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P.O. Box 407
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DOCUMENT TITLE(S) X Geotechnical Site Investigation Pond Property Stevenson, Washington

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LEGAL DESCRIPTION *(Abbreviated: i.e. Lot, Block, Plat or Section, Township, Range, Quarter):* SW 1/4 SE 1/4 Sect. 26 Town 3N, Range 7 East

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Columbia West Engineering, Inc.

GEOTECHNICAL SITE INVESTIGATION
POND PROPERTY
STEVENSON, WASHINGTON

Prepared For: Mr. Michael Pond
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Site Location: Stevenson, Washington

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Date Prepared: November 16, 2006

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GEOTECHNICAL SITE INVESTIGATION
POND PROPERTY
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1.0 INTRODUCTION

Columbia West Engineering, Inc. was retained by Michael Pond to conduct a geotechnical site investigation for the Pond Property, a 10.56-acre parcel located in Stevenson, Washington proposed for a single-family residential home. The purpose of the investigation was to observe and assess subsurface soil conditions at specific locations and provide subsequent appropriate geotechnical engineering analyses to support property development, planning, and design recommendations. The specific scope of services was outlined in a proposal contract dated September 27, 2006, and authorized by client signature on September 29, 2006. 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 and Appendix D. Please note this report contains recommendations for future additional investigation and slope stability analysis prior to single-family home construction.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located east of Myers Road in Stevenson, Washington. The regulatory jurisdictional agency is Skamania County. The approximate latitude and longitude are N 45° 42' 40" and W 121° 54' 29" and the legal description is a portion of the SW ¼ of Section 26, T3N, R7E, Willamette Meridian. The site consists of a single 10.56-acre parcel.

1.2 Proposed Development

Preliminary correspondence with the client indicates that construction of a single-family residential home is proposed for the site. Columbia West has not reviewed a preliminary grading plan, but understands minor cut and fill areas may be proposed. 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 is located at the intersection of the Cascade Range uplift region and the Columbia River Gorge. The region is characterized by deeply dissected mountains that have been incised by the Columbia River and associated tributaries.

According to the *Geological Map of the Hood River Quadrangle, Washington and Oregon (Washington Division of Geology and Earth Resources, Open File Report 87-6, Revised December 1987)*, near-surface geologic strata are expected to consist of lower Miocene, porphyritic, basaltic andesite flows of Stevenson Ridge.

The *Soil Survey of Skamania County Area, Washington (United States Department of Agriculture, Soil Conservation Service [USDA SCS], October 1990)* identifies surface soils as primarily Stevenson loam and Steever stony clay loam. Although soil conditions may vary from the broad USDA descriptions, Stevenson and Steever soils are generally fine-textured and have moderate permeability, high water capacity, and moderate to severe erosion hazard if left in a bare unvegetated condition. Stevenson soils exhibit moderate shrink/swell potential, low shear strength, and are somewhat compressible and generally moisture sensitive. Steep slopes associated with Steever soils limit potential urban development as a primary use. Urban development design parameters are not provided by the USDA for Steever soils. Although not encountered during subsurface investigation at the proposed building site, Steever soils are mapped in the northern portion of the property.

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 several known fault zones in the vicinity of the site that may be capable of generating potentially destructive horizontal accelerations. Some of these fault zones are described briefly in the following text.

Local Faults

Limited data exists concerning two faults lying within approximately six miles of the site. The Wind River Fault is approximately parallel with the west side of the Wind River, east of the subject site, near the town of Carson. West of the site, following the approximate eastern boundary of the Cascade Slide Complex, an inferred unnamed northwest trending fault has been identified in the *Geotechnical Report, Maple Hill Landslide, Skamania County, Washington.* Age and displacement of these two faults is not well established.

Faults near The Dalles

According to the *Geological Map of the Hood River Quadrangle, Washington and Oregon (Washington Division of Geology and Earth Resources, Open File Report 87-6, Revised November 1987)* and the *U.S. Geologic Survey's website* several northwest-striking, right-lateral, strike-slip and minor normal faults exist or are inferred approximately 12 to 35 miles east of the subject site. Offset of Miocene to Pliocene volcanic and volcaniclastic sedimentary rocks indicates Tertiary activity within the fault zone. Possible faulting of intracanyon basalt flows may indicate more recent Quaternary activity. Faults near The Dalles appear to cut east trending folds and faults of the Columbia Hills structures.

Lacamas Creek-Sandy River Fault Zone

The northwest-trending Lacamas Creek Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 23 miles west 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 AND GEOLOGIC FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance and three test pit explorations (TP-1 through TP-3) was conducted at the site. Test pit exploration was performed with a pneumatic-wheeled backhoe. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications. Disturbed soil samples were collected from relevant soil horizons and submitted for laboratory analysis. Laboratory test results are presented in Appendix A. Test pit locations are indicated on Figure 2 and soil logs are presented in Appendix B. Soil descriptions and classification information are provided in Appendix C.

4.1 Surface Investigation and Site Description

Field reconnaissance and review of topography maps indicate the subject property ranges in elevation from approximately 800 feet above mean sea level near the northern border to approximately 560 feet near the southern border. The site is situated on the eastern flank of the Rock Creek drainage basin. Slope grades are south-trending across the parcel and generally range from 10 to 20 percent. A steep topographic step crosses the northern portion of the parcel from west to east and turns southerly near the eastern boundary. Slope grades vary from 50 to more than 100 percent along this topographic feature. A Bonneville Power Administration (BPA) transmission line easement transects the property from north the south.

At the time of investigation the site was primarily undeveloped with the following exceptions. BPA transmission tower 127/3 of the Wautoma-Ostrander line is located approximately 50 to 100 feet west of the proposed building envelope. An access road extends east from Myers road to the tower. An existing shed encroaches on the northeast property corner. Much of the property is heavily treed with coniferous and deciduous species. Other vegetation including shrubs, bushes, and grasses was also present. Height of vegetation is controlled within the BPA easement.

4.2 Geologic Reconnaissance

Geologic reconnaissance was conducted at the site on October 5, 2006. Subsurface data from the test pit investigation, described in Section 4.3, was also used to interpret geological conditions.

The geologic setting of the site is defined and controlled by the Cascade Mountain Range and the Columbia River Gorge. Stevenson Ridge, composed of lower Miocene basaltic andesite flows and flow breccia, dominates local, site-specific, geologic conditions. Gentle to steep slopes and some topographic unconformities comprise site topography. Slopes trend downgradient in a southerly direction.

According to review of the *Geological Map of the Hood River Quadrangle, Washington and Oregon (Washington Division of Geology and Earth Resources, Open File Report 87-6, Revised December 1987)*, Stevenson Ridge volcanic rocks and flows are a part of a larger geologic unit identified as the Ohanapecosh formation. Basaltic andesite of Stevenson Ridge is generally brown to dark gray, and porphyritic, forming thick, platy to massive, columnar-jointed flows. Portions of the unit dip slightly to the southwest. This unit was not observed during site reconnaissance or in subsurface explorations.

4.3 Subsurface Exploration and Investigation

Three test pits were excavated at the site to a maximum depth of 13 feet on October 6, 2006. Test pit locations were selected to observe subsurface soil characteristics in proximity to the proposed residential home.

4.3.1 Soil Type Description

The field investigation indicated the site is generally covered with a topsoil layer approximately 12 to 18 inches thick. Underlying the topsoil, subsurface soils resembling the USDA Stevenson loam description were generally encountered. The subsurface soil profile was relatively similar for all three test pits. Subsurface lithology may generally be described by the soil type identified in the following text.

Soil Type 1 – Silty SAND with gravel

Soil Type 1 was observed to consist of light brown, moist to wet, loose to medium-dense, silty SAND with gravel and intermixed cobbles. Soil Type 1 was observed below the topsoil layer in all test pits and extended to the maximum depth of exploration. Content of fines and sub-angular to rounded cobbles within Soil Type 1 varies significantly depending on location and depth.

Field estimates and laboratory analysis indicate the in situ moisture content varied from approximately 24 to 40 percent. Analytical laboratory testing conducted upon a representative sample of Soil Type 1 indicated approximately 17 to 21 percent by weight passing the No. 200 sieve. Atterberg Limits analysis resulted in a liquid limit ranging from 50 to 53 and a plasticity index of approximately 20. Soil Type 1 is classified SM according to USCS specifications.

4.3.2 Ground Water

Although subsurface ground water seeps were *not* encountered to a depth of 13 feet, extremely wet to saturated soil was observed at the extent of excavation within test pit TP-2. Also, wet soil was observed at the surface in a low-lying area east of the proposed building site. According to the Washington State Department of Ecology website, static ground water at nearby wells has been observed at depths of approximately 180 to 612 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 along slopes or in areas cut below existing grade. Structures, roads, and drainage design should be planned accordingly.

5.0 SLOPE STABILITY

5.1 Literature Review

Large-scale landslides have been identified and studied, including the Cascade Slide Complex and the Maple Hill Landslide, within close proximity to the project site. Columbia West has reviewed several documents regarding nearby mass ground movement.

The mapped eastern boundary of the Red Bluffs Slide, a portion of the Cascade Slide Complex, is approximately concurrent with Rock Creek 2,000 feet southwest of the subject site. Studies of the Cascade Slide Complex indicate several square miles of earth movement that may have moved as recently as the 1700's. These slides dammed and shifted the course of the Columbia River. Slide debris ranged up to 200 feet thick and primarily consists of Columbia River basalts bedded on southerly dipping lower Miocene Eagle Creek sedimentary deposits. Significant infrastructure is founded on Cascade Slide Complex debris including portions of Bonneville Dam, Washington State Route 14, and Interstate Highway 84.

The Maple Hill Landslide occurred in February, 1996 and is associated with a period of prolonged heavy precipitation. Squire Associates, Inc. in association with Otak, Inc. produced a Geotechnical Report identifying tension cracking, debris flows, and damaged structures from the 1996 event on a small portion of the dormant Kanaka Creek Landslide. According to the report, failures occurred primarily within the Eagle Creek Formation that overlies older volcanic rocks and lava flows of the Ohanapecosh formation with slight angular unconformity. These recent mass ground movements occurred approximately 3,000 feet to the east of the subject site.

5.2 Slope Reconnaissance

To observe geomorphic conditions, Columbia West personnel conducted visual and physical reconnaissance of the site. As described previously, the parcel lies on the southern slope of the Rock Creek drainage basin. The property and surrounding adjacent properties generally slope toward the south at grades typically ranging from 10 to 40 percent.

Portions of the site exhibit hummocky topography and several well-defined shallow failures were observed on the face of the steep slope that transects the site. The base of the steep slope is approximately 200 feet east and north of the proposed building site. The steep slope may represent a secondary scarp from an older relatively large scale landslide. Several trees with inclined or rotated trunks were present throughout the site.

Correspondence with the Skamania County road maintenance department indicates that sections of Aalvik and Manning Roads, approximately 1,000 feet south of the site, require annual mitigation due to ground movement. Road signs designate this portion of Aalvik road as a Slide Area. The western edge of the designation approximately corresponds with the western edge of the Wautoma-Ostrander BPA easement.

Columbia West contacted the Geotechnical Department of the Transmission Business Division of the Bonneville Power Administration regarding the Wautoma-Ostrander line that runs through the site from north to south. No line, access road, or geotechnical files revealed information concerning tower 127/3. According to the department, the lack of documentation likely indicates problems arising from ground movement have not been encountered at the transmission tower. The proposed building site lies approximately 50 feet to the east of the BPA easement.

These observations indicate historic mass ground movement may have occurred at the site and is possible in the future. Although catastrophic geologic or climatologic conditions were likely present at the time of past mass ground movement, nearby slides reveal that the potential for failure exists in present conditions. The continued presence of soil creep and shallow soil failures along the steep slope indicate portions of the site exist at grades near or exceeding the soil's shear strength capacity. The potential for slope instability underscores the importance of proper site planning, drainage design, and risk assessment.

6.0 DESIGN RECOMMENDATIONS

The geotechnical site investigation suggests the proposed single-family residential home 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. Findings presented in Section 5.0, *Slope Stability* indicate a relatively high risk of potentially damaging ground movement. Future development should only proceed if this risk is understood and acceptable to the builder. If proper design and construction techniques are utilized, the proposed single-family residence is not likely to exacerbate the global stability of the area. Soil borings and deep-seated slope stability analysis, beyond the scope of this investigation, may further define the risk of potential ground movement and are recommended prior to single-family home construction and significant investment of resources. The following text sections present design recommendations.

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 structural areas. Stripped topsoil may be used 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 areas with heavy vegetation, large trees, existing structures, or undocumented fill. 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 placed or spread at any location onsite should not exceed one foot.

Previously disturbed soils, debris, or undocumented fill encountered during grading or construction activities should be removed completely and thoroughly from structural areas. 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. To limit the risk of potential instability, site-specific grading plans should minimize proposed grade changes and significant cuts and fills.

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 Backfill

Areas proposed for fill placement should be appropriately prepared as described in the preceding text. Surface soils should then be scarified and compacted prior to additional fill placement. Engineered structural fill should be placed in loose lifts not exceeding 12 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within two percentage points of optimum conditions. A field density at least equal to 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 native soils may be suitable for use as structural fill if adequately moisture-conditioned to achieve recommended compaction specifications. Cobbles and boulders larger than 6 inches should be screened and removed from the fill prior to placement. Because they are moisture-sensitive, native soils are often difficult to excavate and nearly impossible to compact during wet weather conditions. If adequate compaction is not achievable with on site native materials, 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

If fill is placed on existing grades steeper than 5H:1V, the area 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 3. Drainage implementations, including subdrains or perforated drain pipe trenches, may also be necessary in proximity to cut and fill slopes if seeps, springs, or soft mottled soils are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by the geotechnical engineer during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion.

Final cut or fill slopes at the site should not exceed 2H:1V or 20 feet in 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 residential structures.

Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Cut or fill slopes greater than 30 feet in height should be terraced in accordance with requirements specified in the *2006 IBC, Section J109.*

Fill slopes should be constructed by placing fill material in maximum 12-inch level lifts, compacting as described in Section 6.2, *Engineered Structural Backfill* 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 the proposed residential home are anticipated to consist of shallow continuous perimeter footings or column spread footings. Maximum expected loads are approximately two to three kips per foot for perimeter footings and 10 to 20 kips per column. Due to the possibility of significant ground movements a slab or mat foundation may be proposed to limit differential settlement. As described previously, additional geotechnical engineering is recommended to determine whether slab or mat foundations are necessary. 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 soils, or engineered structural fill.

To evaluate bearing capacity for the proposed structure, 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 native soils 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 firm native soil or engineered structural fill and in-place poured concrete is 0.35. Lateral forces may also be resisted by an assumed passive soil equivalent fluid pressure of 250 psf/f against embedded footings. The upper six inches of soil should be neglected in passive pressure calculations.

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 additional 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 excavations prior to placing forms or reinforcing bar to verify subgrade support conditions are as anticipated in this report.

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. Increased potential for differential settlement may also be expected where the difference in fill depth between opposite building pad corners exceeds 10 feet. Settlement associated with mass ground movement is anticipated to be much larger and may sever utilities and reduce structural performance below serviceable levels. Future geotechnical study should be conducted to further define settlement estimates prior to single-family home construction.

6.6 Excavation

To install utilities and construct site improvements, subsurface excavation is anticipated. Subsurface soils at the site were excavated to a maximum depth of 13 feet with conventional earthmoving equipment during field exploration activities. Bedrock was not encountered and blasting is not anticipated. If bedrock is encountered during site improvements specialized excavation techniques may be required.

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

The contractor should be held responsible for site safety, sloping, and shoring. Columbia West is not responsible for contractor activities and in no case should excavation be conducted in excess of all applicable local, state, and federal laws. This includes *Washington Administrative Code (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 95 percent of maximum dry density as determined by the modified Proctor test (ASTM D1557). Recommended parameters for lateral earth pressures for undisturbed native soils and engineered structural fill consisting of recompacted native material are presented in Table 1. Lateral pressures are provided for varying exterior grades. If recommended compaction specifications cannot be achieved with recompacted native backfill soils, suitable granular import material should be used.

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

Table 1. Recommended Lateral Earth Pressure Parameters

Backfill Material	Equivalent Fluid Pressure for Level Backfill			Equivalent Fluid Pressure for Sloped Backfill* (3H:1V)		Equivalent Fluid Pressure for Sloped Backfill* (2H:1V)		Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive	Active	Passive	Active	Passive		
in situ, undisturbed native soil	51 pcf	35 pcf	230 pcf	44 pcf	166 pcf	†	†	90 pcf	26°
recompacted native structural fill	50 pcf	34 pcf	263 pcf	42 pcf	194 pcf	62 pcf	117 pcf	95 pcf	28°

** For active pressures, consider slope inclination of wall backfill; for passive pressures, consider slope inclination in front of wall. The upper 6 inches of soil should be neglected in passive pressure calculations.*
† The maximum slope for undisturbed native soil behind the wall is 3H:1V.

If seismic design is required, seismic forces for unrestrained walls may be calculated by superimposing a uniform lateral force of $10H^2$ pounds per lineal foot of wall, where H is the total wall height in feet. The resultant force should be applied at 0.6H from the base of the wall. 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 is assumed behind retaining walls. Geotextile filter fabric should be placed between the drain rock and backfill soil. Specifications for drainpipe design are presented in Section 6.10, *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	0.13 g	0.26 g
0.2 sec Spectral Acceleration	0.31 g	0.62 g
1.0 sec Spectral Acceleration	0.11 g	0.23 g

The listed probabilistic ground motion values are based upon "firm rock" sites with an assumed shear wave velocity of 2,500 ft/s in the upper 100 feet of soil profile. These values should be adjusted for site class effects by applying site coefficients F_a and F_v as defined in *2006 IBC Tables 1613.5.3(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. Subsurface soil properties for the subject site may be represented by Site Class D 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-dense, contain a significant percentage of fines, and generally do not 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

Drainage design in general should conform to Skamania County regulations. Finished site grading should be conducted with positive drainage away from the proposed residential home. Depressions or shallow areas that may retain ponding water should be avoided. Roof drains, low-point crawl space drains, and perimeter foundation drains are recommended for the proposed home. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from the home to an approved discharge location.

Perimeter foundation drains should consist of 3-inch perforated PVC pipe surrounded by a minimum of 1 ft^3 of clean, washed drain rock per linear foot of pipe and wrapped with geotextile filter fabric. Open-graded drain rock with a maximum particle size of 3 inches and less than 2 percent passing the No. 200 sieve is recommended. Geotextile filter fabric should consist of 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 5 presents a typical foundation drain. Perimeter drains may limit increased hydrostatic pressure beneath footings and assist in reducing potential perched moisture areas.

Subdrains should also be considered 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 6 presents a typical perforated drain pipe trench detail.

6.11 Wet Weather Construction Methods and Techniques

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

Construction equipment traffic across exposed native soil should be minimized. Equipment traffic induces dynamic loading, which may result in weak areas and significant reduction in shear strength for soils above plastic limit. Wet weather construction may 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.

Driveway or road construction during wet weather conditions may require increased base thickness. Road base should consist of 3"-0 or 1¼"-0 crushed aggregate and should be placed on previously stripped and structurally competent subgrade. Over-excavation may be necessary to provide a firm base upon which to place crushed aggregate. 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.

6.12 Soil Erosion Potential

According to the *Soil Survey for the Skamania County Area, Washington* the potential erosion hazards for site soils is moderate to severe.

For flat to shallow-gradient portions of the property the erosion hazard is likely to be low. The potential for erosion generally increases in sloped areas. It is recommended that disturbance to vegetation in the sloped areas be minimized during construction activities. Soil is also prone to erosion if unprotected and unvegetated during periods of increased precipitation. Erosion can be minimized by performing construction activities during dry summer months.

Site-specific erosion control measures should be implemented to address the maintenance of slopes or exposed areas. This may include silt fence, biofilter bags, straw wattles, or other suitable methods. During construction activities, all 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.

Finished slopes 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, slope vegetation should be properly maintained. Concentrated water should be prevented from flowing over slope faces. It is also recommended that disturbance to existing native vegetation and surrounding organic soil be minimized during construction activities.

6.13 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 the Skamania County and the site geotechnical engineer. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). The remaining backfill should be compacted to at least 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 parcel grading and construction of the proposed residential home. 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 proposed home should be reviewed and approved by Columbia West as they relate to geotechnical and grading issues prior to construction. Please note this report contains recommendations for additional subsurface investigation and slope stability assessment prior to single-family home construction. 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. Appendix D presents additional report limitations and important information about this document. 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, EIT
Senior Staff Engineer

Lance V. Lehto, PE, MS
President

LANCE V. LEHTO
STATE OF WASHINGTON
36301
REGISTERED
PROFESSIONAL ENGINEER
11-16-06
EXPIRES: 9-10-08

LVL:JLO:arh

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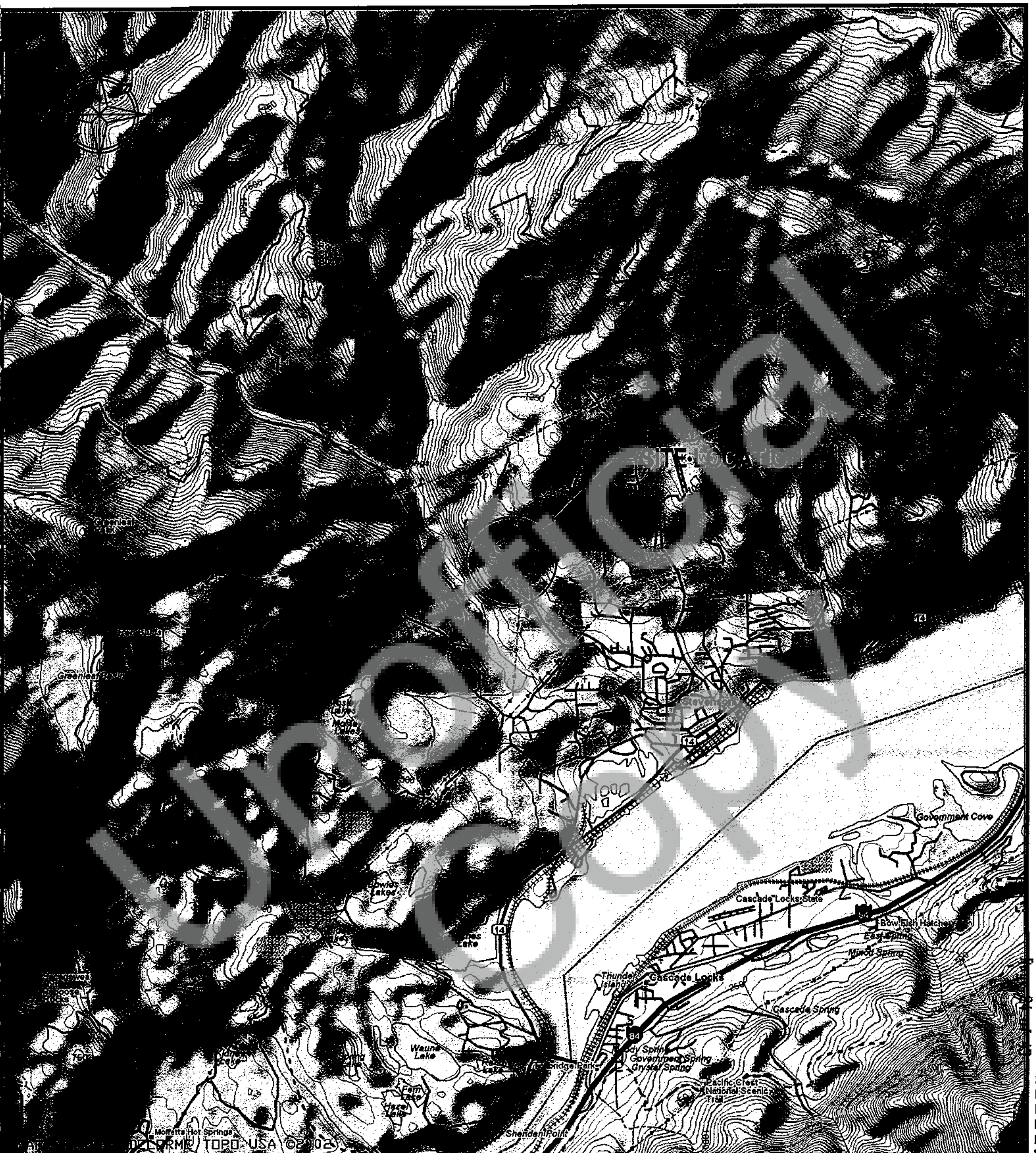
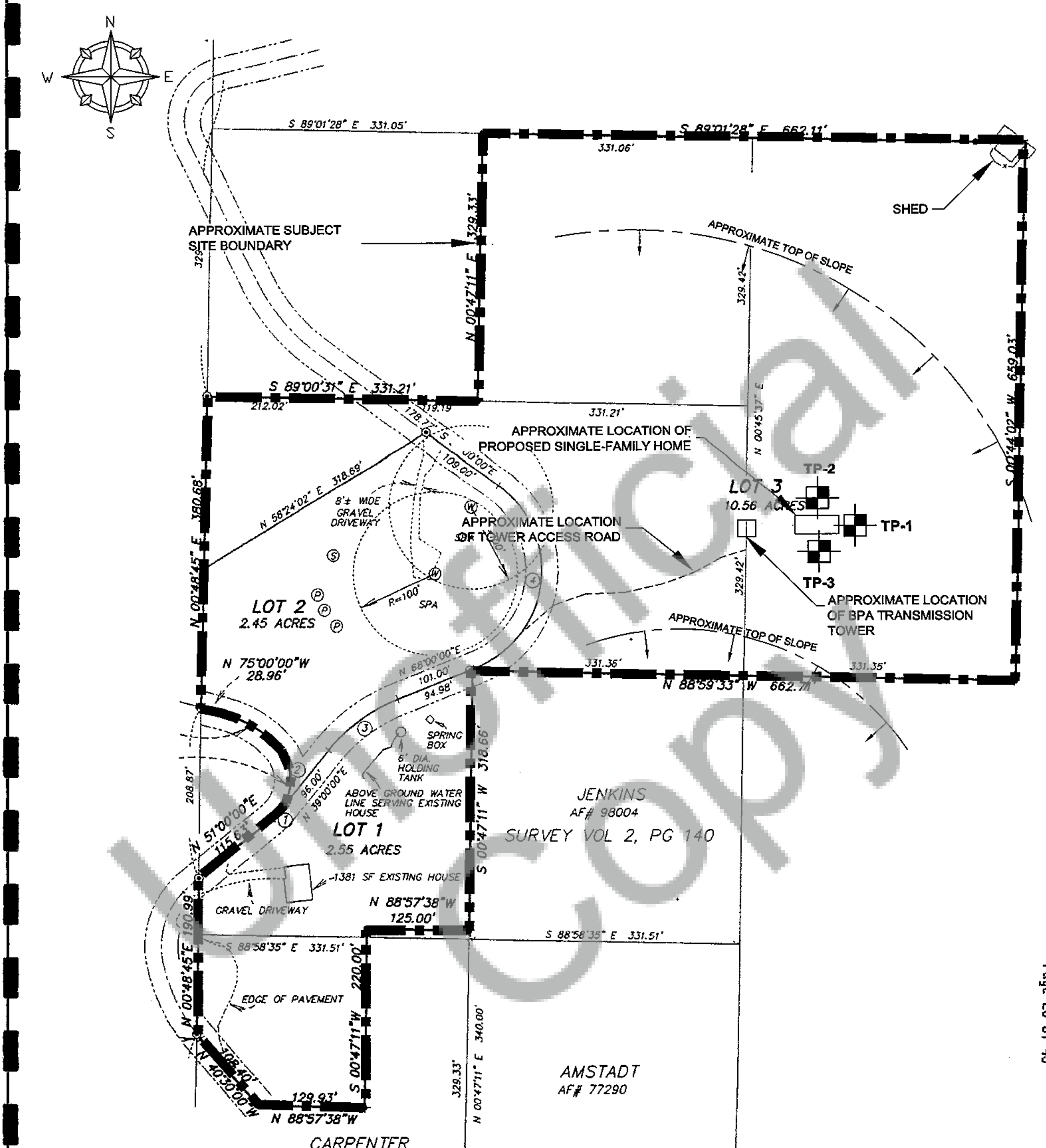
FIGURES



Columbia West Engineering, Inc. 11917 NE 95th Street Vancouver, Washington 98682 p: 360-823-2900 f: 360-823-2901	Design:	Drawn: JLO	SITE LOCATION MAP	FIGURE 1
	Checked: LVL	Date: 10-11-06		
	Client: POND	Rev / By / Date	POND PROPERTY VANCOUVER, WASHINGTON	
	Job No.: 06190			
	CAD File: FIGURE 1			
	Scale: 1:50,000			



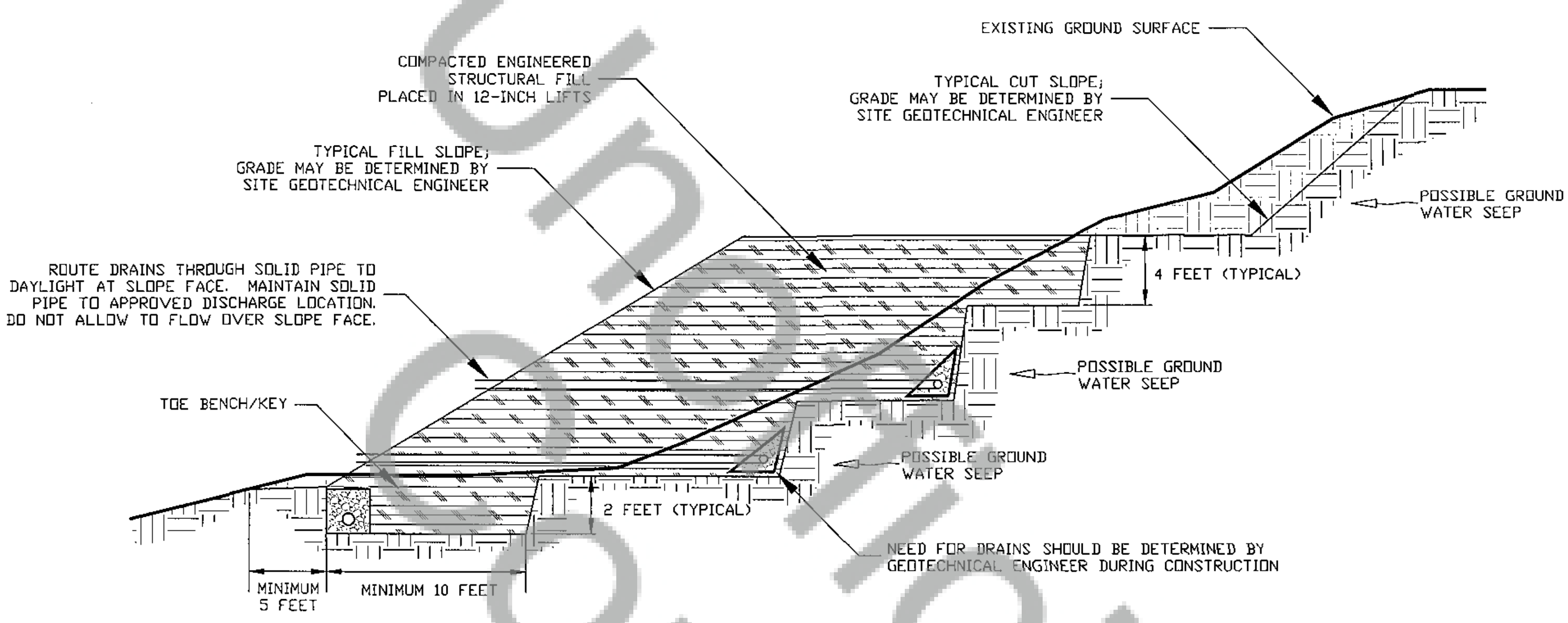
NOTES:
1. DRAWING IS NOT TO SCALE.
2. BASE MAP PROVIDED BY HAGEDORN INC.
3. TEST PIT LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
4. ALL TEST PITS BACKFILLED LOOSELY WITH ON SITE SOIL ON 10-06-06.
5. SUBJECT SITE IS APPROXIMATELY 10.56 ACRES IN SIZE.

APPROXIMATE LOCATION OF TEST PIT EXCAVATION

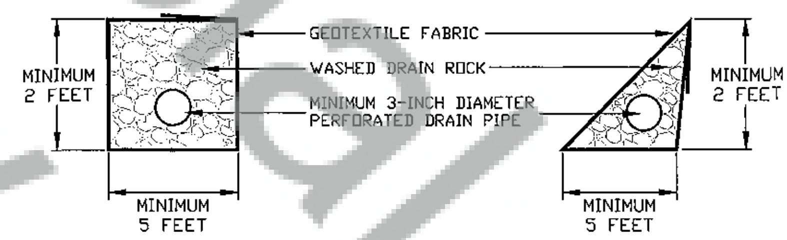
Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901

Design:	Drawn: AJL	TEST PIT LOCATION MAP	FIGURE 2
Checked: LVL	Date: 10-11-06		
Client: MIKE POND	Rev / By / Date	POND PROPERTY STEVENSON, WASHINGTON	
Job No: 06190			
CAD File: FIGURE 2			
Scale: NONE			

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.

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11917 NE 95th Street
Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901

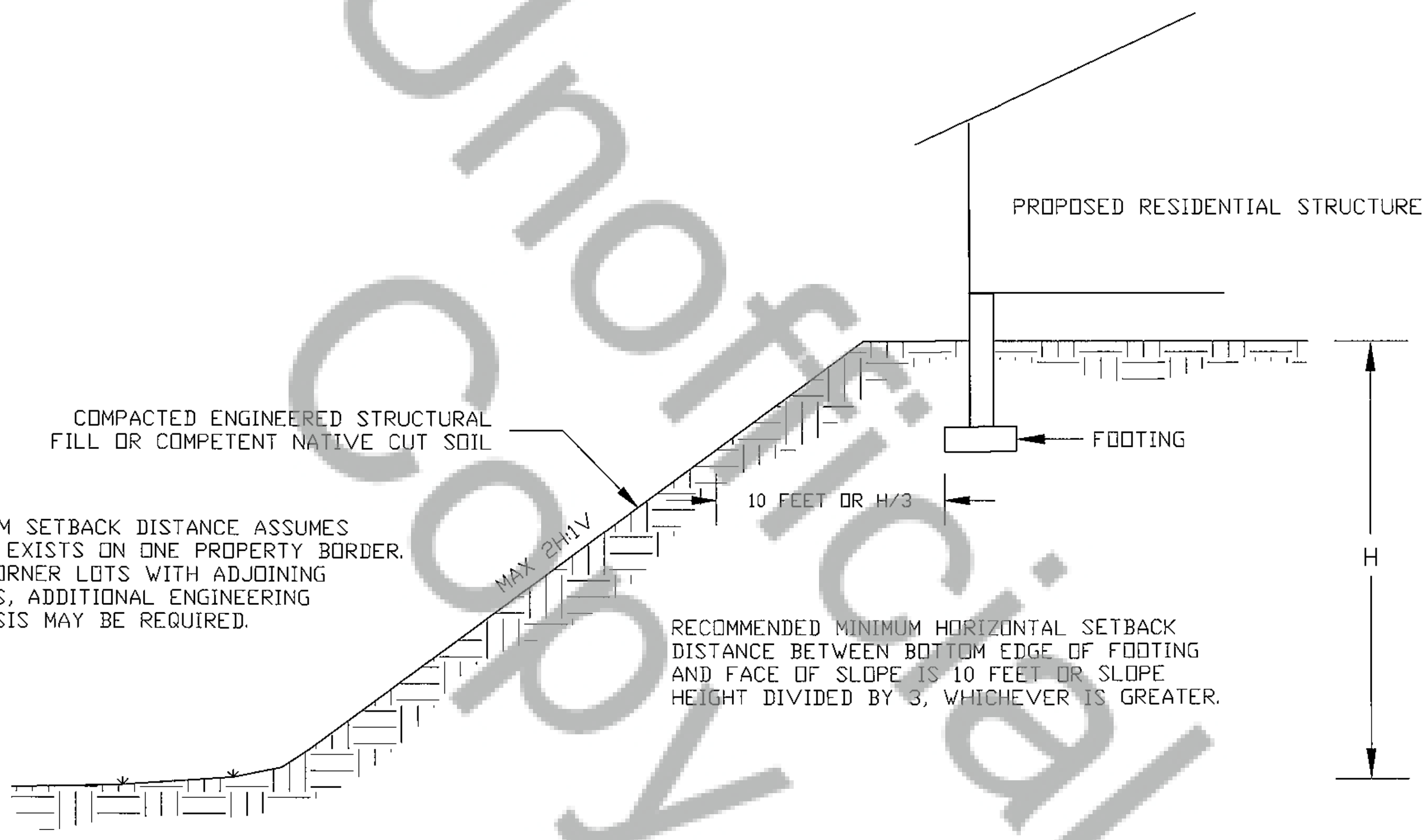
Design:	Drawn: AJL			
Checked: LVL	Date: 11-03-06			
Client: MIKE POND	Rev	By	Date	
Job No: 06190				
CAD File: FIGURE 3				
Scale: NONE				

TYPICAL CUT AND FILL SLOPE SECTION

POND PROPERTY
STEVENSON, WASHINGTON

FIGURE 3

MINIMUM FOUNDATION SLOPE SETBACK DISTANCE



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

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

NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FOUNDATION SETBACK DETAIL, AND MAY NOT BE SITE-SPECIFIC.
4. ALTERNATE SLOPE SETBACK DISTANCES MAY BE APPLICABLE ADJACENT TO STEEP NATIVE SLOPES ONSITE, IF ANY. REFER TO MAIN REPORT TEXT. REPORT SHALL TAKE PRECEDENT OVER FIGURE IN CASE OF CONFLICT.

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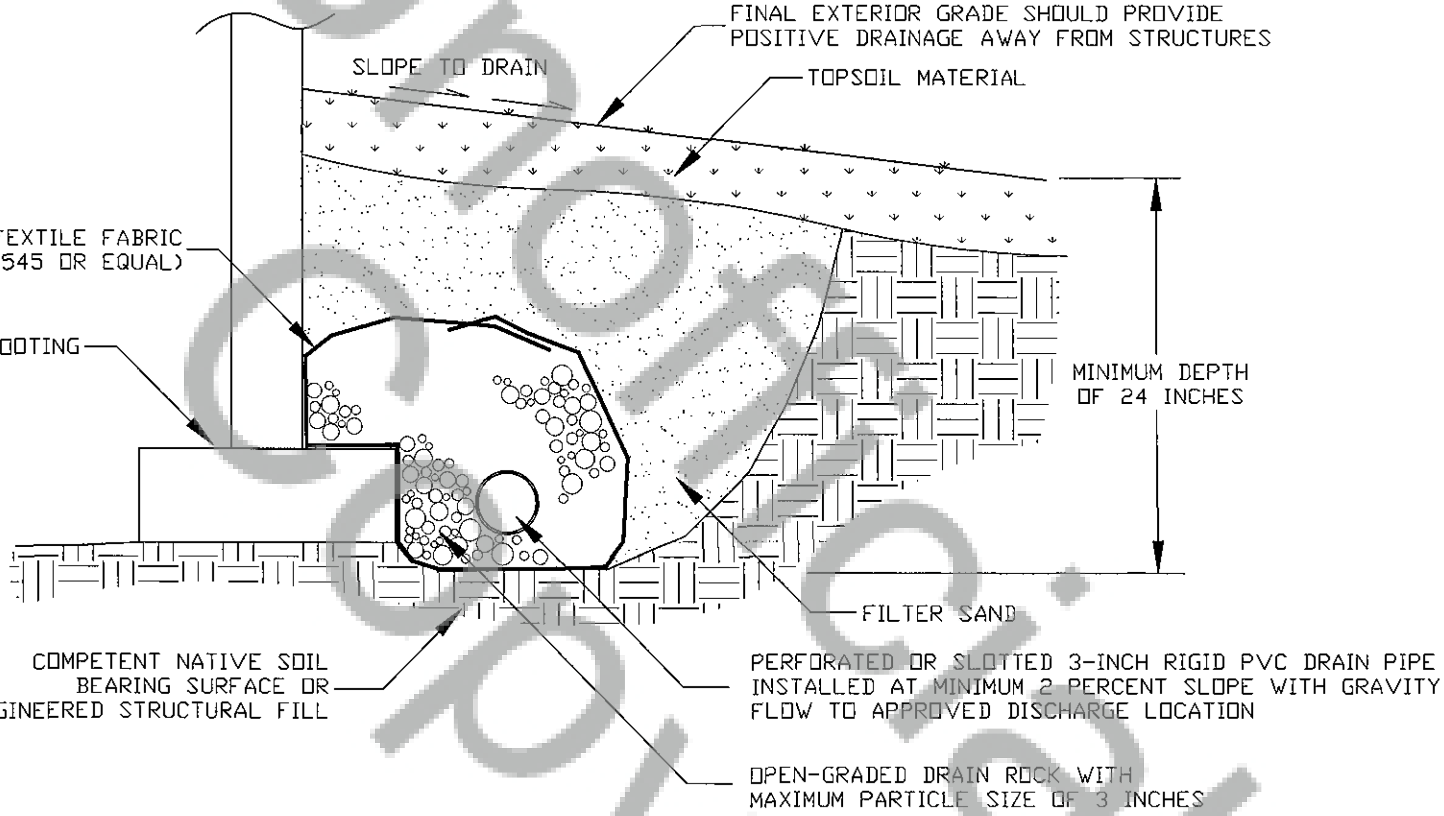
Design:	Drawn: AJL			MINIMUM FOUNDATION SLOPE SETBACK
Checked: LVL	Date: 11-03-06			
Client: MIKE POND	Rev	By	Date	POND PROPERTY STEVENSON, WASHINGTON
Job No: 06190				
CAD File: FIGURE 4				
Scale: NONE				

FIGURE 4

TYPICAL PERIMETER FOOTING DRAIN DETAIL



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

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Vancouver, Washington 98682
p: 360-823-2900 f: 360-823-2901

Design:	Drawn: AJL	FOOTING DRAIN DETAIL	FIGURE 5
Checked: LVL	Date: 11-03-06		
Client: MIKE POND	Rev / By / Date	POND PROPERTY	
Job No: 06190		STEVENSON, WASHINGTON	
CAD File: FIGURE 5			
Scale: NONE			

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

SOIL PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Pond Property Stevenson, Washington	Mr. Michael Pond PO Box 407 Stevenson, Washington 98648	06190	S06-704
		REPORT DATE 10/18/06	FIELD ID TP1.1
		DATE SAMPLED 10/06/06	SAMPLED BY JLO

MATERIAL DATA

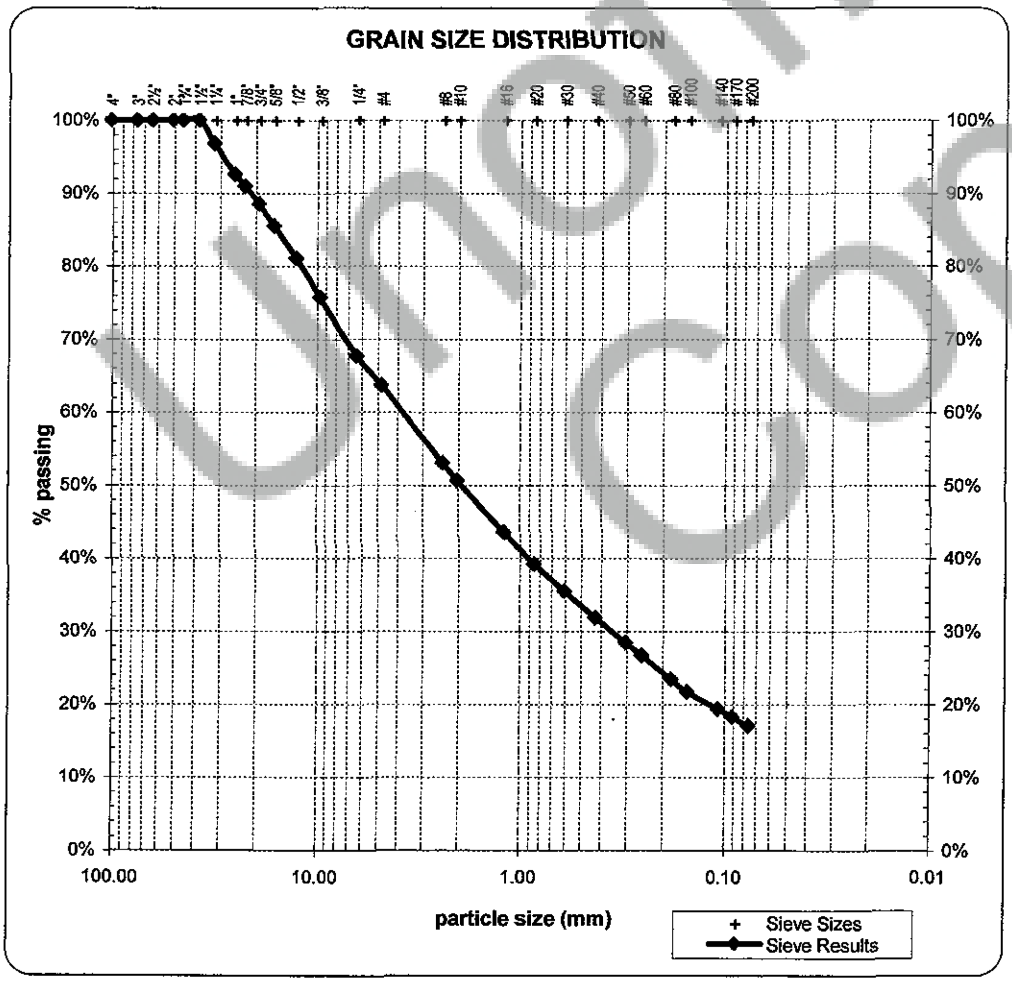
MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty SAND with Gravel	Test Pit TP-01 depth = 3 feet	SM, Silty Sand with Gravel
SPECIFICATIONS none		AASHTO SOIL TYPE A-2-7(0)

LABORATORY TEST DATA

ADDITIONAL DATA

natural moisture content = 23.5%
liquid limit = 49.9%
plastic limit = 29.8%
plasticity index = 20.1%
fineness modulus = 3.90

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = 0.352 mm
$D_{(60)}$ = 3.745 mm



SIEVE DATA

% gravel = 36.4%
% sand = 46.6%
% silt and clay = 17.0%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
	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%		
GRAVEL	1.50"	37.5	100.0%			
GRAVEL	1.25"	31.5		96.8%		
GRAVEL	1.00"	25.0	92.6%			
GRAVEL	7/8"	22.4		90.9%		
GRAVEL	3/4"	19.0	88.5%			
GRAVEL	5/8"	16.0		85.5%		
GRAVEL	1/2"	12.5	81.1%			
GRAVEL	3/8"	9.50		75.7%		
GRAVEL	1/4"	6.30	67.6%			
GRAVEL	#4	4.75	63.6%			
SAND	#8	2.36		53.0%		
SAND	#10	2.00	50.5%			
SAND	#16	1.18		43.5%		
SAND	#20	0.850	39.1%			
SAND	#30	0.600		35.5%		
SAND	#40	0.425	31.8%			
SAND	#50	0.300		28.5%		
SAND	#60	0.250	26.7%			
SAND	#80	0.180		23.5%		
SAND	#100	0.150	21.7%			
SAND	#140	0.106		19.3%		
SAND	#170	0.090		18.2%		
SAND	#200	0.075	17.0%			

DATE TESTED	TESTED BY
10/16/06	JJC

COLUMBIA WEST ENGINEERING, INC. authorized signature

06190 S06-704 silty sand with gravel, 10/18/2006

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Pond Property Stevenson, Washington	Mr. Michael Pond PO Box 407 Stevenson, Washington 98648	06190	S06-704
		REPORT DATE: 10/18/06	FIELD ID: TP1.1
		DATE SAMPLED: 10/06/06	SAMPLED BY: JLO

MATERIAL DATA

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

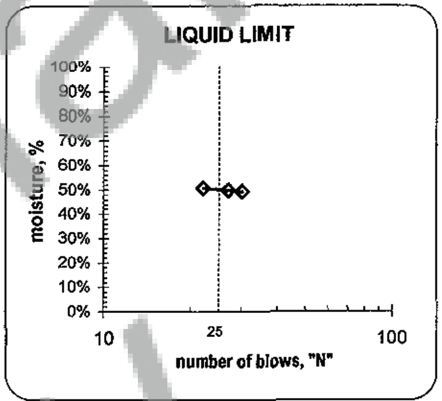
LABORATORY TEST DATA

ATTERBERG LIMITS

liquid limit = 49.9%
plastic limit = 29.8%
plasticity index = 20.1%

LIQUID LIMIT DETERMINATION

	1	2	3	4
wet soil + pan weight, g =	13.58	14.10	10.95	
dry soil + pan weight, g =	9.51	9.83	7.69	
pan weight, g =	1.22	1.22	1.22	
N (blows) =	30	27	22	
moisture, % =	49.1 %	49.6 %	50.4 %	

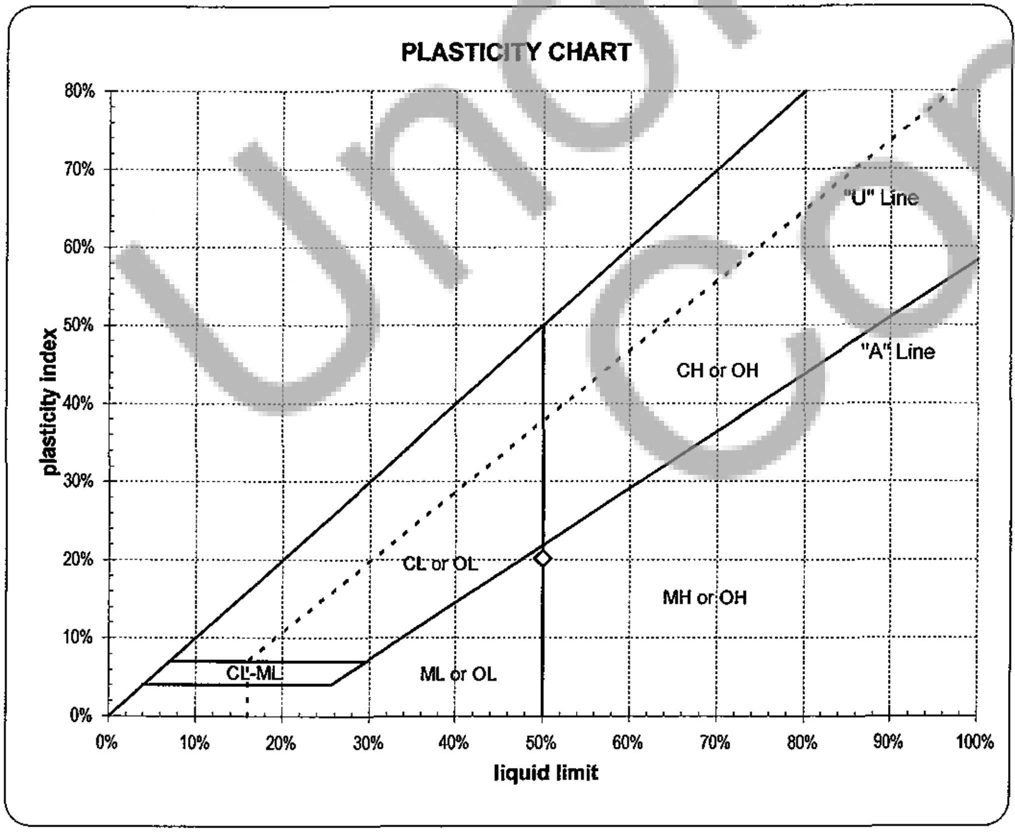


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	1	2	3	4
wet soil + pan weight, g =	9.47	6.95		
dry soil + pan weight, g =	7.59	5.62		
pan weight, g =	1.21	1.21		
moisture, % =	29.5 %	30.2 %		



ADDITIONAL DATA

% gravel = 36.4%
% sand = 46.6%
% silt and clay = 17.0%
% silt = n/a
% clay = n/a
moisture content = 23.5%

DATE TESTED	TESTED BY
10/16/06	JJC

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06190 S06-704 silty sand with gravel, 10/18/2006

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SOIL PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Pond Property Stevenson, Washington	Mr. Michael Pond PO Box 407 Stevenson, Washington 98648	06190	S06-705
		REPORT DATE: 10/18/06	FIELD ID: TP2.2
		DATE SAMPLED: 10/06/06	SAMPLED BY: JLO

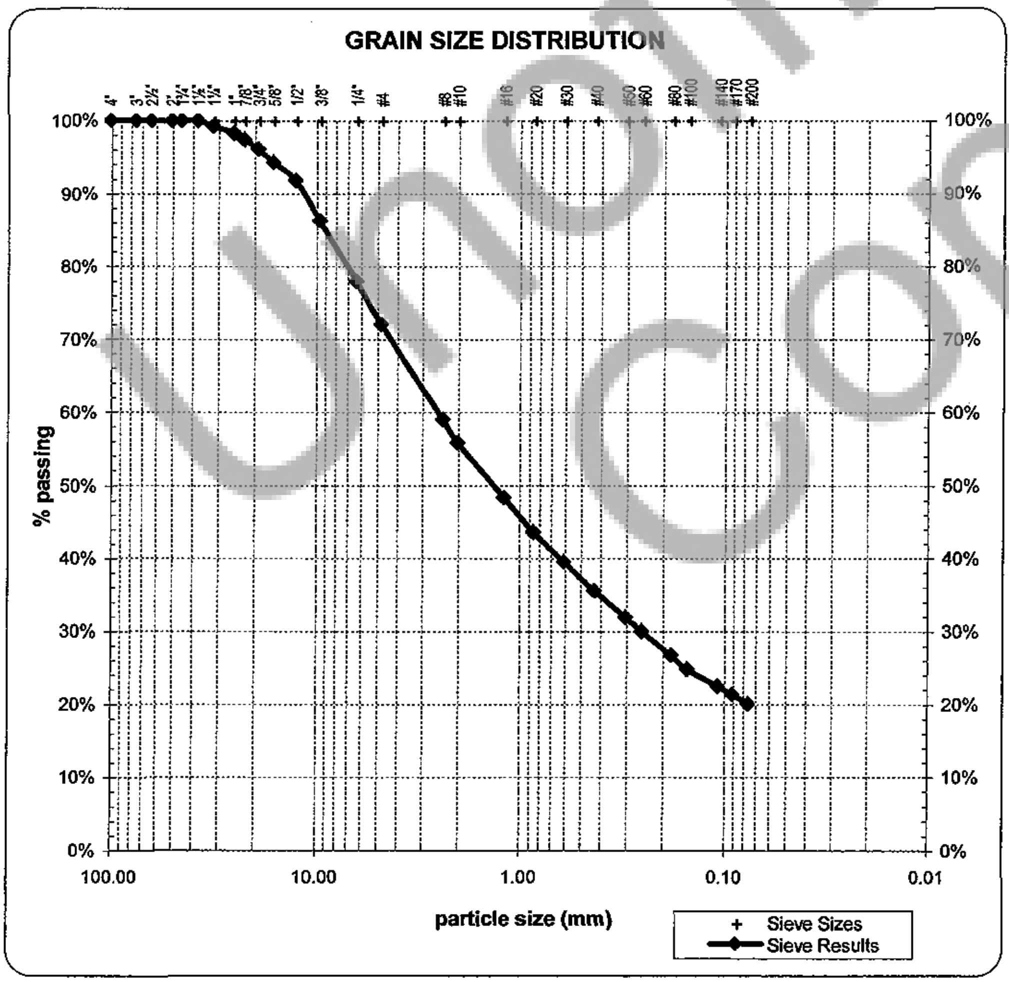
MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty SAND with Gravel	Test Pit TP-02 depth = 13 feet	SM, Silty Sand with Gravel
SPECIFICATIONS: none		AASHTO SOIL TYPE: A-2-7(0)

LABORATORY TEST DATA

ADDITIONAL DATA

natural moisture content =	40.1%	coefficient of curvature, C_C =	n/a
liquid limit =	53.4%	coefficient of uniformity, C_U =	n/a
plastic limit =	33.6%	effective size, $D_{(10)}$ =	n/a
plasticity index =	19.9%	$D_{(30)}$ =	0.250 mm
fineness modulus =	3.42	$D_{(60)}$ =	2.496 mm



SIEVE DATA

% gravel = 28.0%
% sand = 52.0%
% silt and clay = 20.1%

	SIEVE SIZE US	mm	PERCENT PASSING SIEVE act.	interp.	SPECS max	min
	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%		
GRAVEL	1.50"	37.5	100.0%			
	1.25"	31.5		99.3%		
	1.00"	25.0	98.3%			
	7/8"	22.4		97.4%		
	3/4"	19.0	96.1%			
	5/8"	16.0		94.3%		
	1/2"	12.5	91.8%			
	3/8"	9.50		86.3%		
	1/4"	6.30	78.0%			
	#4	4.75	72.0%			
SAND	#8	2.36		59.0%		
	#10	2.00	55.9%			
	#16	1.18		48.3%		
	#20	0.850	43.6%			
	#30	0.600		39.5%		
	#40	0.425	35.5%			
	#50	0.300		31.9%		
	#60	0.250	30.0%			
	#80	0.180		26.7%		
	#100	0.150	24.8%			
	#140	0.106		22.4%		
	#170	0.090		21.3%		
	#200	0.075	20.1%			

DATE TESTED	TESTED BY
10/16/06	JJC

COLUMBIA WEST ENGINEERING, INC. authorized signature

06190 S06-705 silty sand with gravel, 10/18/2006

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

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Phone: 360-823-2900, Fax: 360-823-2901
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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Pond Property Stevenson, Washington	Mr. Michael Pond PO Box 407 Stevenson, Washington 98648	06190	S06-705
		REPORT DATE 10/18/06	FIELD ID TP2.2
		DATE SAMPLED 10/06/06	SAMPLED BY JLO

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty SAND with Gravel	Test Pit TP-02 depth = 13 feet	SM, Silty Sand with Gravel

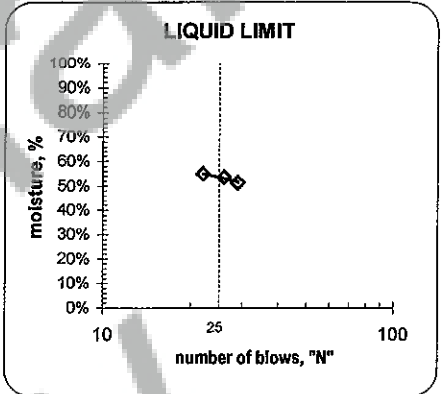
LABORATORY TEST DATA

ATTERBERG LIMITS

liquid limit =	53.4%
plastic limit =	33.6%
plasticity index =	19.9%

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	13.72	13.34	12.62	
dry soil + pan weight, g =	9.49	9.12	8.59	
pan weight, g =	1.23	1.22	1.22	
N (blows) =	29	26	22	
moisture, % =	51.2 %	53.4 %	54.7 %	

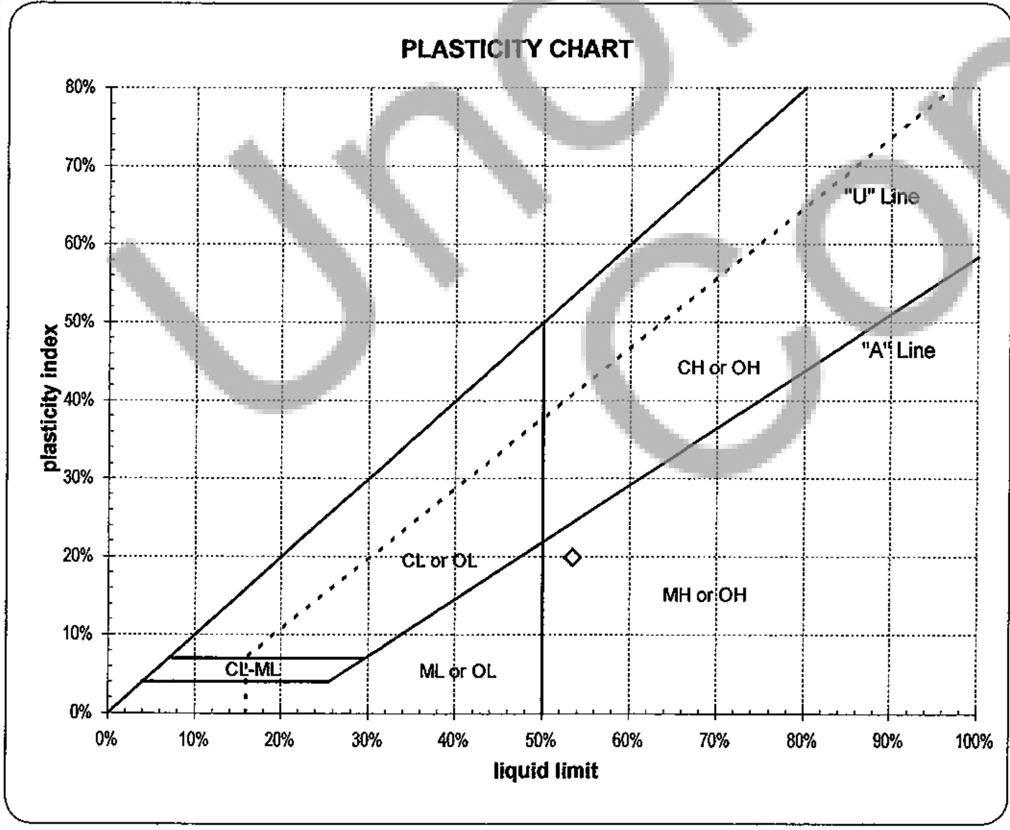


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	10.76	10.10		
dry soil + pan weight, g =	8.35	7.87		
pan weight, g =	1.20	1.20		
moisture, % =	33.7 %	33.4 %		



ADDITIONAL DATA

% gravel =	28.0%
% sand =	52.0%
% silt and clay =	20.1%
% silt =	n/a
% clay =	n/a
moisture content =	40.1%

DATE TESTED	TESTED BY
10/16/06	JJC

06190 S06-705 silty sand with gravel, 10/18/2006

COLUMBIA WEST ENGINEERING, INC. authorized signature

APPENDIX B
TEST PIT EXCAVATION LOGS

Columbia West Engineering, Inc.
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TEST PIT LOG

PROJECT NAME	PROJECT NO.	LOGGED BY	TEST PIT NO.
Pond Property	06190	JLO	TP-1

PROJECT LOCATION	APPROXIMATE SURFACE ELEVATION	DATE	START TIME	FINISH TIME
Stevenson, Washington	710 feet	10-6-06	0950	1030

Depth	Field ID and Type	Estimated SPT N Value	Unconfined Compression Strength (q_u)	Torvane Shear Strength	SCS Soil Survey Description	USCS Soil Type	AASHTO Soil Type	Moisture Content (%)	SOIL DESCRIPTION AND REMARKS (Include soil description, organics, color, moisture, density, gradation, depth to ground water or other characteristics)
0–1 ft									Approximately 12 inches of dark brown organic topsoil with abundant tree roots.
1 ft					Stevenson loam				Light-brown, moist, loose to medium-dense, silty SAND with gravel and some sub-angular to rounded cobbles.
2 ft									
3 ft	TP1.1 (bag)	7				SM	A-2-7(0)	24	Sample TP1.1 was collected from a depth of 3 feet and submitted for laboratory analysis: moisture content = 23.5%, passing No. 200 sieve = 17.0%, liquid limit = 50, plasticity index = 20 Results of nuclear density testing performed at 3 feet: wet density = 81.8 pcf, dry density = 59.2 pcf moisture content = 38.1%, void ratio = 1.846
4 ft									
5 ft									
6 ft									
7 ft									
8 ft									
9 ft									
10 ft									Increasing moisture content.
11 ft									
12 ft									Bottom of test pit at 12 feet. No ground water encountered. Test pit loosely backfilled with onsite soil on October 6, 2006.
13 ft									
14 ft									
15 ft									
16 ft									
17 ft									
18 ft									
19 ft									
20 ft									

NOTES: Unless otherwise noted, all soil parameters are approximate and are based upon field observation. Estimated SPT blow count N value based upon dynamic cone penetrometer testing. Unconfined compression strength based upon pocket penetrometer testing. Shear strength based upon pocket torvane testing. N/A = Not Applicable or Not Tested.

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TEST PIT LOG

PROJECT NAME	PROJECT NO.	LOGGED BY	TEST PIT NO.
Pond Property	06190	JLO	TP-2

PROJECT LOCATION	APPROXIMATE SURFACE ELEVATION	DATE	START TIME	FINISH TIME
Stevenson, Washington	710 feet	10-6-06	1035	1110

Depth	Field ID and Type	Estimated SPT N Value	Unconfined Compression Strength (q_u)	Torvane Shear Strength	SCS Soil Survey Description	USCS Soil Type	AASHTO Soil Type	Moisture Content (%)	SOIL DESCRIPTION AND REMARKS (Include soil description, organics, color, moisture, density, gradation, depth to ground water or other characteristics)
0–1 ft									Approximately 12 inches of dark brown organic topsoil.
1 ft					Stevenson loam				Light brown, moist to wet, silty SAND with gravel and some sub angular to rounded cobbles.
2 ft									
3 ft									
4 ft		8						38	Results of nuclear density testing performed at 4 feet: wet density = 61.9 pcf, dry density = 85.5 pcf moisture content = 38.2%, void ratio = 1.724
5 ft									
6 ft									
7 ft									Encountered red mottles.
8 ft									
9 ft									
10 ft									Increasing moisture content.
11 ft									
12 ft									Wet to saturated soil.
13 ft	TP2.2					SM	A-2-7(0)	40	Sample TP2.2 was collected from a depth of 13 feet and submitted for laboratory analysis: moisture content = 40.1%, passing No. 200 sieve = 20%, liquid limit = 53, plasticity index = 20
14 ft									Bottom of test pit at 13 feet. No ground water encountered. Test pit loosely backfilled with onsite soil on October 6, 2006.
15 ft									
16 ft									
17 ft									
18 ft									
19 ft									
20 ft									

NOTES: Unless otherwise noted, all soil parameters are approximate and are based upon field observation. Estimated SPT blow count N value based upon dynamic cone penetrometer testing. Unconfined compression strength based upon pocket penetrometer testing. Shear strength based upon pocket torvane testing. N/A = Not Applicable or Not Tested.

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TEST PIT LOG

PROJECT NAME	PROJECT NO.	LOGGED BY	TEST PIT NO.
Pond Property	06190	JLO	TP-3

PROJECT LOCATION	APPROXIMATE SURFACE ELEVATION	DATE	START TIME	FINISH TIME
Stevenson, Washington	710 feet	10-6-06	1120	1150

Depth	Field ID and Type	Estimated SPT N Value	Unconfined Compression Strength (q_u)	Torvane Shear Strength	SCS Soil Survey Description	USCS Soil Type	AASHTO Soil Type	Moisture Content (%)	SOIL DESCRIPTION AND REMARKS (Include soil description, organics, color, moisture, density, gradation, depth to ground water or other characteristics)
0–1 ft									Approximately 12 to 18 inches of dark brown organic topsoil with several tree roots.
2 ft					Stevenson loam	SM	A-2	25	Light brown, moist, medium-dense, silty SAND with gravel. Red to light gray mottles. Roots to six feet.
3 ft									
4 ft									
5 ft									
6 ft									
7 ft									Encountered cobbles up to 8 inches.
8 ft									
9 ft									
10 ft									
11 ft									
12 ft									
13 ft									Bottom of test pit at 12.5 feet. No ground water encountered. Test pit loosely backfilled with onsite soil on October 6, 2006.
14 ft									
15 ft									
16 ft									
17 ft									
18 ft									
19 ft									
20 ft									

NOTES: Unless otherwise noted, all soil parameters are approximate and are based upon field observation. Estimated SPT blow count N value based upon dynamic cone penetrometer testing. Unconfined compression strength based upon pocket penetrometer testing. Shear strength based upon pocket torvane testing. N/A = Not Applicable or Not Tested.

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APPENDIX C
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.

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

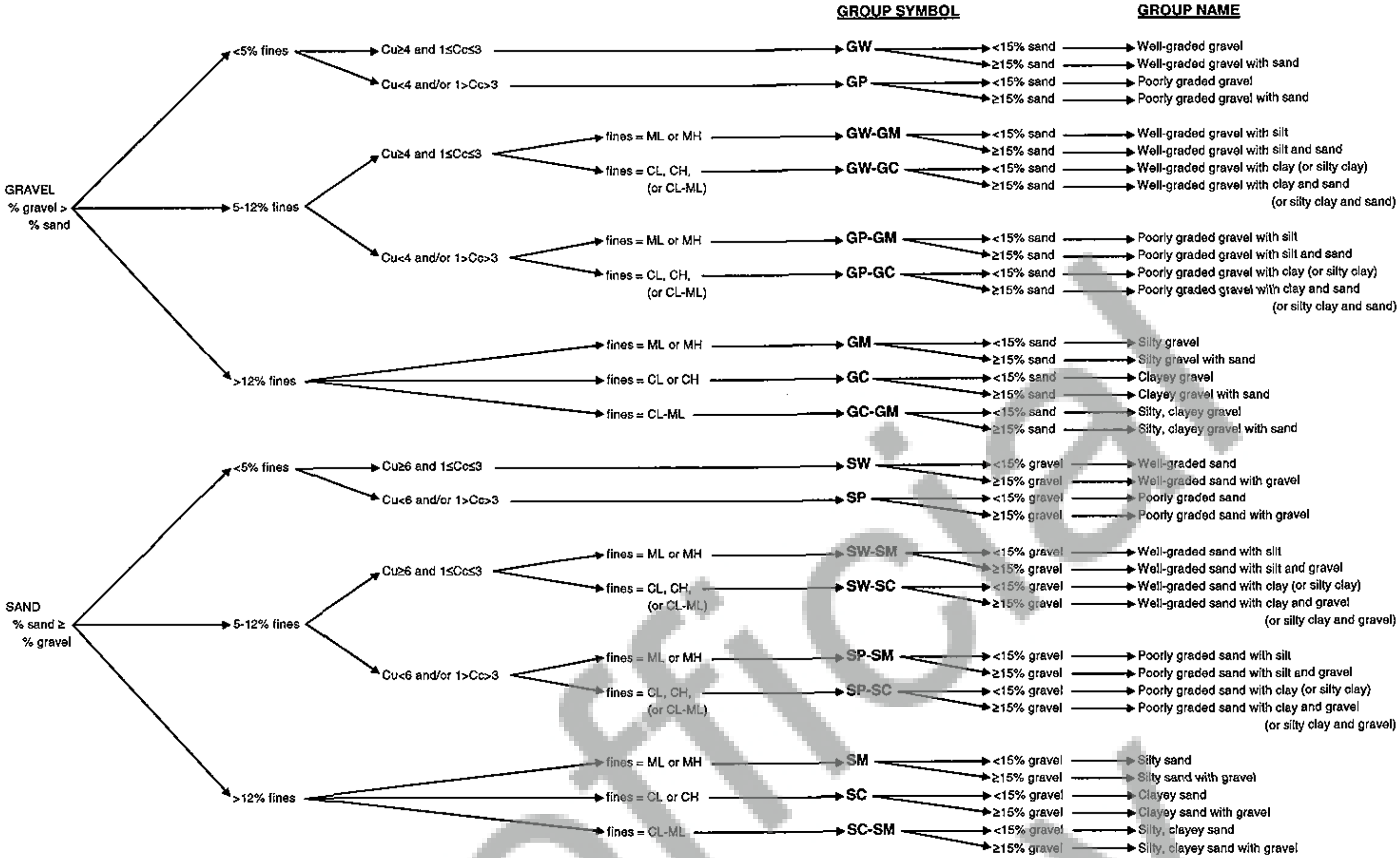
General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00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

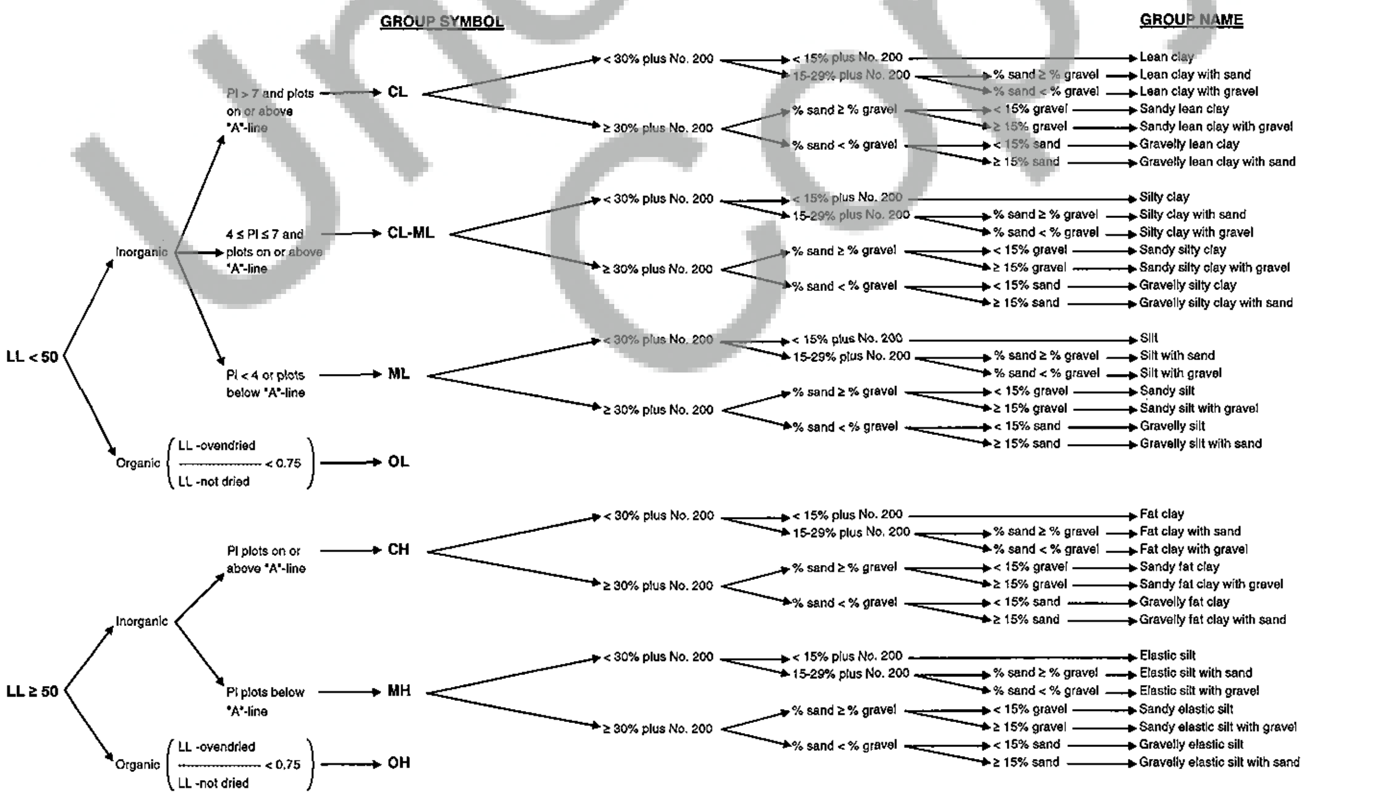
AASHTO = American Association of State Highway and Transportation Officials

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)

APPENDIX D
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Columbia West Engineering, Inc.

Date: November 16, 2006
Project: Pond Property, Stevenson, 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.

Geotechnical and Environmental Engineering and Consulting Services
11917 NE 95th Street Vancouver, Washington 98682 • Phone: 360-823-2900, Fax: 360-823-2901

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.

EXHIBIT 'A'

A portion of the Southwest Quarter of the Southeast Quarter of Section 26, Township 3 North, Range 7 East of the Willamette Meridian, in the County of Skamania, State of Washington, described as follows:

The West Half of the Northeast Quarter of the Southwest Quarter of the Southeast Quarter; the East Half of the Northwest Quarter of the Southwest Quarter of the Southeast Quarter; the West Half of the Northwest Quarter of the Southwest Quarter of the Southeast Quarter. Excepting that portion lying North and East of County Road which crosses said tract diagonally in a Northwesterly and Southwesterly directions, conveyed to Curtis L. Samsel by Deed recorded at Page 126, Volume 30, Records of Skamania County, Washington.

The West Half of the Southwest Quarter of the Southwest Quarter of the Southeast Quarter. Excepting therefrom that portion conveyed to William D. Carpenter, et ux, by Deed recorded June 1, 1966 in Auditor File No. 66966, Skamania County Deed Records.

Also except that portion lying West of Aalvik Road as conveyed to Louis L. Wellman, et ux, by Deed recorded July 16, 1981 in Book 80 of Deeds, Page 18, Auditor File No. 92791, Skamania County Records.