Geotechnical, Environmental, & Materials Testing Services

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

# Environmental Geoprobe Log

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | SB-01 |
| PROJECT LOCATION: La Center, Washington | DRILLING CONTRACTOR: VanDeHey Drilling | DRILL RIG: Simco 2400SK-1 | ENGINEER: ASR | PAGE NO.: 1 of 2 |
| BORING LOCATION: see figure 2 | DRILLING METHOD: trailer-mounted auger | SAMPLING METHOD: split spoon | START DATE: 12/13/07 | START TIME: 0900 |
| REMARKS: | APPROX. SURFACE ELEVATION: 103 feet | GROUND WATER DEPTH: 24 feet | FINISH DATE: 12/13/07 | FINISH TIME: 1115 |

| Depth (feet) | Sample Type | Drive/ Recovery (inches) | SPT Blow Count | Field ID | Sample Depth (feet) | USCS Soil Type | Graphic Log | LITHOLOGIC DESCRIPTION AND REMARKS | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | CL | | Reddish brown lean CLAY with sand, moist, medium stiff, plastic. Fine sand and gray mottles. [Soil Type 1] | | | | | | |
| | SS | 18/18 | 2 2 4 | 01.1 | 2.5 | | | | 24.0 | 74.8 | 31 | 13 | 0.1 | |
| 5 | SS | 18/18 | 4 4 6 | 01.2 | 5 | | | | 30.2 | | | | | |
| | SS | 18/18 | 3 6 7 | 01.3 | 7.5 | | | | 27.2 | | | | | |
| 10 | SS | 18/18 | 4 5 6 | 01.4 | 10 | | | | 27.6 | | | | | |
| 15 | SS | 18/12 | 3 3 5 | 01.5 | 15 | | | | 27.7 | | | | 0.12 | |

BORING LOG 04133, BORING LOGS.GPJ BORING.GDT 10/3/08

Columbia West Engineering, Inc.
Geotechnical, Environmental, & Materials Testing Services
11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# Environmental Geoprobe Log

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | SB-01 |
| PROJECT LOCATION | DRILLING CONTRACTOR | DRILL RIG | ENGINEER | PAGE NO. |
| La Center, Washington | VanDeHey Drilling | Simco 2400SK-1 | ASR | 2 of 2 |
| BORING LOCATION | DRILLING METHOD | SAMPLING METHOD | START DATE | START TIME |
| see figure 2 | trailer-mounted auger | split spoon | 12/13/07 | 0900 |
| REMARKS | APPROX. SURFACE ELEVATION | GROUND WATER DEPTH | FINISH DATE | FINISH TIME |
| | 103 feet | 24 feet | 12/13/07 | 1115 |

| Depth (feet) | Sample Type | Drive/ Recovery (inches) | SPT Blow Count | Field ID | Sample Depth (feet) | USCS Soil Type | Graphic Log | LITHOLOGIC DESCRIPTION AND REMARKS | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | SS | 18/12 | 2 3 4 | 01.6 | 20 | | | | 29.8 | | | | | |
| | | | | | | CL | | Bluish gray lean CLAY, moist, medium stiff, plastic. [Soil Type 1A] | | | | | | |
| | ▽ | | | | | | | | | | | | | |
| 25 | SS | 18/18 | 2 2 5 | 01.7 | 25 | | | | 34.8 | | | | | |
| 30 | SH | 24/15 | | 01.8 | 30 | | | | 29.0 | 72.2 | 33 | 16 | | |
| | SS | 18/18 | 4 4 10 | 01.9 | 32 | | | | 31.0 | | | | | |
| | | | | | | | | Light gray to reddish-brown with varying mottles, severely weathered, CONGLOMERATE consisting primarily of sub-angular gravels and cobbles in a cemented silt, clay, and sand matrix. [Soil Type 2] | | | | | | |
| 35 | SS | 11/4 | 25 50 | 01.10 | 35 | | | | | | | | | |
| | | | | | | | | Bottom of borehole at 36 feet. Ground water encountered at 24 feet. Borehole backfilled with bentonite on 12/13/2007. | | | | | | |

Columbia West Engineering, Inc.
Geotechnical, Environmental, & Materials Testing Services
11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# Environmental Geoprobe Log

| | | | | |
|---|---|---|---|---|
| PROJECT NAME: Goode Property | CLIENT: Randy Goode | | PROJECT NO.: 04133 | BORING NO.: SB-02 |
| PROJECT LOCATION: La Center, Washington | DRILLING CONTRACTOR: VanDeHey Drilling | DRILL RIG: Simco 2400SK-1 | ENGINEER: ASR | PAGE NO.: 1 of 1 |
| BORING LOCATION: see figure 2 | DRILLING METHOD: trailer-mounted auger | SAMPLING METHOD: split spoon | START DATE: 12/13/07 | START TIME: 1130 |
| REMARKS: | APPROX. SURFACE ELEVATION: 126 feet | GROUND WATER DEPTH: 2.5 feet | FINISH DATE: 12/13/07 | FINISH TIME: 1225 |

| Depth (feet) | Sample Type | Drive/ Recovery (inches) | SPT Blow Count | Field ID | Sample Depth (feet) | USCS Soil Type | Graphic Log | LITHOLOGIC DESCRIPTION AND REMARKS | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | CL | | Reddish brown lean CLAY with sand, moist, medium stiff, plastic. Fine sand and gray mottles. [Soil Type 1] | | | | | | |
| | SS | 18/18 | 3 5 7 | 02.1 | 2.5 | | | | 32.6 | | | | | |
| 5 | SS | 18/18 | 5 5 7 | 02.2 | 5 | | | | 28.4 | | | | | |
| | SS | 18/12 | 4 4 5 | 02.3 | 7.5 | | | | 26.2 | | | | | |
| | | | | | | | | Slightly weathered, angular BASALT gravels in sandy CLAY matrix. Becomes more competent with depth. Inferred as massive Basalt bedrock due to large outcrop nearby. [Soil Type 3] | | | | | | |
| 10 | SS | 8/4 | 49 50 | 02.4 | 10 | | | | 36.9 | | | | | |
| | | | | | | | | Bottom of borehole at 11 feet. Ground water encountered at 2.5 feet. Borehole backfilled with bentonite on 12/13/2007. | | | | | | |
| 15 | | | | | | | | | | | | | | |

Columbia West Engineering, Inc.
Geotechnical, Environmental, & Materials Testing Services
11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# Environmental Geoprobe Log

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | SB-03 |
| PROJECT LOCATION: La Center, Washington | DRILLING CONTRACTOR: VanDeHey Drilling | DRILL RIG: Simco 2400SK-1 | ENGINEER: ASR | PAGE NO.: 1 of 2 |
| BORING LOCATION: see figure 2 | DRILLING METHOD: trailer-mounted auger | SAMPLING METHOD: split spoon | START DATE: 12/13/07 | START TIME: 1240 |
| REMARKS: | APPROX. SURFACE ELEVATION: 150 feet | GROUND WATER DEPTH: 2.5 feet | FINISH DATE: 12/13/07 | FINISH TIME: 1350 |

| Depth (feet) | Sample Type | Drive/ Recovery (inches) | SPT Blow Count | Field ID | Sample Depth (feet) | USCS Soil Type | Graphic Log | LITHOLOGIC DESCRIPTION AND REMARKS | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | CL | | Reddish brown lean CLAY with sand, moist, medium stiff, plastic. Fine sand and gray mottles. [Soil Type 1] | | | | | | |
| | ∇ SS | 18/18 | 3 4 6 | 03.1 | 2.5 | | | | 31.0 | | | | | |
| 5 | SS | 18/18 | 4 4 6 | 03.2 | 5 | | | | 28.9 | | | | | |
| | SS | 18/18 | 3 3 3 | 03.3 | 7.5 | | | | 29.9 | | | | | |
| 10 | SS | 18/18 | 2 3 4 | 03.4 | 10 | | | | 33.9 | 89.2 | 41 | 22 | | |
| 15 | SS | 18/18 | 6 7 11 | 03.5 | 15 | | | | 27.6 | | | | | |

Columbia West Engineering, Inc.
Geotechnical, Environmental, & Materials Testing Services
11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# Environmental Geoprobe Log

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | SB-03 |
| PROJECT LOCATION | DRILLING CONTRACTOR | DRILL RIG | ENGINEER | PAGE NO. |
| La Center, Washington | VanDeHey Drilling | Simco 2400SK-1 | ASR | 2 of 2 |
| BORING LOCATION | DRILLING METHOD | SAMPLING METHOD | START DATE | START TIME |
| see figure 2 | trailer-mounted auger | split spoon | 12/13/07 | 1240 |
| REMARKS | APPROX. SURFACE ELEVATION | GROUND WATER DEPTH | FINISH DATE | FINISH TIME |
| | 150 feet | 2.5 feet | 12/13/07 | 1350 |

| Depth (feet) | Sample Type | Drive/ Recovery (inches) | SPT Blow Count | Field ID | Sample Depth (feet) | USCS Soil Type | Graphic Log | LITHOLOGIC DESCRIPTION AND REMARKS | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | SS | 18/12 | 7 13 17 | 03.6 | 20 | CL | | Reddish brown lean CLAY with sand, moist, medium stiff, plastic. Fine sand and gray mottles. [Soil Type 1] [continued] | 32.8 | | | | | |
| 25 | SS | 18/12 | 6 40 50 | 03.7 | 25 | | | | 23.4 | | | | | |
| | | | | | | | | Light gray to reddish-brown with varying mottles, severely weathered, CONGLOMERATE consisting primarily of sub-angular gravels and cobbles in a cemented silt, clay, and sand matrix. [Soil Type 2] | | | | | | |
| | | | | | | | | Bottom of borehole at 27 feet.<br>Ground water encountered at 2.5 feet.<br>Borehole backfilled with bentonite on 12/13/2007. | | | | | | |
| 30 | | | | | | | | | | | | | | |
| 35 | | | | | | | | | | | | | | |

Columbia West Engineering, Inc.
Geotechnical, Environmental, & Materials Testing Services
11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# Environmental Geoprobe Log

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | SB-04 |
| PROJECT LOCATION: La Center, Washington | DRILLING CONTRACTOR: VanDeHey Drilling | DRILL RIG: Simco 2400SK-1 | ENGINEER: ASR | PAGE NO.: 1 of 3 |
| BORING LOCATION: see figure 2 | DRILLING METHOD: trailer-mounted auger | SAMPLING METHOD: split spoon | START DATE: 12/13/07 | START TIME: 1410 |
| REMARKS: | APPROX. SURFACE ELEVATION: 100 feet | GROUND WATER DEPTH: not recorded | FINISH DATE: 12/13/07 | FINISH TIME: 1500 |

| Depth (feet) | Sample Type | Drive/ Recovery (inches) | SPT Blow Count | Field ID | Sample Depth (feet) | USCS Soil Type | Graphic Log | LITHOLOGIC DESCRIPTION AND REMARKS | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | CL | | Reddish brown lean CLAY with sand, moist, medium stiff, plastic. Fine sand and gray mottles. [Soil Type 1] | | | | | | |
| 5 | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | | |

Columbia West Engineering, Inc.
Geotechnical, Environmental, & Materials Testing Services
11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# Environmental Geoprobe Log

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | SB-04 |
| PROJECT LOCATION: La Center, Washington | DRILLING CONTRACTOR: VanDeHey Drilling | DRILL RIG: Simco 2400SK-1 | ENGINEER: ASR | PAGE NO.: 2 of 3 |
| BORING LOCATION: see figure 2 | DRILLING METHOD: trailer-mounted auger | SAMPLING METHOD: split spoon | START DATE: 12/13/07 | START TIME: 1410 |
| REMARKS: | APPROX. SURFACE ELEVATION: 100 feet | GROUND WATER DEPTH: not recorded | FINISH DATE: 12/13/07 | FINISH TIME: 1500 |

| Depth (feet) | Sample Type | Drive/ Recovery (inches) | SPT Blow Count | Field ID | Sample Depth (feet) | USCS Soil Type | Graphic Log | LITHOLOGIC DESCRIPTION AND REMARKS | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | CL | | Bluish gray lean CLAY, moist, medium stiff, plastic. [Soil Type 1A] | | | | | | |
| 25 | | | | | | | | | | | | | | |
| 30 | | | | | | | | | | | | | | |
| 35 | | | | | | | | | | | | | | |
| | | | | | | | | Light gray to reddish-brown with varying mottles, severely weathered, CONGLOMERATE consisting primarily of sub-angular gravels and cobbles in a cemented silt, clay, and sand matrix. [Soil Type 2] | | | | | | |

Columbia West Engineering, Inc.
Geotechnical, Environmental, & Materials Testing Services
11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# Environmental Geoprobe Log

| | | | | |
|---|---|---|---|---|
| PROJECT NAME: Goode Property | CLIENT: Randy Goode | | PROJECT NO.: 04133 | BORING NO.: SB-04 |
| PROJECT LOCATION: La Center, Washington | DRILLING CONTRACTOR: VanDeHey Drilling | DRILL RIG: Simco 2400SK-1 | ENGINEER: ASR | PAGE NO.: 3 of 3 |
| BORING LOCATION: see figure 2 | DRILLING METHOD: trailer-mounted auger | SAMPLING METHOD: split spoon | START DATE: 12/13/07 | START TIME: 1410 |
| REMARKS: | APPROX. SURFACE ELEVATION: 100 feet | GROUND WATER DEPTH: not recorded | FINISH DATE: 12/13/07 | FINISH TIME: 1500 |

| Depth (feet) | Sample Type | Drive/ Recovery (inches) | SPT Blow Count | Field ID | Sample Depth (feet) | USCS Soil Type | Graphic Log | LITHOLOGIC DESCRIPTION AND REMARKS | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Light gray to reddish-brown with varying mottles, severely weathered, CONGLOMERATE consisting primarily of sub-angular gravels and cobbles in a cemented silt, clay, and sand matrix. [Soil Type 2] [continued] | | | | | | |
| 45 | | | | | | | | | | | | | | |
| 50 | | | | | | | | | | | | | | |
| | | | | | | | | Bottom of borehole at 53 feet.<br>Ground water not recorded.<br>Borehole backfilled with bentonite on 12/13/2007. | | | | | | |
| 55 | | | | | | | | | | | | | | |

# APPENDIX C
## TEST PIT EXCAVATION LOGS

# TEST PIT LOG

| PROJECT NAME | CLIENT | | PROJECT NO. | TEST PIT NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | TP-01 |
| **PROJECT LOCATION** | **CONTRACTOR** | **EQUIPMENT** | **ENGINEER** | **DATE** |
| La Center, Washington | | Excavator | JGH | 10/14/05 |
| **TEST PIT LOCATION** | **APPROX. SURFACE ELEVATION** | **GROUND WATER DEPTH** | **START TIME** | **FINISH TIME** |
| see figure 2 | 95 feet | not encountered | 1430 | 1535 |

| Depth (feet) | Sample Type | 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 | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2 | | | | | | | Approximately 24 inches of dark brown TOPSOIL, moist, organic. Large roots exist up to 3 feet in depth. | | | | | | |
| 2–6 | bag | TP1.1 | | | CL | | Reddish brown lean CLAY with sand, moist, medium stiff, plastic. Fine sand and gray mottles. [Soil Type 1]<br><br>nuclear gauge test @ 4 feet: wet density=98.5 pcf, dry density=82.8 pcf, moisture content=19.0%, void ratio=1.036 | | | | | | |
| 6–8 | | | | | | | Light gray to reddish-brown with varying mottles, severely weathered, CONGLOMERATE consisting primarily of sub-angular gravels and cobbles in a cemented silt, clay, and sand matrix. [Soil Type 2] | | | | | | |
| 8–9 | | | | | | | Fractured Basalt bedrock. Refusal at 9ft. [Soil Type 3] | | | | | | |
| 9 | | | | | | | Bottom of test pit at 9 feet.<br>Ground water not encountered. | | | | | | |

Depth scale: 5, 10, 15, 20

Columbia West Engineering, Inc.
Geotechnical and Environmental Engineering and Consulting Services

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

# TEST PIT LOG

| PROJECT NAME | CLIENT | | PROJECT NO. | TEST PIT NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | TP-02 |
| PROJECT LOCATION: La Center, Washington | CONTRACTOR: | EQUIPMENT: Excavator | ENGINEER: JGH | DATE: 10/14/05 |
| TEST PIT LOCATION: see figure 2 | APPROX. SURFACE ELEVATION: 95 feet | GROUND WATER DEPTH: possible seeps at 20 feet | START TIME: 1540 | FINISH TIME: 1650 |

| Depth (feet) | Sample Type | 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 | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 12 inches brown, organic TOPSOIL | | | | | | |
| | | | | | CL | | Brown lean CLAY with sand, moist, very stiff, low plasticity. [Soil Type 1] | | | | | | |
| | bag | TP2.1 | | | | | nuclear gauge test at 4 feet: wet density=107.5 pcf, dry density=84.6 pcf, moisture content=26.9%, void ratio=0.993 | | | | | 3.5 | 0.9 |
| 5 | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | |
| | | TP2.2 | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | |
| | | | | | CL | | Bluish gray lean CLAY, moist, medium stiff, plastic. [Soil Type 1A] | | | | | | |
| | | TP2.3 | | | | | | | | | | 1.8 | |
| 20 | | | | | | | possible groundwater seeps | | | | | | |
| | | | | | | | Bottom of test pit at 21 feet.<br>Ground water possible seeps at 20 feet. | | | | | | |

Columbia West Engineering, Inc.
Geotechnical and Environmental Engineering and Consulting Services

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

# TEST PIT LOG

| PROJECT NAME | CLIENT | | PROJECT NO. | TEST PIT NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | TP-03 |
| **PROJECT LOCATION** | **CONTRACTOR** | **EQUIPMENT** | **ENGINEER** | **DATE** |
| La Center, Washington | | Excavator | JGH | 10/14/05 |
| **TEST PIT LOCATION** | **APPROX. SURFACE ELEVATION** | **GROUND WATER DEPTH** | **START TIME** | **FINISH TIME** |
| see figure 2 | 126 feet | not encountered | 1700 | 1820 |

| Depth (feet) | Sample Type | 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 | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | 12 inches dark brown, organic TOPSOIL | | | | | | |
| 1 | | | | | CL | | Brown lean CLAY with sand, moist, very stiff, low plasticity. [Soil Type 1] | | | | | | |
| 4 | | | | | | | nuclear gauge test at 4 feet: wet density=105.6 pcf, dry density=83.6 pcf, moisture content=26.1%, void ratio=1.06 | | | | | | |
| 8 | | | | | | | increasing plasticity and clay content | | | | | | |
| 18 | | | | | | | increasing moisture | | | | | | |
| 20 | | | | | CL | | Blueish gray lean CLAY, wet, stiff, low plasticity. [Soil Type 1A] | | | | | | |
| 21 | | | | | | | Bottom of test pit at 21 feet.<br>Ground water not encountered. | | | | | | |

Columbia West Engineering, Inc.
Geotechnical and Environmental Engineering and Consulting Services

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

# HAND AUGER LOG

| PROJECT NAME | CLIENT | | PROJECT NO. | HAND AUGER NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | HA-01 |
| PROJECT LOCATION | CONTRACTOR | EQUIPMENT | ENGINEER | DATE |
| La Center, Washington | JLO | JLO | JLO | 12/13/07 |
| TEST PIT LOCATION | APPROX. SURFACE ELEVATION | GROUND WATER DEPTH | START TIME | FINISH TIME |
| see figure 2 | 170 feet | not encountered | 1245 | 1345 |

| Depth (feet) | Sample Type | 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 | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Approximately 12 inches organic TOPSOIL. | | | | | | |
| | | | | | CL | | Reddish brown lean CLAY with sand, moist, medium stiff, plastic. Fine sand. [Soil Type 1] | | | | | | |
| 5 | | | | | | | Bottom of perc test hole. | | | | | | |
| 10 | | | | | | | Bottom of hand auger boring at 10 feet. Ground water not encountered. | | | | | | |
| 15 | | | | | | | | | | | | | |

Columbia West Engineering, Inc.
Geotechnical and Environmental Engineering and Consulting Services
11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# HAND AUGER LOG

| PROJECT NAME | CLIENT | | PROJECT NO. | HAND AUGER NO. |
|---|---|---|---|---|
| Goode Property | Randy Goode | | 04133 | HA-02 |
| PROJECT LOCATION: La Center, Washington | CONTRACTOR: ASR/JLO | EQUIPMENT: ASR/JLO | ENGINEER: ASR/JLO | DATE: 12/13/07 |
| TEST PIT LOCATION: see figure 2 | APPROX. SURFACE ELEVATION: 210 feet | GROUND WATER DEPTH: not encountered | START TIME: 1515 | FINISH TIME: 1545 |

| Depth (feet) | Sample Type | 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 | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Approximately 12 inches organic TOPSOIL. | | | | | | |
| | | | | | CL | | Reddish brown lean CLAY with sand, moist, medium stiff, plastic. Fine sand. [Soil Type 1] | | | | | | |
| 5 | | | | | | | Bottom of perc test hole. | | | | | | |
| | | | | | | | Hand auger rejection due to cobble. Bottom of hand auger boring at 7.5 feet. Ground water not encountered. | | | | | | |
| 10 | | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | |

HAND AUGER LOG 04133, BORING LOGS.GPJ TESTPITGEO.GDT 10/3/08

Columbia West Engineering, Inc.
Geotechnical and Environmental Engineering and Consulting Services
11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900, Fax: 360-823-2901
www.columbiawestengineering.com

# HAND AUGER LOG

| PROJECT NAME | CLIENT | PROJECT NO. | HAND AUGER NO. |
|---|---|---|---|
| Goode Property | Randy Goode | 04133 | HA-03 |

| PROJECT LOCATION | CONTRACTOR | EQUIPMENT | ENGINEER | DATE |
|---|---|---|---|---|
| La Center, Washington | ASR/JLO | ASR/JLO | ASR/JLO | 12/13/07 |

| TEST PIT LOCATION | APPROX. SURFACE ELEVATION | GROUND WATER DEPTH | START TIME | FINISH TIME |
|---|---|---|---|---|
| see figure 2 | 210 feet | not encountered | 1545 | 1615 |

| Depth (feet) | Sample Type | 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 | Pocket Penetrometer (tsf) | Torvane Shear (tsf) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Approximately 12 inches organic TOPSOIL. | | | | | | |
| | | | | | CL | | Reddish brown lean CLAY with sand, moist, medium stiff, plastic. Fine sand. [Soil Type 1] | | | | | | |
| 5 | | | | | | | Bottom of perc test hole. | | | | | | |
| | bag | HA3.1 | | | | | | 26.7 | 76.3 | 46 | 26 | | |
| 10 | | | | | | | Bottom of hand auger boring at 10 feet. Ground water not encountered. | | | | | | |
| 15 | | | | | | | | | | | | | |

# APPENDIX D
## SOIL CLASSIFICATION INFORMATION

# SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

## Particle-Size Classification

| COMPONENT | ASTM/USCS size range | ASTM/USCS sieve size range | AASHTO size range | AASHTO 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

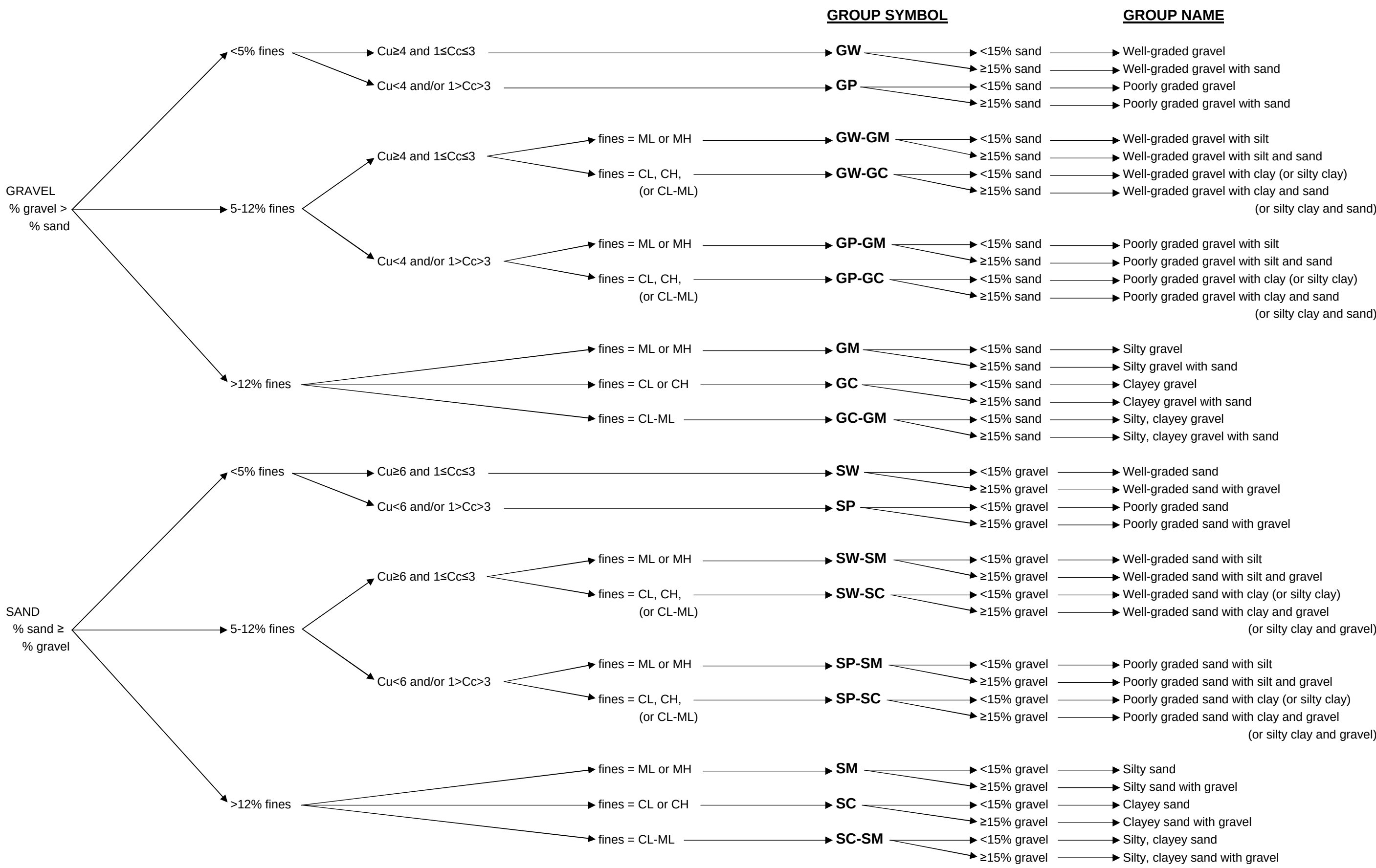
| RELATIVE DENSITY | SPT N-VALUE (BLOWS PER FOOT) |
|---|---|
| Very Loose | 0 to 4 |
| Loose | 4 to 10 |
| Medium Dense | 10 to 30 |
| Dense | 30 to 50 |
| Very Dense | more than 50 |

## Moisture Designations

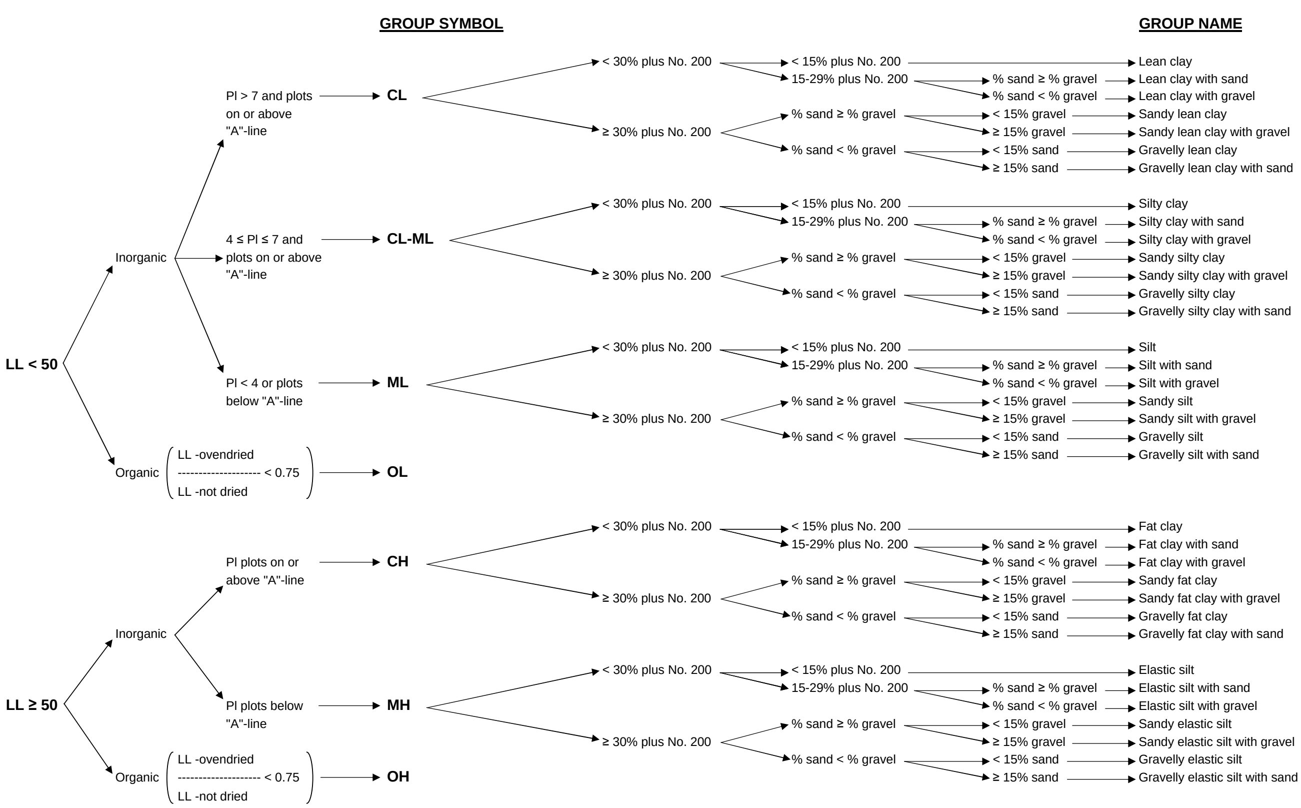
| TERM | FIELD IDENTIFICATION |
|---|---|
| Dry | No moisture. Dusty or dry. |
| Damp | Some moisture. Cohesive soils are usually below plastic limit and are moldable. |
| Moist | Grains appear darkened, but no visible water is present. Cohesive soils will clump. Sand will bulk. Soils are often at or 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. |

# ASTM SOIL CLASSIFICATION SYSTEM

ASTM D2487-02: Standard Classification of Soils for Engineering Purposes (Unified 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)

# 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

# APPENDIX E
## SLOPE STABILITY ANALYSIS

# APPENDIX E: SLOPE STABILITY ANALYSIS
# GOODE PROPERTY, LA CENTER, WASHINGTON
# JANUARY 2008

### Analysis method

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

### Soil Layers and Parameters

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

| Soil Type | Moist Unit Weight (pcf) | Cohesion (psf) | Drained Friction Angle (degrees) |
|---|---|---|---|
| Lean Clay with sand (Soil Type 1) | 120 | 140 | 24 |
| Blue Clay (Soil Type 1A) | 120 | 140 | 24 |
| Conglomerate (Soil Type 2) | 140 | 600* | 46* |
| Basalt Bedrock (Soil Type 3) | - | - | - |

**Equivalent friction angles and cohesive strength for rock mass are based upon Hoek-Brown failure criterion (see Appendix F).*

Soil parameters used in the analyses were estimated based upon laboratory test results, the Hoek-Brown failure criterion for rock, soil and geologic data, and field observations. Soil parameters were generally conservatively chosen to account for the non-homogeneous nature of soils. Therefore, selected friction angles and cohesion values were generally lower than the values obtained from test results. Basalt bedrock was modeled using SLOPE/W's predetermined parameters for bedrock. Layer thicknesses were estimated and interpolated using information from boring, test, and topographic data as well as field observations.

### Piezometric Surface

Piezometric surfaces were estimated based upon field observations of soils, ground water, springs and seeps, and topography and review of well logs. For the purposes of obtaining conservative stability analyses, ground water levels were assumed to be at the ground surface.

### Determination of Seismic Coefficient

Various guidelines exist for selection of the seismic coefficient for use in pseudostatic slope stability analyses. Based upon the type of development, the size and geometry of the slope, existing soil and rock conditions, and local seismicity, a horizontal acceleration of 0.094g (or a seismic coefficient of 0.094) was selected for pseudostatic slope stability analyses. This horizontal acceleration is one-half of 0.187g, the peak ground acceleration for a 475-year return seismic event for the subject site (i.e., a seismic event with a 10 percent chance of occurring in the next 50 years). This is in general accordance with the current geotechnical state of the practice.

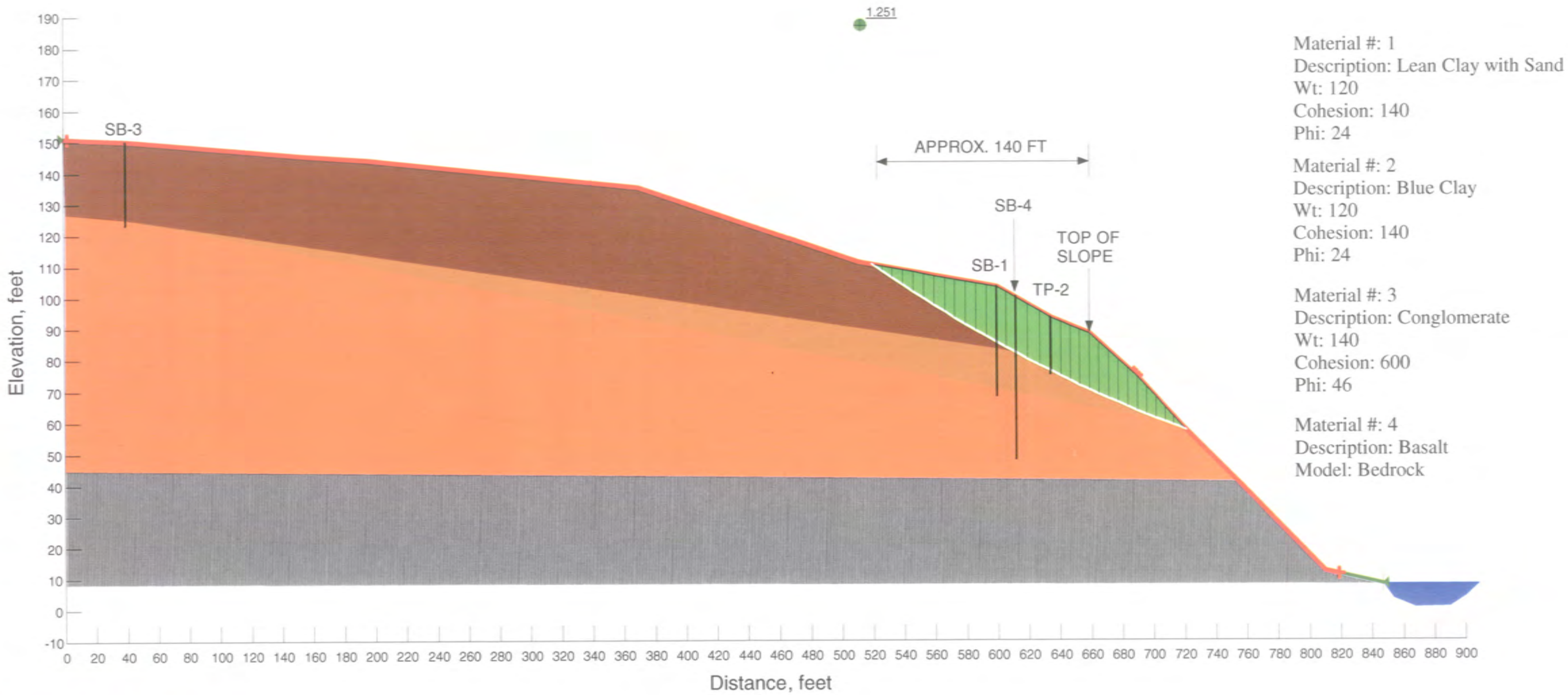
### Interpretation of Results

Attached are graphical results for cross-section A-A', with critical slip surfaces for static and pseudostatic conditions, indicating potential failure along the edge of the river sideslopes on the subject property.

Individual soil layers are designated by color. Piezometric surfaces are indicated as dashed blue lines. The ground surface is shown as a black line and entry and exit ranges for slip surfaces are shown as red lines on the ground surface. Test pit and soil boring locations are shown where appropriate.

On each cross-section, the critical slip surface is indicated as a white line and the individual slices analyzed for the critical slip surface are shown. The lowest factor of safety is shown next to the radius point of the critical slip surface.

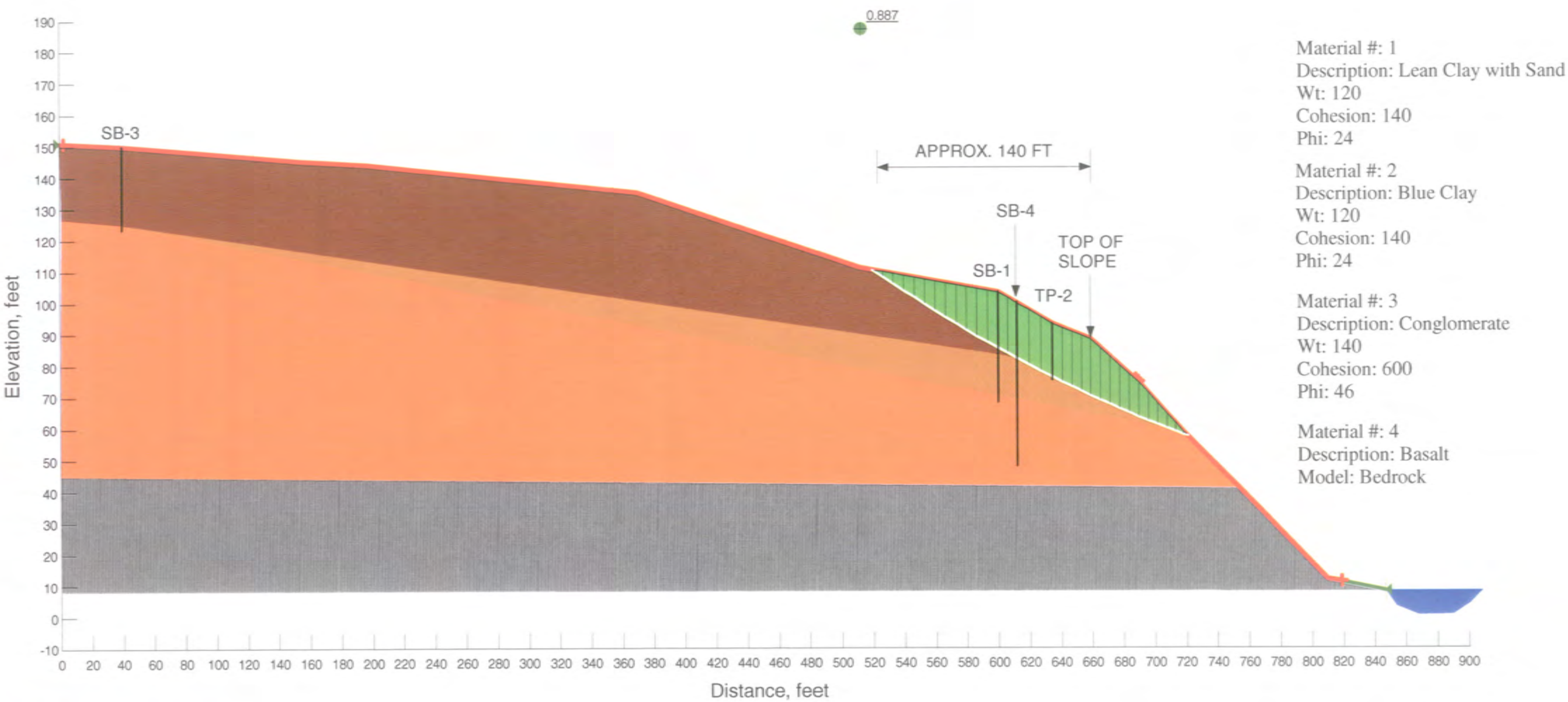
GOODE PROPERTY
LA CENTER, WASHINGTON
04133, CROSS SECTION A-A'
HORIZONTAL SEISMIC ACCELERATION: 0g




Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901

04133, A-A' cross-section 012208.gsz, rev. 1/24/2008

GOODE PROPERTY
LA CENTER, WASHINGTON
04133, CROSS SECTION A–A'
HORIZONTAL SEISMIC ACCELERATION: 9.4e–002g




Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901

04133, A-A' cross-section Seismic 012208.gsz, rev. 1/30/2008

# APPENDIX F
## HOEK-BROWN FAILURE CRITERION

# APPENDIX F: HOEK-BROWN PARAMETERS FOR ROCK AND SEDIMENTARY CONGLOMERATE
# GOODE PROPERTY, LA CENTER, WASHINGTON
# JANUARY 2008

## Background

The Hoek-Brown failure criterion for rock masses can be used to determine equivalent friction angles and cohesive strengths for rock masses based upon their observable properties. In general, Mohr-Coulomb failure criteria are estimated by fitting an average linear relationship to a Mohr envelope derived by evaluating the rock strength for a range of major and minor principal stresses. These empirically derived equivalent parameters can be appropriate for use in slope stability analyses.

## Equivalent Soil Parameters

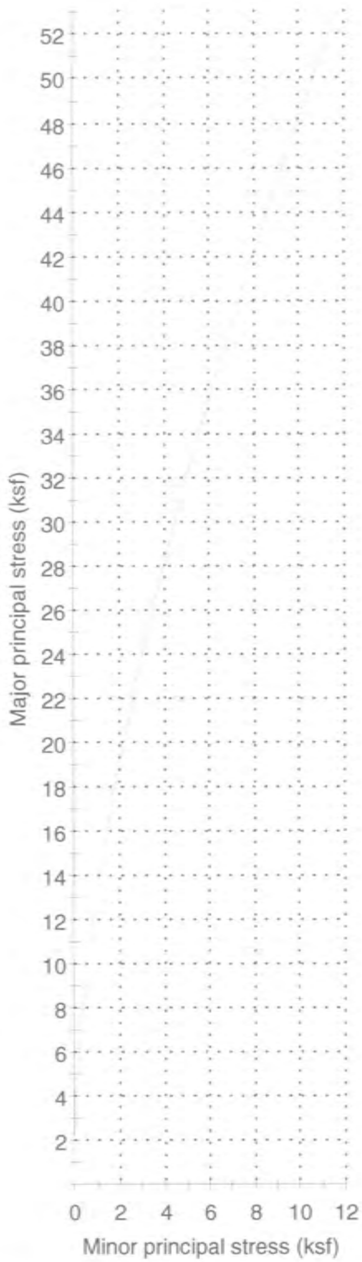
Equivalent soil parameters for the conglomerate layer (Soil Type 2) were determined with RocLab software using the Hoek-Brown failure criterion. Values for strength characteristics of the conglomerate used in the analysis are shown in Table F1.

*Table F1: RocLab strength parameters for weathered basalt bedrock*

| Rock Parameter | Value | Notes |
|---|---|---|
| Intact uniaxial compressive strength, $\sigma_i$ | 60 ksf | Conglomerate |
| Geological Strength Index (GSI) | 40 | blocky/disturbed rock |
| Material constant, $m_i$ | 21 | Conglomerate |
| Disturbance factor, D | 0 | undisturbed rock mass |

Based upon the Mohr-Coulomb fit for the values above, an equivalent friction angle of 46º and a cohesive strength of 600 psf were used to describe the conglomerate. For the purpose of obtaining conservative stability analyses, the equivalent friction angle and cohesive strength were assumed lower than the values determined by the Hoek-Brown failure criterion.

## Analysis of Rock Strength using RocLab

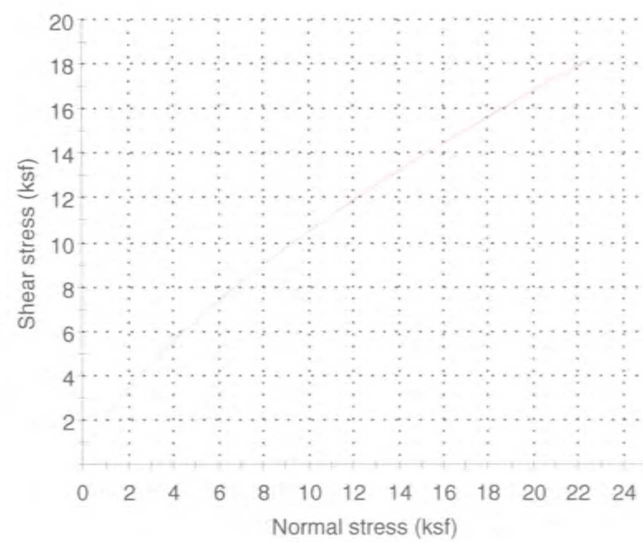



**Hoek-Brown Classification**
intact uniaxial comp. strength (sigci) = 60 ksf
GSI = 40   mi = 21   Disturbance factor (D) = 0
intact modulus (Ei) = 21000 ksf
modulus ratio (MR) = 350

**Hoek-Brown Criterion**
mb = 2.464   s = 0.0013   a = 0.511

**Mohr-Coulomb Fit**
cohesion = 2.741 ksf   friction angle = 35.93 deg

**Rock Mass Parameters**
tensile strength = -0.031 ksf
uniaxial compressive strength = 1.984 ksf
global strength = 12.148 ksf
deformation modulus = 3352.70 ksf



# APPENDIX G
## REPORT LIMITATIONS AND IMPORTANT INFORMATION

Date: January 31, 2008
Project: Goode 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.