Columbia West Engineering, Inc.
Innovative Solutions for the New Millennium

Geotechnical Site Investigation
Highland Terrace
La Center, Washington

September 30, 2005

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GEOTECHNICAL SITE INVESTIGATION
HIGHLAND TERRACE
LA CENTER, WASHINGTON

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1.0 INTRODUCTION

Columbia West Engineering, Inc. was retained by Altius Corporation to conduct a geotechnical site investigation for Highland Terrace, a proposed single-family residential subdivision located in La Center, Washington. The purpose of the investigation was to observe and assess subsurface soil conditions at specific locations and provide subsequent appropriate geotechnical engineering analyses to support property development, planning, and design recommendations. The specific scope of services was outlined in a proposal contract dated August 12, 2005 and authorized by client signature on August 12, 2005. 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 5.17 and Appendix D.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located south of the intersection of NW Bolen Street and NW 14th Avenue near Pacific Highway in La Center, Washington. The regulatory jurisdictional agency is the City of La Center, Washington. The approximate latitude and longitude are N 45° 52' 17" and W 122° 41' 10" and the legal description is a portion of the SE ¼ of Section 33, T5N, R1E, Willamette Meridian. The site includes several contiguous parcels that collectively comprise approximately 20 acres.

1.2 Proposed Development

Preliminary correspondence with the client indicates the site is planned for future single-family residential development. Columbia West has not reviewed a preliminary grading plan, but understands cut and fill areas are likely. This report is based upon proposed development as described above and may not be applicable if modified.

2.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

The subject site lies within the Willamette Valley/Puget Sound Lowland, a wide physiographic depression flanked by the mountainous Coast Range on the west and the Cascade Range on the east. Inclined or uplifted structural zones within the Willamette Valley/Puget Sound Lowland constitute highland areas and depressed structural zones form sediment-filled basins. The site is located near the north end of the Portland/Vancouver Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide.

According to 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)*, near-surface soils are expected to consist of upper-Pleistocene, fine-textured, rhythmically bedded periglacial deposits derived from catastrophic outburst floods of Glacial

Lake Missoula (Qs) and Pleistocene-Miocene sedimentary deposits of the Troutdale Formation (QTtd).

The *Soil Survey of Clark County, Washington (United States Department of Agriculture, Soil Conservation Service [USDA SCS], November 1972)* identifies surface soils as Hesson clay loam Gee silt loam, Cove silty clay loam, and Odne silt loam. These soils are generally fine-textured and have slow to moderate permeability, high water capacity, and slight to severe erosion hazard if left in a bare unvegetated condition. The shrink/swell potential is moderate, the shear strength is low, and the soils are compressible and moisture sensitive. Odne and Cove soils are generally hydric and 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 very large earthquake event with strong localized ground movement may be underestimated. Earthquakes in the Pacific Northwest have been shown to cause landslides and ground subsidence, in addition to severe flooding near coastal areas. Earthquakes may also induce soil liquefaction, which occurs when elevated horizontal ground acceleration and velocity cause soil particles to interact as a fluid as opposed to a solid. Liquefaction of soil can result in lateral spreading and temporary loss of bearing capacity and shear strength.

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

Portland Hills Fault Zone

The Portland Hills Fault Zone consists of several northwest-trending faults located along the eastern boundary of the Portland Hills. The fault zone is approximately 25 miles in length and is located nearly 20 miles from 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. Additionally, a M3.5 earthquake possible associated with the fault zone occurred approximately 1.3 miles east of the fault in 1991. Therefore, the Portland Hills Fault Zone is generally thought to be potentially active and capable of producing possible damaging earthquakes.

Gales Creek-Newberg-Mt. Angel Fault Zone

Located approximately 35 miles southwest of the site, the 50-mile long Gales Creek-Newberg-Mt. Angel Structural Zone consists of a series of discontinuous northwest-trending faults. As defined by *Seismic Design Mapping, State of Oregon, Geomatrix Consultants, 1995,* possible late-Quaternary geomorphic surface deformation may exist along the structural zone. Although no definitive evidence of impacts to Holocene sediments has reportedly been observed, an 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 southeast of the site. According to *Geology and Groundwater Conditions of Clark County Washington, USGS Water Supply Paper 1600, Mundorff, 1964* and *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. According to *Seismic Design Mapping, State of Oregon, Geomatrix Consultants, 1995*, 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.

4.0 GEOTECHNICAL FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance and nine test pit excavations (TP-01 through TP-09) was conducted at the site. Exploration was conducted with a track-mounted excavator. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications. Several field tests were performed, including nuclear density gauge, torvane shear, dynamic cone penetrometer, and pocket penetrometer tests. As indicated on the test pit logs, the field tests provided measurements and estimates of relative density, shear strength, and in situ penetration resistance. Soil samples were also collected from relevant soil horizons and submitted for laboratory analysis. Sample results are presented in Appendix A. Test pit locations are indicated on Figure 2 and soil logs are presented in Appendix B. Soil description and classification information is 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 270 feet above mean sea level at the north end of the site to approximately 180 feet near the southern border. The northern portion of the site is relatively flat. Gradual south-trending downgradient slopes with grades ranging from approximately 5 to 15 percent begin near the center of the site and characterize most of the southern portion of the property. Several wetlands have been delineated within the low areas near the south end of the site adjacent to Pacific Highway.

Several existing residential homes and associated detached outbuildings are located in the central portion of the site. The remaining areas are undeveloped. Vegetation consists primarily of open deforested grassland. Several native deep-rooted conifer and deciduous trees of various sizes are located near the houses.

4.2 Subsurface Exploration and Investigation

Nine test pits were excavated at the site to a maximum depth of 22 feet on September 13, 2005. The test pit locations were selected to observe subsurface soil characteristics in proximity to proposed structures and developed areas.

4.2.1 Soil Type Description

The field investigation indicated the site is generally covered with a topsoil layer approximately 8 to 12 inches thick. Underlying the topsoil, subsurface soils resembling the USDA Gee and Hesson soil series descriptions were generally encountered throughout the site. Soils resembling the Cove silt loam description were encountered in one test pit (TP-06) excavated in the northwest corner of the site. Hydric soils resembling the Odne silt loam description were not encountered but may exist adjacent to wetland areas located in the southern portion of the site. Subsurface lithology may generally be described by soil types identified in the following text.

Soil Type 1 – Lean CLAY with sand

Soil Type 1 was observed to consist of brown, damp to moist, very stiff to hard, lean CLAY with sand. Soil Type 1 was consistent with the USDA Gee and Hesson soil descriptions and was encountered below the topsoil layer in all test pits. Soil Type 1 typically extended to depths ranging from 2½ feet to 8 feet. Field estimates and laboratory analysis indicated the in situ moisture content, expressed as the ratio of weight of water to weight of dry soil, varied from approximately 12 to 20 percent, with lower moistures generally closer to the ground surface. Analytical laboratory testing conducted upon a representative sample of Soil Type 1 indicated approximately 81 percent by weight passing the No. 200 sieve. Atterberg Limits analysis resulted in a liquid limit of 26 and a plasticity index of 8. Soil Type 1 is classified CL according to USCS specifications.

Soil Type 1A – Fat CLAY

Soil Type 1A was observed to consist of light gray, moist, very stiff, fat CLAY. Soil Type 1A was observed underlying Soil Type 1 in test pit TP-06 from depths of approximately 2½ to 5 feet.

Analytical laboratory testing conducted upon a representative sample of Soil Type 1A indicated an in situ moisture content of approximately 29 percent and approximately 91 percent by weight passing the No. 200 sieve. Atterberg Limits analysis resulted in a liquid limit of 63 and a plasticity index of 45. This indicates Soil Type 1A is highly plastic with high expansion potential. Soil Type 1A is classified CH according to USCS specifications.

Soil Type 2 – Lean CLAY

Soil Type 2 was observed to consist of brown with various mottling, moist, very stiff to hard, lean CLAY. Soil Type 2 was encountered underlying Soil Types 1 and 1A at depths of 2.5 to 8 feet in four test pits (TP-01, TP-03, TP-04, and TP-06) and extended to depths ranging from 9 to 17 feet, the maximum depth explored in several test pits. The transition from Soil Type 1 to Soil Type 2 was gradual. Field estimates and laboratory analysis indicate the in situ moisture content varied from approximately 23 to 30 percent. Analytical laboratory testing conducted upon a representative sample of Soil Type 2 indicated approximately 92 percent by weight passing the No. 200 sieve, a liquid limit of 37, and a plasticity index of 18. Soil Type 2 is classified CL according to USCS specifications.

Soil Type 2A – Sandy CLAY

Soil Type 2A was observed to consist of brown with various mottling, moist, very stiff to hard, sandy CLAY. Soil Type 2A was encountered underlying Soil Type 1 at depths of 3 to 4 feet in test pits TP-05 and TP-08 and extended to depths of 8 to 11 feet. The transition from Soil Type 1 to Soil Type 2A was gradual. Laboratory analysis indicated the in situ moisture content varied from approximately 20 to 35 percent. Soil Type 2A is classified CL according to USCS specifications.

Soil Type 3 – Sedimentary Conglomerate of the Troutdale Formation

Soil Type 3 was observed to consist of brown and reddish brown with various mottling, moist, sedimentary conglomerate of clay, silt, sand and gravel. Soil Type 3 most likely represents severely to completely weathered, consolidated, Pleistocene-Miocene sedimentary deposits of the Troutdale Formation. The Troutdale Formation is defined by 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)* as well-indurated to weakly consolidated, vitric sand and quartzite-bearing gravel, fine-grained sand, silt and clay, micaceous quartzose sand, and cobbly and bouldery vitric sand.

Soil Type 3 was observed underlying Soil Types 2 and 2A in four test pits (TP-02, TP-03, TP-05, TP-08, and TP-09) at depths of 7 to 11 feet and extended to the maximum depth of excavation. Laboratory analysis indicated the in situ moisture content varied from approximately 22 to 48 percent. Analytical laboratory testing conducted upon representative samples of Soil Type 3 indicated approximately 41 to 54 percent by weight passing the No. 200 sieve. Atterberg Limits analysis resulted in a liquid limit of 33 percent and a plasticity index of 15 percent.

4.2.2 Ground Water

Ground water seeps were encountered during the field investigation at a depth of 17 feet in test pit TP-02. Ground water was not encountered in any other test pits. According to *Ground Water Data for the Portland Basin, Oregon and Washington, USGS, 1990, Open-File Report 90-126*, static ground water at nearby wells has been observed at depths varying from approximately 50 to 105 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 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 fine-textured soils, drainage, and potential expansive soils. The following text sections present design recommendations.

5.1 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 for most of the site is anticipated to be approximately 8 to 12 inches. The required stripping depth may increase in areas with heavy vegetation, large trees, 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 or undocumented fill encountered at the site 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. At least three septic systems are known to be present at the site. 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 *2003 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.

5.1.1 Over-Excavation of Expansive Soils

As described previously, a layer of gray fat clay (Soil Type 1A) was encountered in the northwest portion of the site at a depth of approximately 2½ feet below ground surface. Because the fat clay has a high expansion potential, foundations or structures that bear directly upon the fat clay layer may be subject to heave. Therefore, it is recommended that the top of the fat clay layer be at least two feet below the base of the lowest foundation elevation. Depending upon the final grading plan, this may require over-excavation of the fat clay layer and replacement with non-expansive structural fill or additional fill placement on existing soils overlying the fat clay layer. Native lean clay soils would be suitable for structural fill placement above the fat clay layer. Fill should be placed in accordance with recommendations outlined below.

Exposed fat clay should be prevented from substantial drying and should be moisture-conditioned prior to placement of fill above it to reduce future expansion potential. Excavated fat clay material may be reused as structural fill elsewhere on the site if thoroughly mixed with native lean sandy clays, properly moisture conditioned, and compacted. Grading activities in expansive soil areas should be closely observed by a licensed geotechnical engineer or designated representative.

5.2 Engineered Structural Backfill

For areas proposed for fill placement, the surface should be appropriately prepared as described in Section 5.1, *Site Preparation and Grading*. 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 fine-textured native 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 often difficult to excavate and nearly impossible to compact during wet weather conditions. If adequate compaction is not achievable with clean native soils, import structural fill consisting of well-graded granular material with a maximum particle size of three inches and no more than five percent passing the No. 200 sieve is recommended.

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

5.3 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. Figure 3 presents a typical fill slope cross-section. Drainage implementations, including subdrains or perforated drain pipe trenches, may also be necessary for installation 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 and 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 slope face 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 *2003 IBC, Section J109.*

Fill slopes should be constructed by placing fill material in maximum 12-inch level lifts, compacting as described in Section 5.2, *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.

5.4 Foundations

Foundations for proposed residential structures are anticipated to consist of shallow continuous perimeter footings or spread footings. If slabs or basements are proposed, additional geotechnical engineering is recommended. Footing design should conform to requirements specified in the *2003 IBC, Table 1805.4.2, Footings Supporting Walls of Light-Frame*

Construction, with exceptions as noted. Footings should bear upon firm native soil or engineered structural fill.

To evaluate bearing capacity for proposed structures, serviceability and reliability of shear resistance for subsurface soils was considered. Allowable bearing capacity is typically a function of footing dimension and subsurface soil properties, including settlement and shear resistance. Based upon in situ field testing and laboratory analysis, the estimated allowable bearing capacity for 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 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.

5.5 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.

5.6 Excavation

Subsurface soils at the site were excavated to a maximum depth of 22 feet with conventional earthmoving equipment during field exploration activities. Volcanic igneous bedrock was not encountered and blasting is not anticipated.

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.*

5.7 Lateral Earth Pressure

Lateral earth pressure should be carefully considered for design of site retaining walls. Hydrostatic pressure and additional surcharge loading should also be considered. Walls that are free to deflect and retain native soils may be designed for an active equivalent fluid pressure of 40 pcf for level exterior backfill slopes. For walls with maximum 3H:1V exterior backfill slopes, an active equivalent fluid pressure of 50 pcf is recommended. For walls restrained from rotation, an at-rest equivalent fluid pressure of 60 pcf may be used for design. Passive equivalent fluid pressure may be estimated at 350 pcf. These are the best estimates of actual static earth pressure and do not include hydrostatic pressure, seismic forces, surcharge loading, or a factor of safety. If fill soil is retained, specific earth pressures should be developed by the geotechnical engineer after wall locations are identified.

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 5.4, *Foundations.*

An adequate perforated drain system with gravity discharge should also be installed at the base of site retaining walls. The drain system should consist of perforated drainpipe surrounded by drain rock and wrapped in geotextile filter fabric. Specifications for drainpipe design are presented in Section 5.9, *Drainage.*

Free-draining granular material with maximum particle size of three inches and less than five percent passing the No. 200 sieve should be used as backfill directly behind retaining walls to reduce hydrostatic pressure. The backfill material within two feet of retaining walls should be compacted to 90 percent of standard Proctor maximum dry density (ASTM D698). Increased or excessive compaction of this material can result in elevated lateral earth pressures. The remainder of the backfill should be compacted to at least 90 percent of modified Proctor (ASTM D1557) maximum dry density.

5.8 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 1.

Table 1. 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	0.86 g
1.0 sec Spectral Acceleration	0.15 g	0.32 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 *2003 IBC Tables 1615.1.2(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 are most likely represented by Site Class D as defined in *2003 IBC Table 1615.1.1*. This assessment is preliminary and based upon field exploration and research of existing published literature. A deeper soil boring exploration may provide more definitive site classification information.

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 or evaluation of liquefaction potential for the site is beyond the scope of this investigation. If site structures are designed in accordance with recommendations specified in the *2003 IBC*, the potential for peak ground accelerations in excess of the adjusted and amplified values should be understood.

5.9 Drainage

The presence of fine-textured site soils and mottled coloring indicates potential for reduced soil permeability and underscores 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 City of La Center regulations. Finished site grading should be conducted with positive drainage away from structures. There should be no depressions or shallow areas that may retain ponding water. Roof drains, low-point crawl space drains, and perimeter foundation drains are recommended for residential structures. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from foundations into the storm water system or approved discharge location.

Perimeter foundation drains should consist of 3-inch perforated PVC pipe surrounded by a minimum of 1 ft^3 of clean, washed drain rock per linear foot of pipe and wrapped with geotextile filter fabric. Open-graded drain rock with a maximum particle size of 3 inches and less than 2 percent passing the No. 200 sieve is recommended. Geotextile filter fabric should consist of 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 significantly 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.

5.10 Storm Water Facility Construction

If earthen berms are utilized for storm water facility construction, a base key should be installed with minimum depth and width of H/3 and W/2, respectively, where H is the total berm height and W is the total berm width. Figure 7 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 Sections 5.1 and 5.2. 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. A composite sample of structural berm fill should be submitted to the geotechnical engineer for approval prior to construction.

A licensed geotechnical engineer should review final grading and earthwork plans for the storm water facility prior to final design approval. The site geotechnical engineer should also observe, test, and document earthwork and construction activities associated with the storm water facility.

5.11 Slope Stability

Columbia West reviewed *Slope Stability, Clark County, Washington, Allen Fiksdal, Washington Division of Geology and Earth Resources, Open File Report 75-10, 1975* to assess site slope characteristics. The report identifies four levels of potential slope instability within Clark County: (1) stable areas – no slides or unstable slopes, (2) areas of potential instability because of underlying geologic conditions and physical characteristics associated with steepness, (3) areas of historical or still active landslides, and (4) older landslide debris. The site is mapped as (1) stable areas – no slides or unstable slopes.

5.12 Bituminous Asphalt and Portland Cement Concrete

Based upon preliminary correspondence with the client, the site is anticipated to include flexible asphalt concrete residential streets. Columbia West has reviewed the City of La Center's standards for public works construction and recommends adherence to identified pavement thickness sections for proposed roads.

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 5.13, *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 90 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 92 percent of maximum Rice density. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with Washington Department of Transportation specifications.

Portland cement concrete curbs and sidewalks should be installed in accordance with City of La Center 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.

5.13 Wet Weather Construction Methods and Techniques

Wet weather construction at sites with fine-textured soils often results in significant shear strength reduction and creation of soft areas that may rut or deflect. Installation of granular working blankets may be necessary to provide a firm support base and sustain construction equipment. Granular blankets should consist of bank run gravel, pit run quarry rock, 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.

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. Geotextile filter fabric is also recommended. Crushed aggregate base should be installed in a single lift with trucks end-dumping off 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 by a moderately heavy 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 12-cubic yard, double-axle 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.

5.14 Soil Shrink/Swell Potential

Columbia West's field exploration indicated a 2½-foot-thick layer of potentially expansive fat clay located within the northwest portion of the site. Based upon laboratory analysis, the fat clay contained approximately 91 percent by weight passing the No. 200 sieve. Atterberg Limits analysis indicated a liquid limit and plasticity index of 63 and 45, respectively. This may indicate increased potential for soil shrinking and swelling. The *Soil Survey of Clark County, Washington (United States Department of Agriculture, Soil Conservation Service (USDA SCS), November 1972)* also indicates potential for shrinking and swelling.

To minimize potential risks, mitigative measures identified previously in Section 5.1.1, *Over-Excavation of Expansive Soils*, should be followed. It is also recommended that an experienced geotechnical engineer or designated representative closely monitor fill placement and compaction activities in areas with fat clays and potentially expansive soils. The potential for soil swelling can be minimized by thoroughly mixing expansive clays with less plastic soils and properly controlling moisture content during fill placement.

5.15 Soil Erosion Potential

The *Soil Survey of Clark County, Washington* indicates potential erosion hazards for fine-textured site soils. For flat to shallow-gradient portions of the property the erosion hazard is likely to be low. The potential for erosion generally increases in moderate-gradient or sloped areas. Soils are 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 and disturbance to existing native vegetation and surrounding organic soil should be minimized during construction activities.

5.16 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 the City of La Center and the site geotechnical engineer. It is anticipated that onsite fine-textured native soils will not be suitable for use as trench backfill. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). The remaining backfill should be compacted to at least 95 percent of maximum dry density as determined by the standard Proctor moisture-density test (ASTM D698). Clean, free-draining, fine bedding sand is recommended for use in the pipe zone. With exception of the pipe zone, backfill should be placed in loose lifts not exceeding 12 inches in thickness.

Compaction of utility trench backfill material should be verified by nuclear gauge field compaction testing performed in accordance with ASTM 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.

5.17 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. 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 relevant parties.

Sincerely,

COLUMBIA WEST ENGINEERING, INC.

Lance V. Lehto, PE, MS
President

LANCE V. LEHTO
STATE OF WASHINGTON
36301
REGISTERED
PROFESSIONAL ENGINEER
10-4-05
EXPIRES: 9-10-06

REFERENCES

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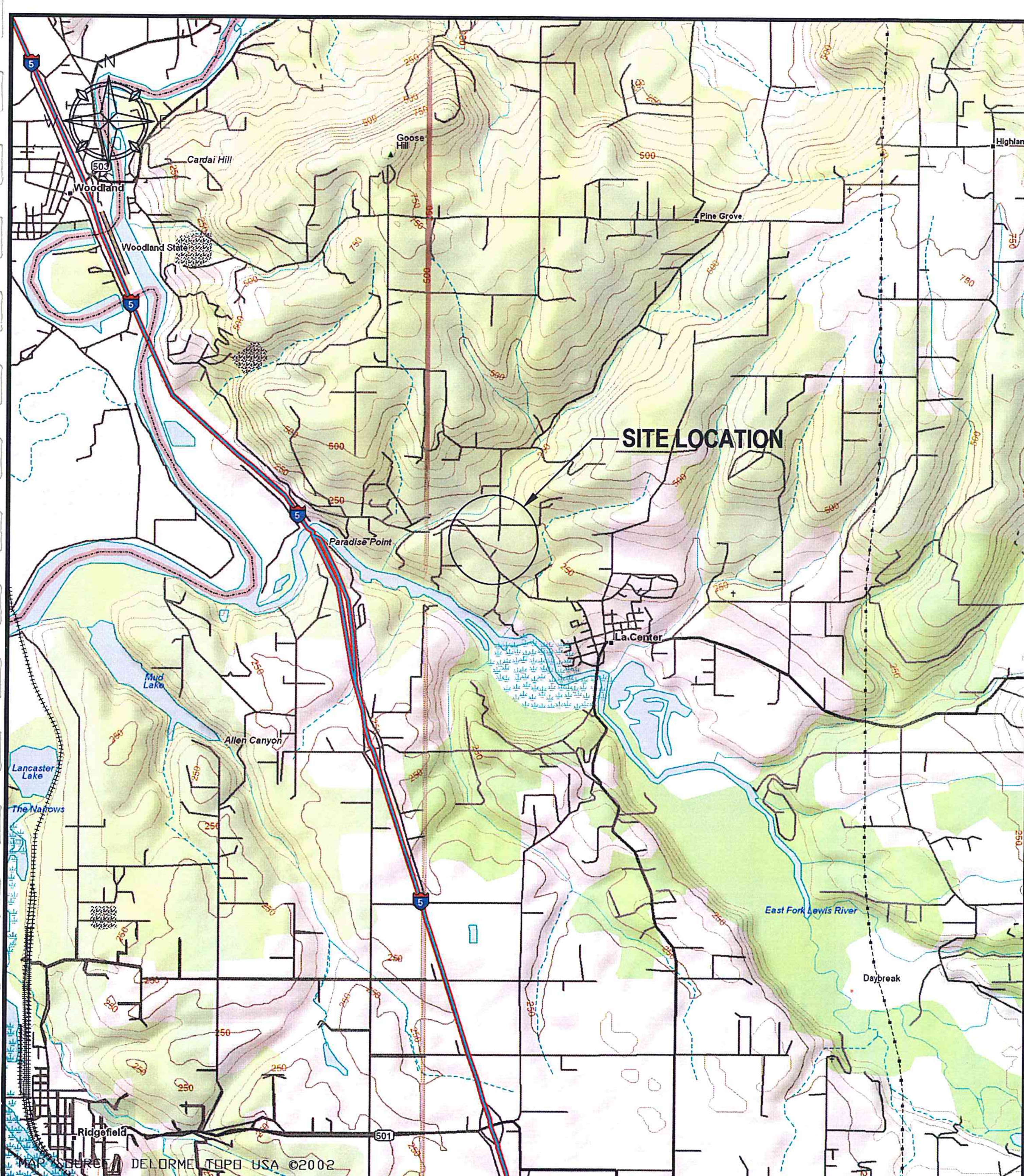
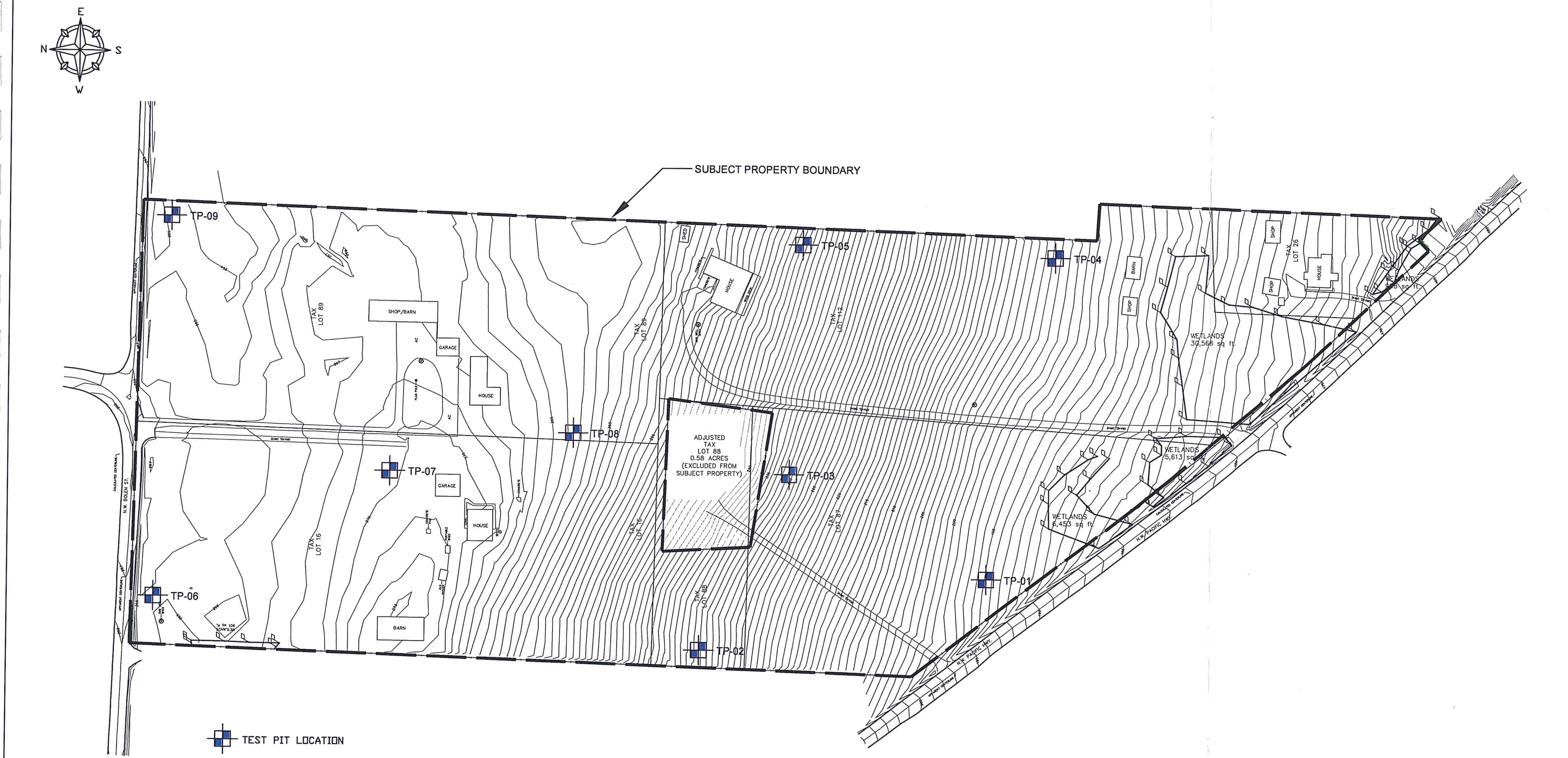
FIGURES



Columbia West Engineering, Inc. 8019 NE 13th Avenue Vancouver, Washington 360-993-2879	Design:	Drawn: LVL	SITE LOCATION MAP	FIGURE 1
	Checked: DEL	Date: 08/10/05		
	Client: ALTIUS	Rev / By / Date	HIGHLAND TERRACE LA CENTER, WASHINGTON	
	Job No.: 05135			
	CAD File: FIGURE 1			
	Scale: 1:50,000			



NOTES:
1. DRAWING IS NOT TO SCALE.
2. SITE MAP PROVIDED BY MINISTER-GLAESER SURVEYING, INC.
3. TEST PITS LOCATIONS SURVEYED USING HAND-HELD GPS EQUIPMENT.
4. ALL TEST PIT EXCAVATIONS WERE BACKFILLED LOOSELY WITH ONSITE SOIL ON SEPTEMBER 13, 2005.

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Checked: LVL	Date: 09-19-05			
Client: ALTIUS	Rev	By	Date	
Job No: 05135				
CAD File: FIGURE 2				
Scale: NONE				

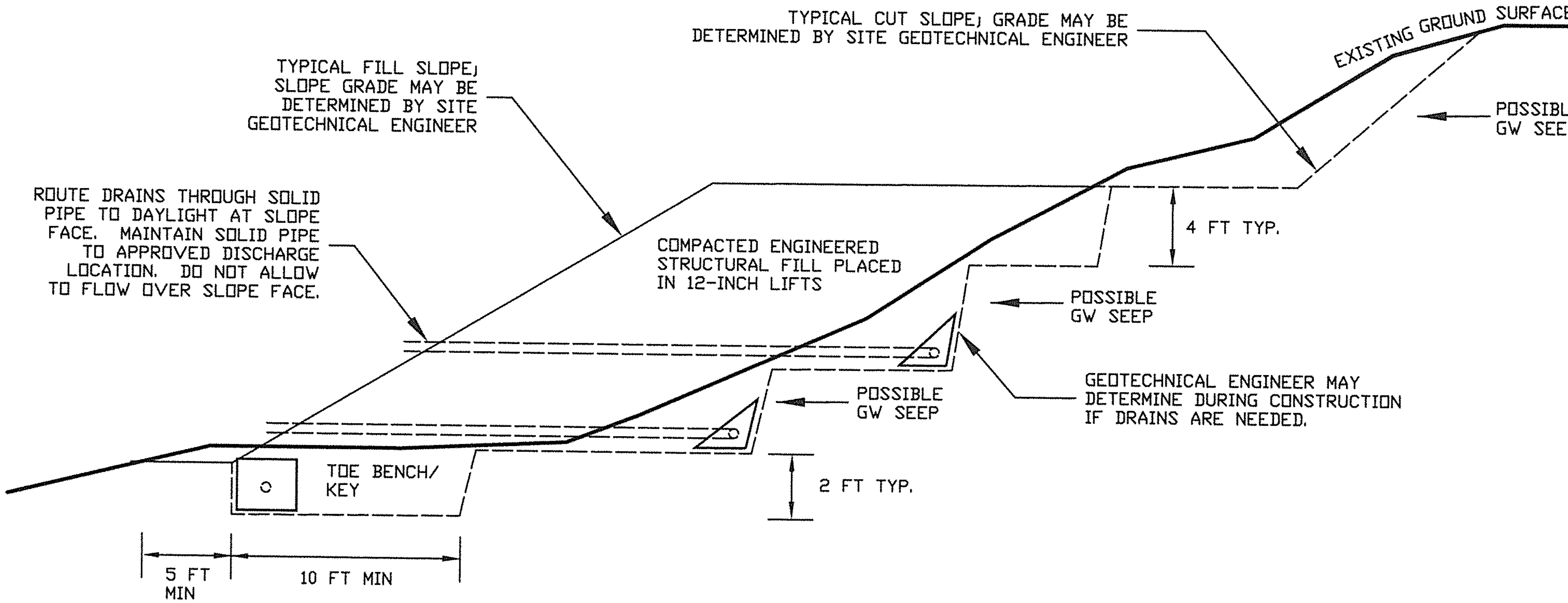
TEST PIT LOCATION MAP

HIGHLAND TERRACE

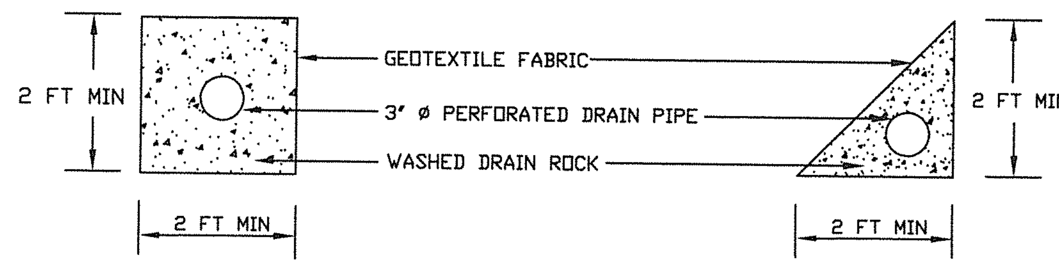
LA CENTER, WASHINGTON

FIGURE 2

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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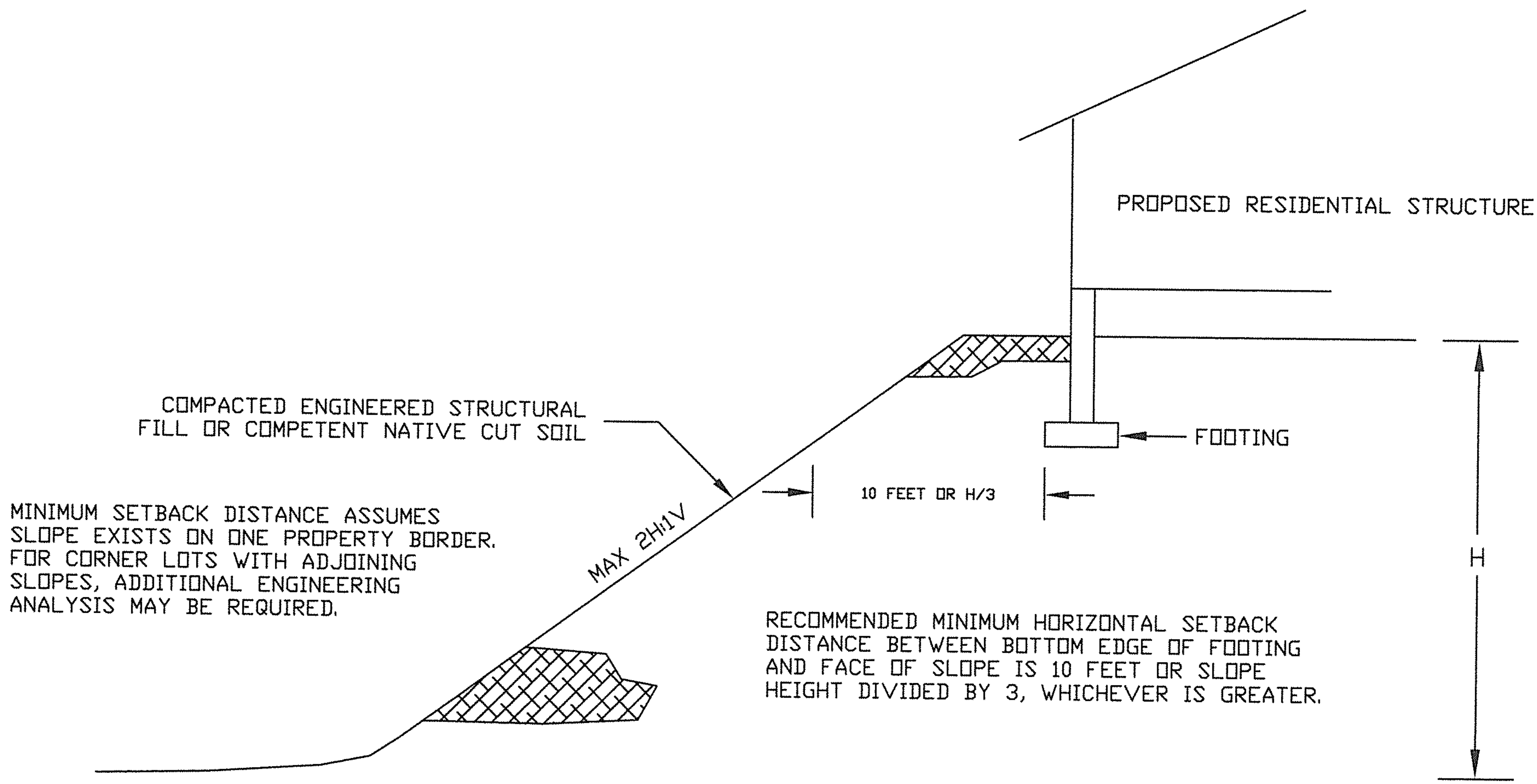
Job No: 05135				
CAD File: FIGURE 3				
Scale: NONE				

TYPICAL CUT AND FILL SLOPE SECTION

HIGHLAND TERRACE
LA CENTER, WASHINGTON

FIGURE 3

MINIMUM FOUNDATION SLOPE SETBACK DISTANCE



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

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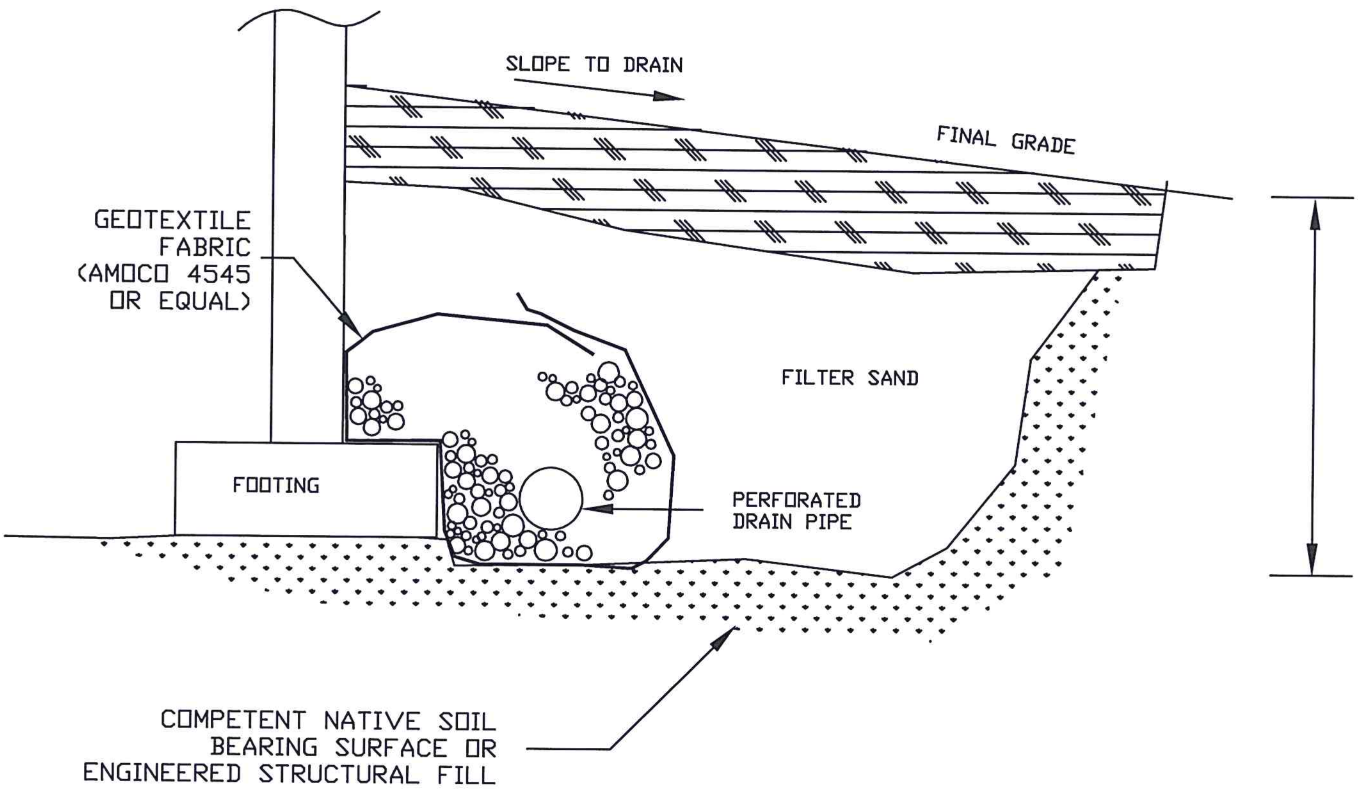
Design:	Drawn: JGH			MINIMUM FOUNDATION SLOPE SETBACK
Checked: LVL	Date: 09-29-05			
Client: ALTIUS	Rev	By	Date	HIGHLAND TERRACE LA CENTER, WASHINGTON
Job No: 05135				
CAD File: FIGURE 4				
Scale: NONE				

FIGURE 4

TYPICAL PERIMETER FOOTING DRAIN DETAIL

NOT TO SCALE



LEGEND

(drain rock symbol)	OPEN-GRADED DRAIN ROCK W/ MAX. 3-INCH PART. SIZE
(topsoil symbol)	TOPSOIL MATERIAL
(stippled symbol)	COMPETENT NATIVE SOIL OR COMPACTED ENGINEERED STRUCTURAL FILL

NOTES:

1. DRAIN PIPE SHOULD CONSIST OF PERFORATED OR SLOTTED 3-INCH RIGID PVC PIPE.
2. FINAL GRADE SHOULD PROVIDE POSITIVE DRAINAGE AWAY FROM SITE STRUCTURES.
3. DRAIN PIPE SHALL BE INSTALLED AT MINIMUM 2% SLOPE AND SHALL DISCHARGE BY GRAVITY FLOW TO AN APPROVED DISCHARGE LOCATION.

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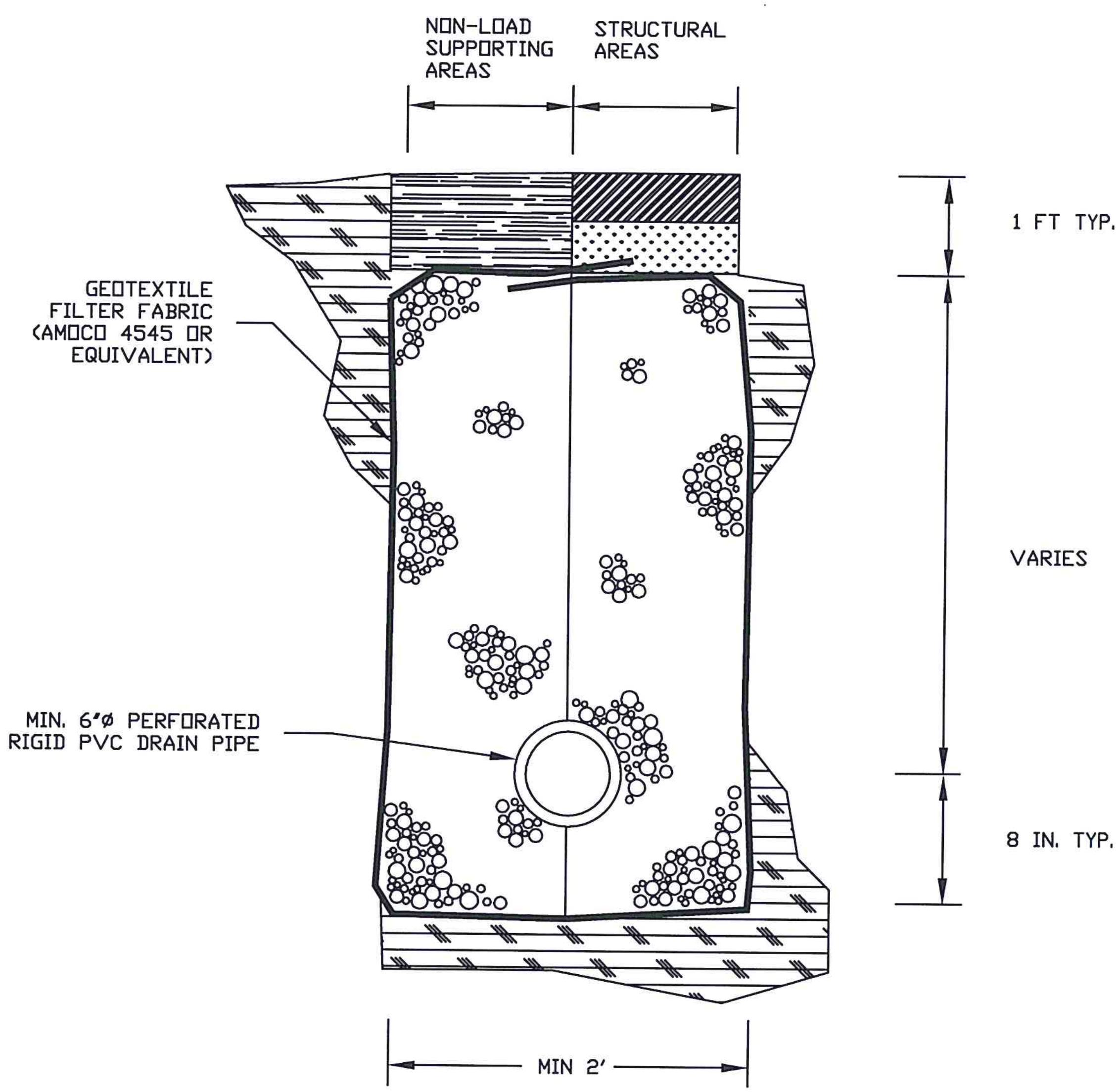
Design:	Drawn: LVL	FOOTING DRAIN DETAIL	FIGURE 5
Checked: DEL	Date: 08-10-05		
Client: ALTIUS	Rev / By / Date	HIGHLAND TERRACE	
Job No: 05135		LA CENTER, WASHINGTON	
CAD File: FIGURE 5			
Scale: NONE			

PERFORATED DRAIN PIPE TRENCH DETAIL



LEGEND

- OPEN-GRADED DRAIN ROCK WITH MAXIMUM 3-INCH PARTICLE SIZE
- IN-SITU NATIVE SOIL OR COMPACTED ENGINEERED FILL
- FLEXIBLE ASPHALT PAVEMENT OR CONCRETE PAVEMENT
- CRUSHED AGGREGATE SUB-BASE OR BASE MATERIAL
- COMPACTED ENGINEERED FILL

NOTE: LOCATION, INVERT ELEVATION, DEPTH OF TRENCH, AND EXTENT OF PERFORATED PIPE REQUIRED MAY BE RECOMMENDED ON A CASE-BY-CASE BASIS BY THE GEOTECHNICAL ENGINEER DURING CONSTRUCTION AND MAY BE MODIFIED ACCORDINGLY.

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Design:	Drawn: LVL	PERFORATED DRAIN PIPE DETAIL	FIGURE 6
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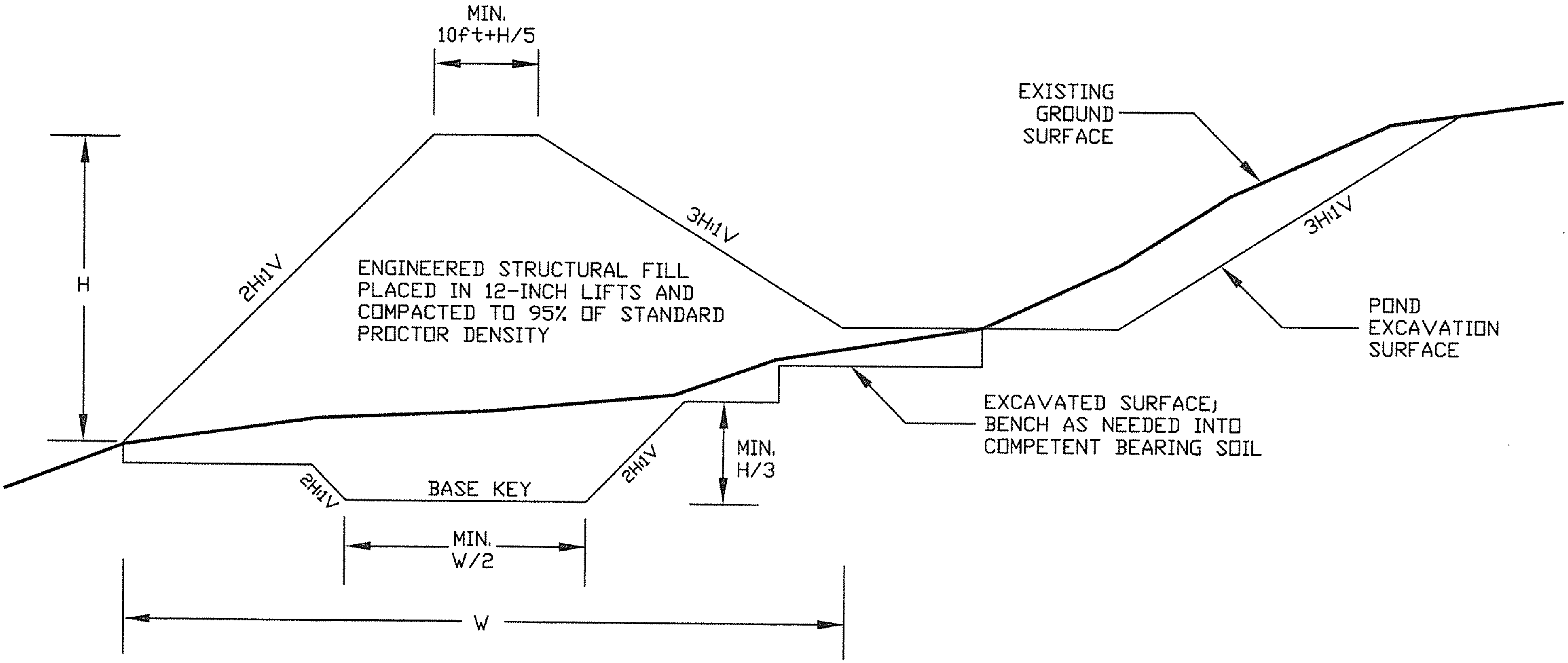
Client: ALTIUS	Rev / By / Date	HIGHLAND TERRACE	
Job No: 05135		LA CENTER, WASHINGTON	
CAD File: FIGURE 6			
Scale: NONE			

TYPICAL STORM WATER FACILITY BERM 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.

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Job No: 05135			
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Scale: NONE			

TYPICAL BERM SECTION

HIGHLAND TERRACE
LA CENTER, WASHINGTON

FIGURE 7

APPENDIX A
ANALYTICAL LABORATORY TEST REPORTS

Columbia West Engineering, Inc.
Professional Geotechnical Engineering and Environmental Consulting Services
P.O. Box 872216, Vancouver, Washington 98687
Phone: 360-993-2879, Fax: 360-574-4811
www.columbiawestengineering.com

SOIL PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Highland Terrace Property La Center, Washington	Altius Corporation Mr. Chris Sundstrom 13217 NW 30th Court Vancouver, Washington 98685	05135	S05-397
		REPORT DATE: 09/27/05	FIELD ID: S1.1
		DATE SAMPLED: 09/13/05	SAMPLED BY: JGH

MATERIAL DATA

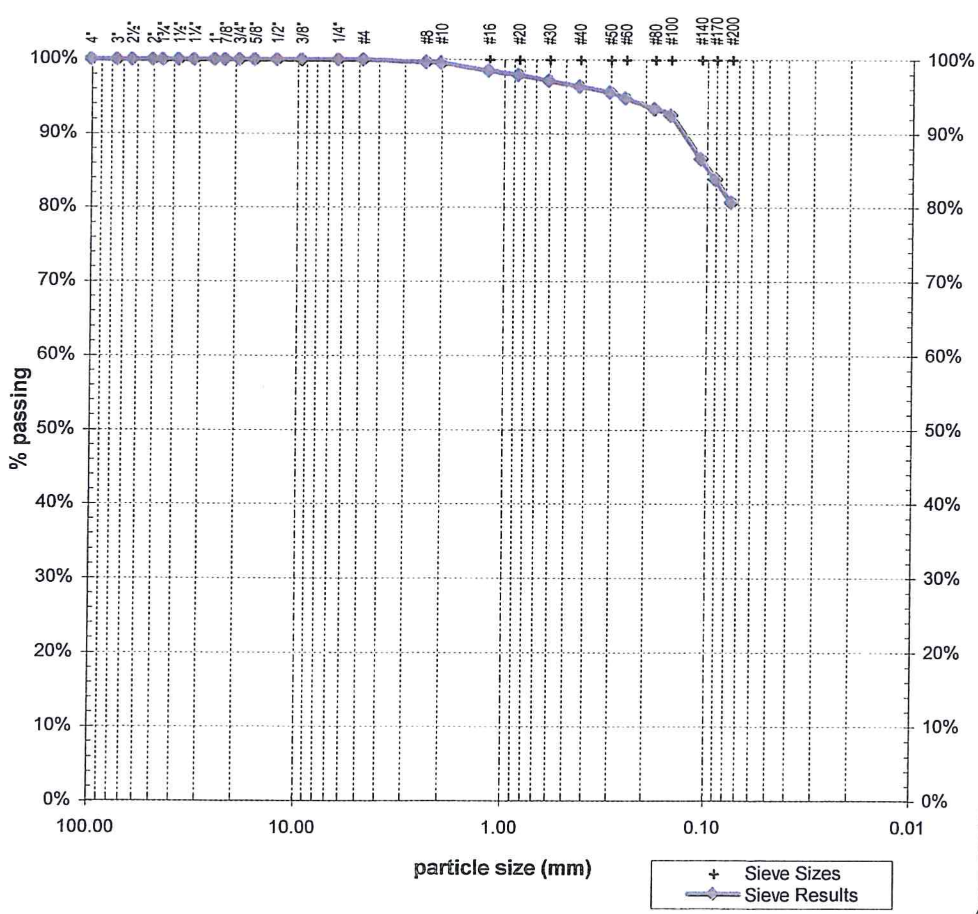
MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY with Sand	Test Pit 01, depth = 2 feet	CL, Lean Clay with Sand
SPECIFICATIONS: none		AASHTO SOIL TYPE: A-4(5)

LABORATORY TEST DATA

ADDITIONAL DATA

natural moisture content =	14.5%	coefficient of curvature, C_C =	n/a
liquid limit =	25.6%	coefficient of uniformity, C_U =	n/a
plastic limit =	17.6%	effective size, $D_{(10)}$ =	n/a
plasticity index =	8.0%	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

GRAIN SIZE DISTRIBUTION



SIEVE DATA

% gravel = 0.0%
% sand = 19.2%
% silt and clay = 80.8%

	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		99.6%		
	#10	2.00	99.5%			
	#16	1.18		98.6%		
	#20	0.850	98.0%			
	#30	0.600		97.2%		
	#40	0.425	96.3%			
	#50	0.300	95.6%			
	#60	0.250		94.8%		
	#80	0.180		93.3%		
	#100	0.150	92.5%			
	#140	0.106		86.6%		
	#170	0.090		83.9%		
	#200	0.075	80.8%			

DATE TESTED	TESTED BY
09/23/05	BKJ

COLUMBIA WEST ENGINEERING, INC. authorized signature

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Phone. 360-993-2879, Fax 360-574-4811
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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Highland Terrace Property La Center, Washington	Altius Corporation Mr. Chris Sundstrom 13217 NW 30th Court Vancouver, Washington 98685	05135	S05-397
		REPORT DATE: 09/27/05	FIELD ID: S1.1
		DATE SAMPLED: 09/13/05	SAMPLED BY: JGH

MATERIAL DATA

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

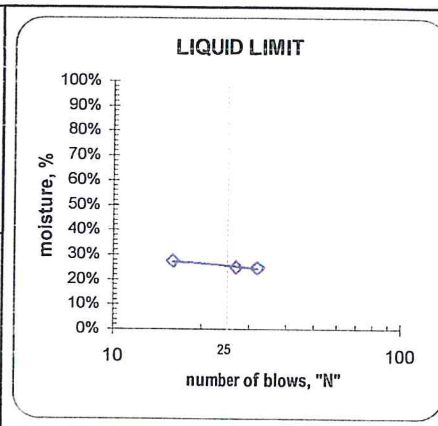
LABORATORY TEST DATA

ATTERBERG LIMITS

liquid limit = 25.6%
plastic limit = 17.6%
plasticity index = 8.0%

LIQUID LIMIT DETERMINATION

	①	②	③	④
wet soil + pan weight, g =	15.80	15.48	12.79	
dry soil + pan weight, g =	12.93	12.64	10.31	
pan weight, g =	1.23	1.22	1.23	
N (blows) =	32	27	16	
moisture, % =	24.5 %	24.9 %	27.3 %	

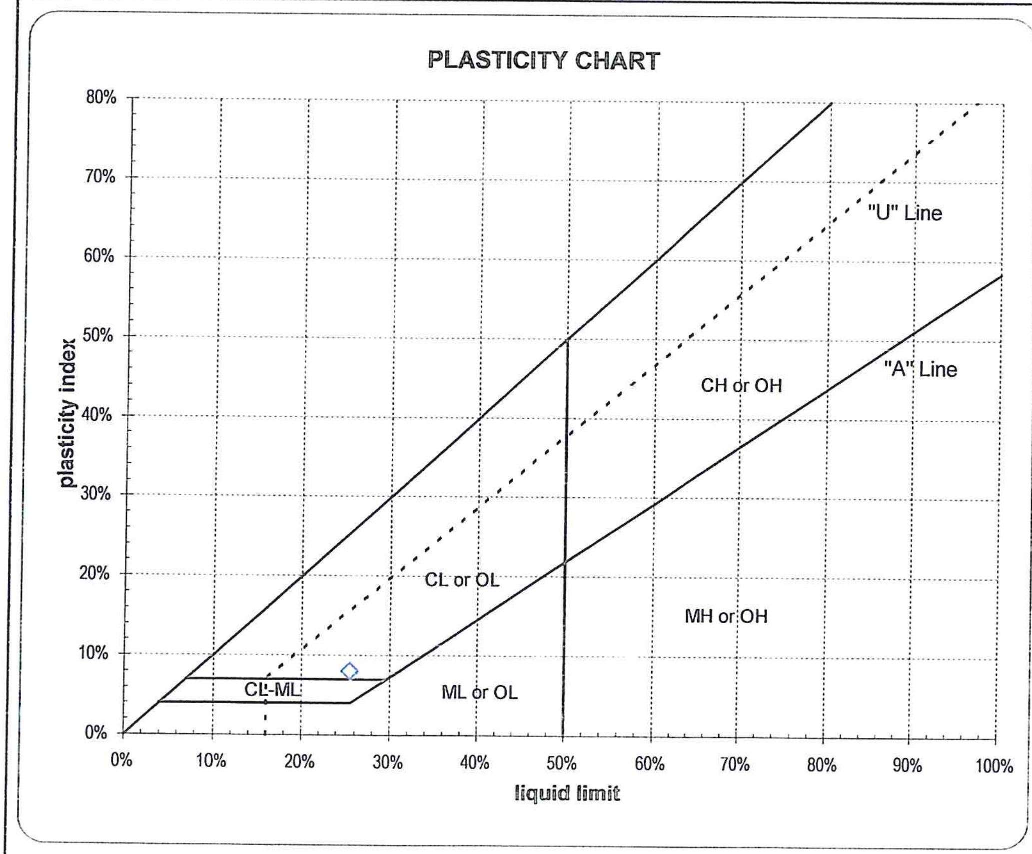


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	①	②	③	④
wet soil + pan weight, g =	10.40	9.19		
dry soil + pan weight, g =	9.02	8.00		
pan weight, g =	1.21	1.22		
moisture, % =	17.7 %	17.6 %		



ADDITIONAL DATA

% gravel = 0.0%
% sand = 19.2%
% silt and clay = 80.8%
% silt = n/a
% clay = n/a
moisture content = 14.5%

DATE TESTED	TESTED BY
09/26/05	JJC

COLUMBIA WEST ENGINEERING, INC. authorized signature

05135 S05-397 lean clay with sand, 9/27/2005

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

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www.columbiawestengineering.com

SOIL PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Highland Terrace Property La Center, Washington	Altius Corporation Mr. Chris Sundstrom 13217 NW 30th Court Vancouver, Washington 98685	05135	S05-398
		REPORT DATE: 09/27/05	FIELD ID: S1.2
		DATE SAMPLED: 09/13/05	SAMPLED BY: JGH

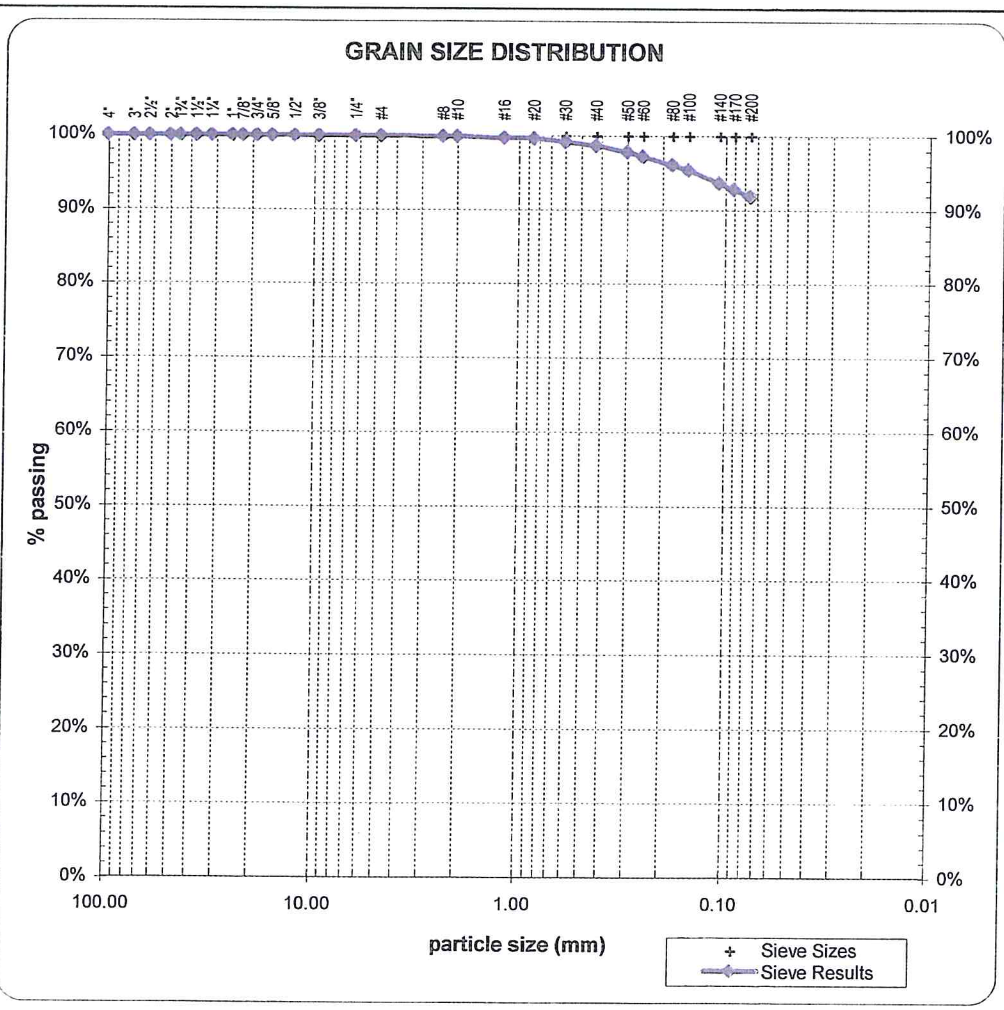
MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Test Pit 01, depth = 10 feet	CL, Lean Clay
SPECIFICATIONS: none		AASHTO SOIL TYPE: A-6(17)

LABORATORY TEST DATA

ADDITIONAL DATA

natural moisture content =	29.5%	coefficient of curvature, C_C =	n/a
liquid limit =	36.7%	coefficient of uniformity, C_U =	n/a
plastic limit =	18.6%	effective size, $D_{(10)}$ =	n/a
plasticity index =	18.1%	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a



SIEVE DATA

% gravel = 0.0%
% sand = 8.1%
% silt and clay = 91.9%

	SIEVE SIZE US	mm	PERCENT PASSING SIEVE act.	interp.	SPECS max	min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		100.0%		
	1.75"	45.0		100.0%		
	1.50"	37.5		100.0%		
	1.25"	31.5		100.0%		
	1.00"	25.0		100.0%		
	7/8"	22.4		100.0%		
	3/4"	19.0		100.0%		
	5/8"	16.0		100.0%		
	1/2"	12.5		100.0%		
	3/8"	9.50		100.0%		
	1/4"	6.30		100.0%		
	#4	4.75	100.0%			
SAND	#8	2.36		99.9%		
	#10	2.00	99.9%			
	#16	1.18		99.7%		
	#20	0.850	99.6%			
	#30	0.600		99.2%		
	#40	0.425	98.7%			
	#50	0.300	97.9%			
	#60	0.250		97.2%		
	#80	0.180		96.1%		
	#100	0.150	95.4%			
	#140	0.106		93.7%		
	#170	0.090		92.8%		
	#200	0.075	91.9%			

DATE TESTED	TESTED BY
09/23/05	BKJ

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Highland Terrace Property La Center, Washington	Altius Corporation Mr. Chris Sundstrom 13217 NW 30th Court Vancouver, Washington 98685	05135	S05-398
		REPORT DATE: 09/27/05	FIELD ID: S1.2
		DATE SAMPLED: 09/13/05	SAMPLED BY: JGH

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Lean CLAY	Test Pit 01, depth = 10 feet	CL, Lean Clay

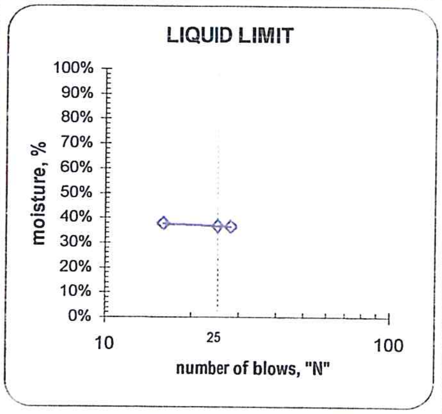
LABORATORY TEST DATA

ATTERBERG LIMITS

liquid limit = 36.7%
plastic limit = 18.6%
plasticity index = 18.1%

LIQUID LIMIT DETERMINATION

	①	②	③	④
wet soil + pan weight, g =	15.76	18.08	13.37	
dry soil + pan weight, g =	11.78	13.54	10.13	
pan weight, g =	1.21	1.21	1.22	
N (blows) =	16	25	28	
moisture, % =	37.7 %	36.8 %	36.4 %	

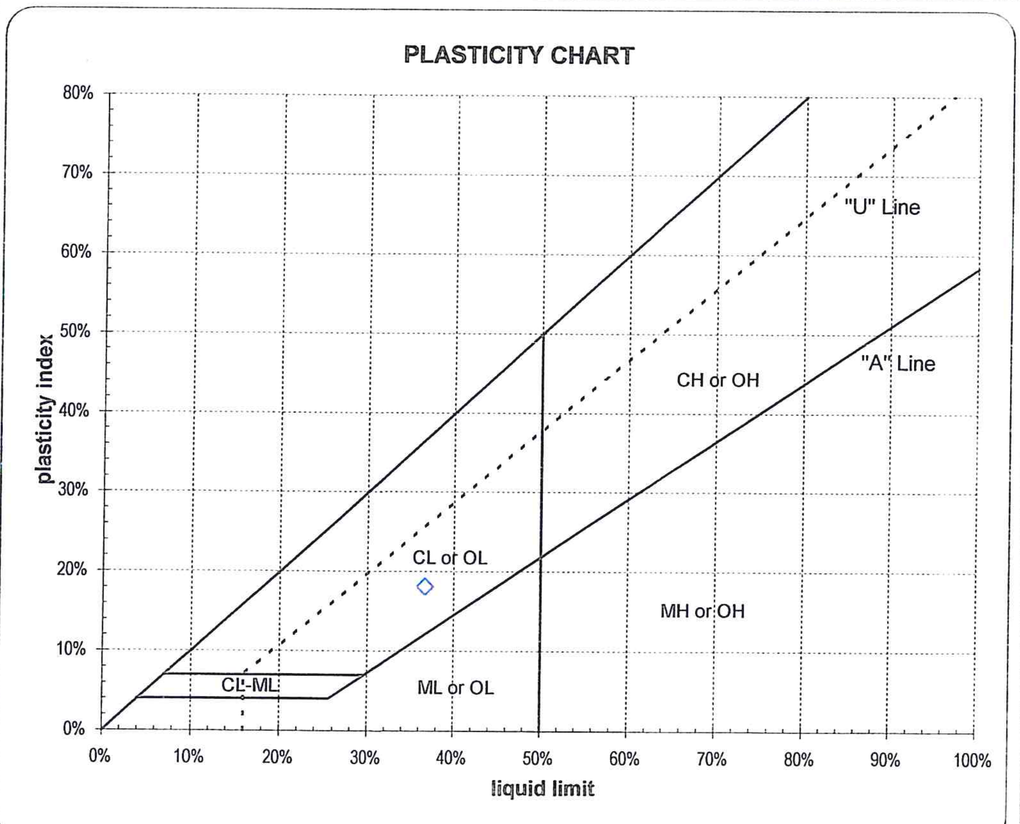


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	①	②	③	④
wet soil + pan weight, g =	5.11	5.52		
dry soil + pan weight, g =	4.49	4.85		
pan weight, g =	1.21	1.20		
moisture, % =	18.9 %	18.4 %		



ADDITIONAL DATA

% gravel = 0.0%
% sand = 8.1%
% silt and clay = 91.9%
% silt = n/a
% clay = n/a
moisture content = 29.5%

DATE TESTED	TESTED BY
09/26/05	EJC

05135 S05-398 lean clay, 9/27/2005

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SOIL PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Highland Terrace Property La Center, Washington	Altius Corporation Mr. Chris Sundstrom 13217 NW 30th Court Vancouver, Washington 98685	05135	S05-399
		REPORT DATE: 09/27/05	FIELD ID: S2.2
		DATE SAMPLED: 09/13/05	SAMPLED BY: JGH

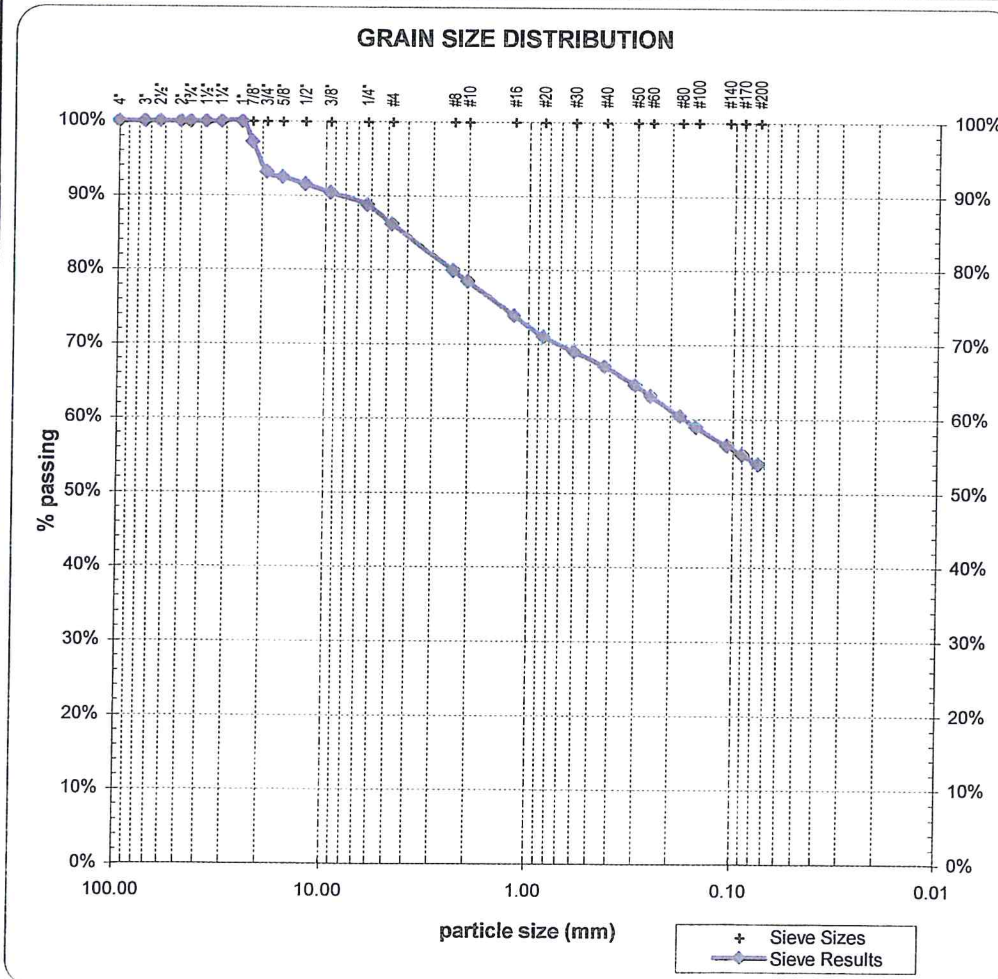
MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Lean CLAY	Test Pit 02, depth = 8 feet	CL, Sandy Lean Clay
SPECIFICATIONS: none		AASHTO SOIL TYPE: A-6(5)

LABORATORY TEST DATA

ADDITIONAL DATA

natural moisture content =	22.0%	coefficient of curvature, C_C =	n/a
liquid limit =	32.6%	coefficient of uniformity, C_U =	n/a
plastic limit =	17.9%	effective size, $D_{(10)}$ =	n/a
plasticity index =	14.8%	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	0.170 mm



SIEVE DATA

% gravel = 13.8%
% sand = 32.2%
% silt and clay = 53.9%

	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		97.3%		
	3/4"	19.0	93.2%			
	5/8"	16.0		92.5%		
	1/2"	12.5		91.5%		
	3/8"	9.50		90.4%		
	1/4"	6.30	88.7%			
	#4	4.75	86.2%			
SAND	#8	2.36		79.9%		
	#10	2.00	78.4%			
	#16	1.18		73.9%		
	#20	0.850	71.0%			
	#30	0.600		69.1%		
	#40	0.425	67.1%			
	#50	0.300	64.6%			
	#60	0.250		63.1%		
	#80	0.180		60.4%		
	#100	0.150	59.0%			
	#140	0.106		56.4%		
	#170	0.090		55.2%		
	#200	0.075	53.9%			

DATE TESTED	TESTED BY
09/23/05	BKJ

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05135 S05-399 sandy lean clay, 9/27/2005

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Highland Terrace Property La Center, Washington	Altius Corporation Mr. Chris Sundstrom 13217 NW 30th Court Vancouver, Washington 98685	05135	S05-399
		REPORT DATE: 09/27/05	FIELD ID: S2.2
		DATE SAMPLED: 09/13/05	SAMPLED BY: JGH

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Sandy Lean CLAY	Test Pit 02, depth = 8 feet	CL, Sandy Lean Clay

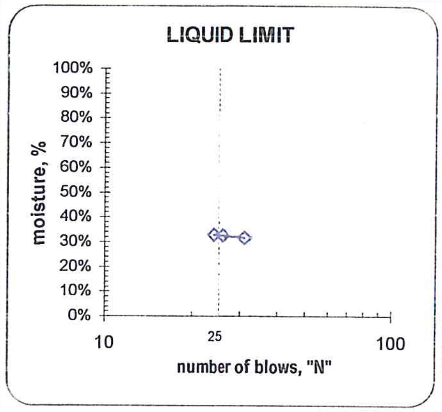
LABORATORY TEST DATA

ATTERBERG LIMITS

liquid limit = 32.6%
plastic limit = 17.9%
plasticity index = 14.8%

LIQUID LIMIT DETERMINATION

	①	②	③	④
wet soil + pan weight, g =	10.58	11.59	10.99	
dry soil + pan weight, g =	8.33	9.05	8.57	
pan weight, g =	1.22	1.22	1.20	
N (blows) =	31	26	24	
moisture, % =	31.7 %	32.4 %	32.8 %	

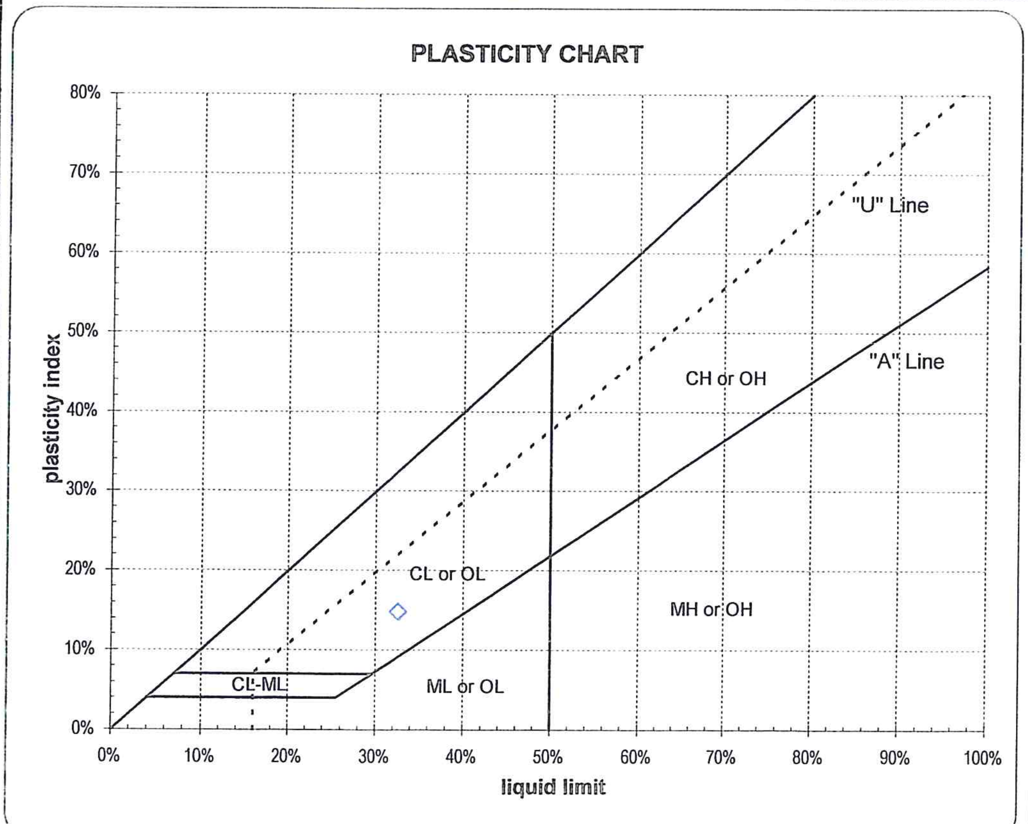


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	①	②	③	④
wet soil + pan weight, g =	6.59	7.58		
dry soil + pan weight, g =	5.77	6.62		
pan weight, g =	1.20	1.22		
moisture, % =	17.9 %	17.8 %		



ADDITIONAL DATA

% gravel = 13.8%
% sand = 32.2%
% silt and clay = 53.9%
% silt = n/a
% clay = n/a
moisture content = 22.0%

DATE TESTED	TESTED BY
09/26/05	JJC

05135 S05-399 sandy lean clay, 9/27/2005

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SOIL PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Highland Terrace Property La Center, Washington	Altius Corporation Mr. Chris Sundstrom 13217 NW 30th Court Vancouver, Washington 98685	05135	S05-400
		REPORT DATE: 09/27/05	FIELD ID: S6.1
		DATE SAMPLED: 09/13/05	SAMPLED BY: JGH

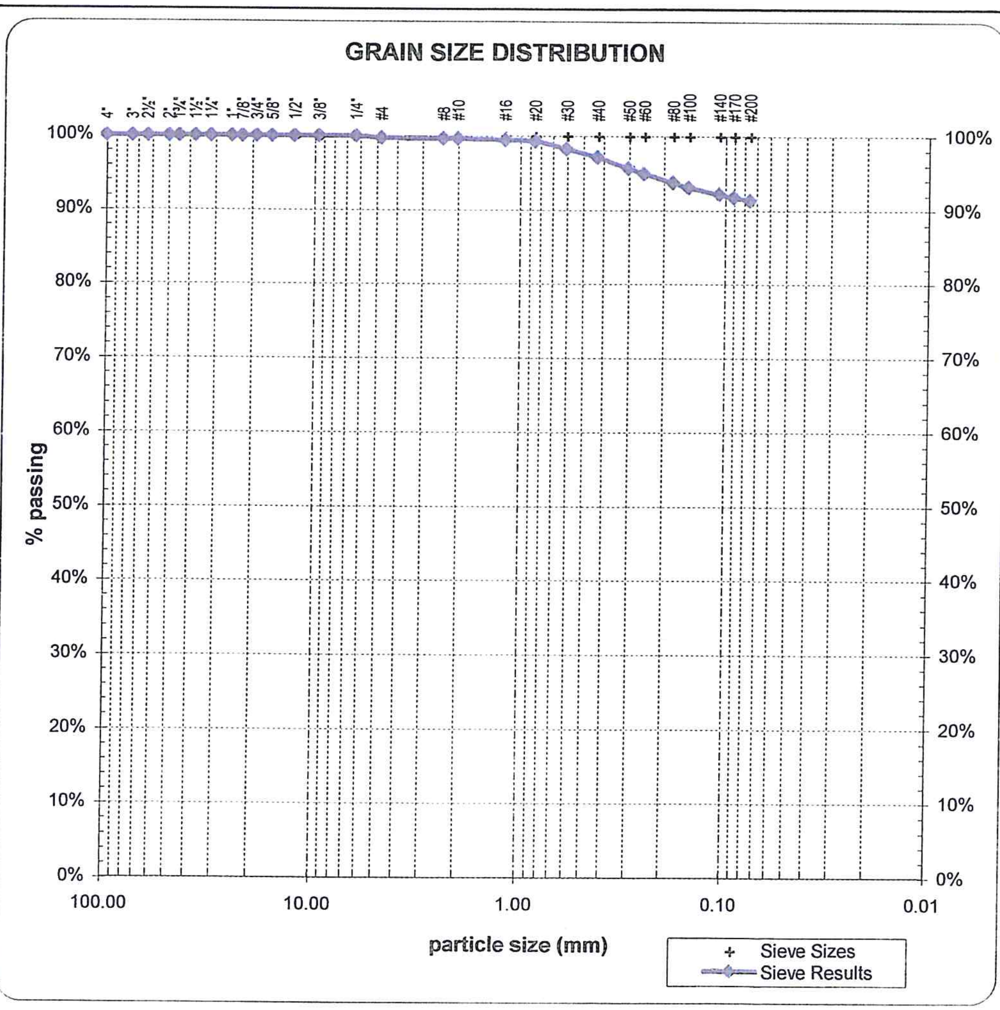
MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY	Test Pit 06, depth = 3 feet	CH, Fat Clay
SPECIFICATIONS: none		AASHTO SOIL TYPE: A-7-6(45)

LABORATORY TEST DATA

ADDITIONAL DATA

natural moisture content =	28.7%	coefficient of curvature, C_C =	n/a
liquid limit =	63.4%	coefficient of uniformity, C_U =	n/a
plastic limit =	18.2%	effective size, $D_{(10)}$ =	n/a
plasticity index =	45.2%	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a



SIEVE DATA

% gravel = 0.2%
% sand = 8.4%
% silt and clay = 91.3%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100.0%		
	4.00"	100.0		100.0%		
	3.00"	75.0		100.0%		
	2.50"	63.0		100.0%		
	2.00"	50.0		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	99.8%			
SAND	#8	2.36		99.7%		
	#10	2.00	99.7%			
	#16	1.18		99.5%		
	#20	0.850	99.3%			
	#30	0.600		98.3%		
	#40	0.425	97.2%			
	#50	0.300	95.7%			
	#60	0.250		95.0%		
	#80	0.180		93.8%		
	#100	0.150	93.1%			
	#140	0.106		92.2%		
	#170	0.090		91.8%		
	#200	0.075	91.3%			

DATE TESTED	TESTED BY
09/23/05	BKJ

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05135 S05-400 fat clay, 9/27/2005

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Highland Terrace Property La Center, Washington	Altius Corporation Mr. Chris Sundstrom 13217 NW 30th Court Vancouver, Washington 98685	05135	S05-400
		REPORT DATE: 09/27/05	FIELD ID: S6.1
		DATE SAMPLED: 09/13/05	SAMPLED BY: JGH

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY	Test Pit 06, depth = 3 feet	CH, Fat Clay

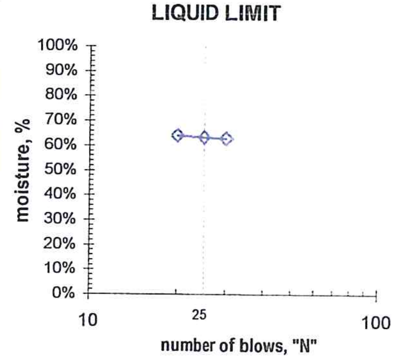
LABORATORY TEST DATA

ATTERBERG LIMITS

liquid limit = 63.4%
plastic limit = 18.2%
plasticity index = 45.2%

LIQUID LIMIT DETERMINATION

	①	②	③	④
wet soil + pan weight, g =	9.15	9.02	9.10	
dry soil + pan weight, g =	6.09	6.00	6.02	
pan weight, g =	1.22	1.23	1.21	
N (blows) =	30	25	20	
moisture, % =	62.8 %	63.3 %	64.0 %	

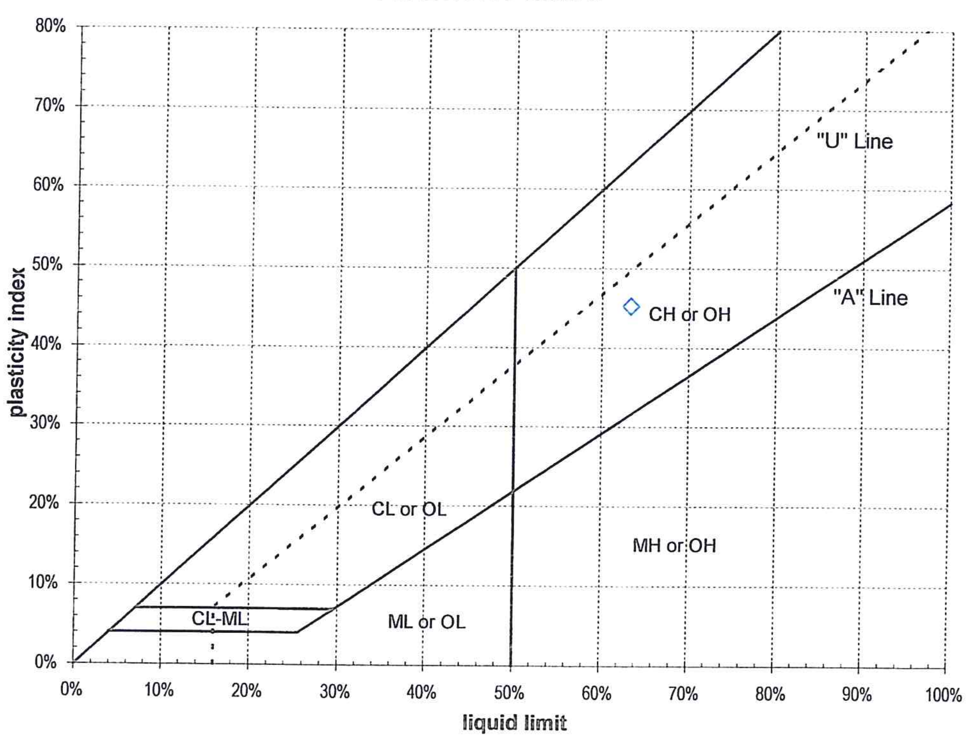


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	①	②	③	④
wet soil + pan weight, g =	3.62	5.58		
dry soil + pan weight, g =	3.25	4.91		
pan weight, g =	1.22	1.21		
moisture, % =	18.2 %	18.1 %		



ADDITIONAL DATA

% gravel = 0.2%
% sand = 8.4%
% silt and clay = 91.3%
% silt = n/a
% clay = n/a
moisture content = 28.7%

DATE TESTED	TESTED BY
09/26/05	JJC

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05135 S05-400 fat clay, 9/27/2005

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SOIL PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Highland Terrace Property La Center, Washington	Altius Corporation Mr. Chris Sundstrom 13217 NW 30th Court Vancouver, Washington 98685	05135	S05-401
		REPORT DATE: 09/27/05	FIELD ID: S9.1
		DATE SAMPLED: 09/13/05	SAMPLED BY: JGH

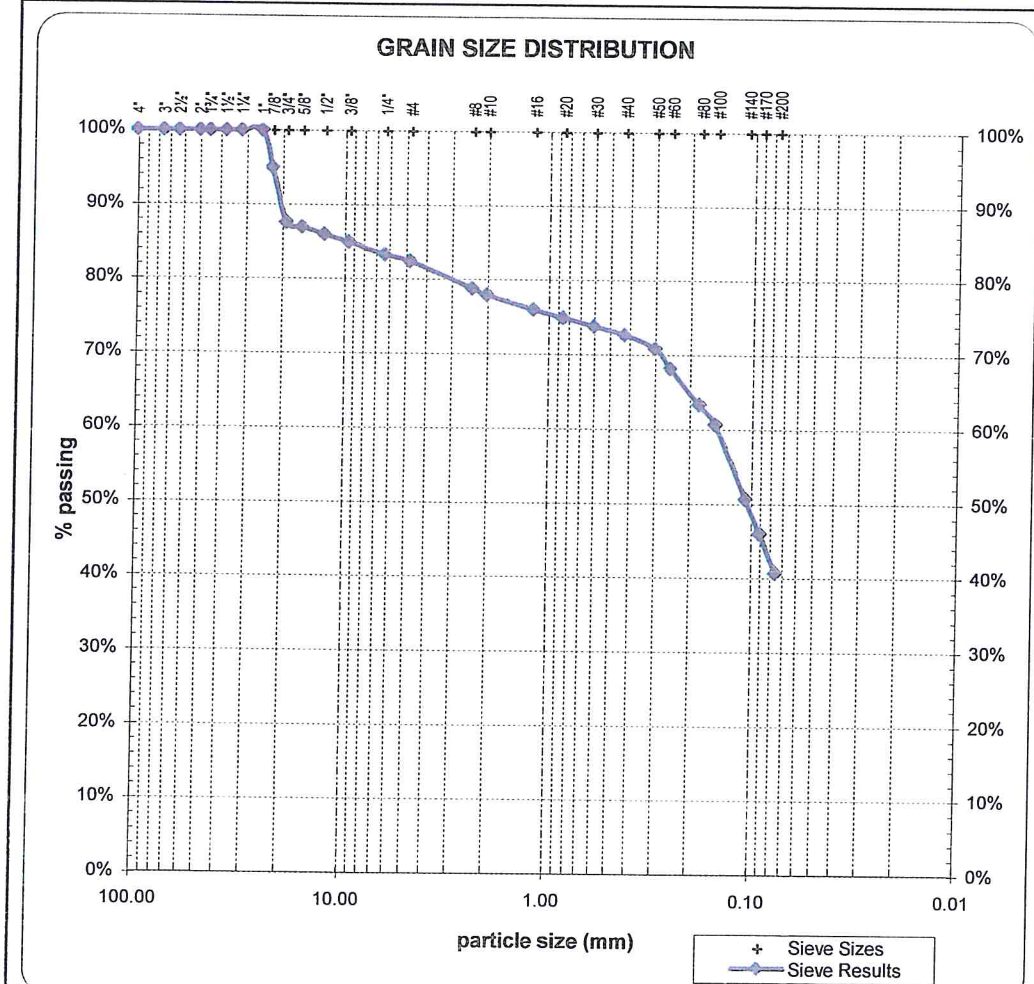
MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty SAND with Gravel	Test Pit 09, depth = 9 feet	SM, Silty Sand with Gravel
SPECIFICATIONS: none		AASHTO SOIL TYPE: A-4(0)

LABORATORY TEST DATA

ADDITIONAL DATA

natural moisture content =	26.2%	coefficient of curvature, C_C =	n/a
liquid limit =	0.0%	coefficient of uniformity, C_U =	n/a
plastic limit =	0.0%	effective size, $D_{(10)}$ =	n/a
plasticity index =	0.0%	$D_{(30)}$ =	n/a
fineness modulus =	1.85	$D_{(60)}$ =	0.147 mm



SIEVE DATA

% gravel = 17.5%
% sand = 41.8%
% silt and clay = 40.7%

	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		95.0%		
	3/4"	19.0	87.6%			
	5/8"	16.0		86.9%		
	1/2"	12.5		86.0%		
	3/8"	9.50		84.9%		
	1/4"	6.30	83.4%			
	#4	4.75	82.5%			
SAND	#8	2.36		78.8%		
	#10	2.00	78.0%			
	#16	1.18		76.1%		
	#20	0.850	74.9%			
	#30	0.600		73.8%		
	#40	0.425	72.7%			
	#50	0.300	70.9%			
	#60	0.250		68.2%		
	#80	0.180		63.3%		
	#100	0.150	60.6%			
	#140	0.106		50.7%		
	#170	0.090		46.0%		
	#200	0.075	40.7%			

DATE TESTED	TESTED BY
09/23/05	BKJ

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05135 S05-401 sandy silt with gravel, 9/27/2005

APPENDIX B
TEST PIT EXCAVATION LOGS

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TEST PIT LOG

PROJECT NAME	PROJECT NO.	LOGGED BY	TEST PIT NO.
Highland Terrace	05135	JGH	TP-01

PROJECT LOCATION	SURFACE ELEVATION	DATE	START TIME	FINISH TIME
NW Pacific Hwy at Larsen Dr, La Center, WA	195 feet amsl	09/13/05	0815	0915

Depth	Sample No. 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									10 to 12 inches topsoil with grass rootlets
2 ft	S1.1 bag	19	4.5+		Gee Silt Loam	CL	A-4(5)	14.5	Lean CLAY with sand, brown with gray and orange-brown mottling, moist, very stiff to hard, low plasticity, fine- to medium-grained sand nuclear density test results at 2 feet: wet density = 105.7 pcf, dry density = 94.5 pcf, moisture content = 11.8% sample S1.1 from 2 feet submitted for laboratory analysis: moisture content = 14.5%, silt and clay content = 80.8%, liquid limit = 25.6%, plasticity index = 8.0%
4 ft		10	4.0					18.5	nuclear density test results at 4 feet: wet density = 113.4 pcf, dry density = 95.7 pcf, moisture = 18.5%, void ratio = 0.761
8 ft									increasing clayey fines
9 ft						CL	A-6(17)		Lean CLAY, brown with gray mottling, moist, very stiff, moderate plasticity, some fine-grained sand
10 ft	S1.2 bag		2.4	1.0				29.5	sample S1.2 from 10 feet submitted for laboratory analysis: moisture content = 29.5%, silt and clay content = 91.9%, liquid limit = 36.7%, plasticity index = 18.1%
12 ft									trace medium-grained sand
14 ft	S1.3 bag							22.8	increasing plasticity
17–20 ft									Bottom of test pit excavation at 17 feet. No ground water encountered.

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.
Highland Terrace	05135	JGH	TP-02

PROJECT LOCATION	SURFACE ELEVATION	DATE	START TIME	FINISH TIME
NW Pacific Hwy at Larsen Dr, La Center, WA	233 feet amsl	09/13/05	0925	1045

Depth	Sample No. 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)
									10 to 12 inches topsoil with grass rootlets
1 ft – 2 ft					Gee Silt Loam	CL	A-4		Lean CLAY with sand, brown with gray and orange-brown mottling, moist, very stiff to hard, low plasticity, fine- to medium-grained sand
3 ft	S2.1 bag	12	4.5+	0.8				19.7	nuclear density test results at 3 feet: wet density = 117.8 pcf, dry density = 98.4 pcf, moisture = 19.7%, void ratio = 0.717
4 ft – 6 ft									
7 ft – 8 ft	S2.2 bag							22.0	Troutdale Formation Sedimentary Conglomerate, brown with yellowish brown and reddish brown mottling, severely to completely weathered, clayey soil matrix with subrounded to rounded gravel and occasional cobbles, fine- to coarse-grained sand
9 ft – 10 ft									sample S2.2 from 8 feet submitted for laboratory analysis: moisture content = 22.0%, silt and clay content = 53.9%, sand content = 32.2%, gravel content = 13.8%, liquid limit = 32.6%, plasticity index = 14.8%
11 ft – 12 ft									
13 ft	S2.3 bag							43.8	increasing moisture
14 ft – 16 ft									
17 ft	▽								[ground water seep at 17 feet]
18 ft									color grades to gray and light brown, completely weathered pockets of sandstone and siltstone
19 ft – 20 ft									
21 ft – 22 ft									Bottom of test pit excavation at 22 feet. Ground water seep encountered at 17 feet.

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.
Highland Terrace	05135	JGH	TP-03

PROJECT LOCATION	SURFACE ELEVATION	DATE	START TIME	FINISH TIME
NW Pacific Hwy at Larsen Dr, La Center, WA	231 feet amsl	09/13/05	1050	1135

Depth	Sample No. 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									10 to 12 inches topsoil with grass rootlets
1–6 ft			4.5+		Gee Silt Loam	CL	A-4	~15	Lean CLAY with sand, brown with gray and orange-brown mottling, moist, very stiff to hard, low plasticity, fine-grained sand
6–9 ft						CL	A-6	~25	Lean CLAY, brown with black mottles, moist, very stiff, moderate plasticity, some fine- to medium-grained sand
9–14 ft									Troutdale Formation Sedimentary Conglomerate, brown with gray and black mottling, severely to completely weathered, clayey soil matrix with subrounded to rounded gravel and occasional cobbles, fine- to coarse-grained sand
12 ft	S3.1 bag							47.9	color grades to reddish brown
14–20 ft									Bottom of test pit excavation at 14 feet. No ground water encountered.

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.
Highland Terrace	05135	JGH	TP-04

PROJECT LOCATION	SURFACE ELEVATION	DATE	START TIME	FINISH TIME
NW Pacific Hwy at Larsen Dr, La Center, WA	200 feet amsl	09/13/05	1145	1220

Depth	Sample No. 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)
									10 to 12 inches topsoil with grass rootlets
1 ft									
2 ft					Gee Silt Loam	CL	A-4	~15	Lean CLAY with sand, brown, moist, very stiff to hard, low plasticity, fine-to medium-grained sand
3 ft			4.5+						
4 ft									
5 ft									
6 ft			2.0	0.6					
7 ft									
8 ft	S4.1 bag					CL	A-6	~30	Lean CLAY, brown with gray mottling, moist, very stiff to hard, moderate plasticity, fine -grained sand
9 ft									
10 ft									
11 ft									
12 ft			2.0						
13 ft									
14 ft									increasing fine sand, occasional black mottles
15 ft									
16 ft									
17 ft									
18 ft									Bottom of test pit excavation at 17 feet. No ground water encountered.
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.
Highland Terrace	05135	JGH	TP-05

PROJECT LOCATION	SURFACE ELEVATION	DATE	START TIME	FINISH TIME
NW Pacific Hwy at Larsen Dr, La Center, WA	242 feet amsl	09/13/05	1225	1305

Depth	Sample No. 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									10 to 12 inches topsoil with grass rootlets
2 ft					Gee Silt Loam	CL	A-4	~15	Lean CLAY with sand, brown, moist, very stiff to hard, low plasticity, fine-to medium-grained sand
3 ft			4.5+						
5 ft						CL	A-6		Sandy CLAY, brown with gray and yellowish brown, green and black mottling, moist, very stiff to hard, low to moderate plasticity, fine- to coarse-grained sand, some fine to coarse subrounded gravel
6 ft	S5.1 bag		4.5+					34.9	
7 ft									
9 ft									Troutdale Formation Sedimentary Conglomerate, reddish brown, severely to completely weathered, fine- to coarse-grained sandy soil matrix with subrounded to rounded gravel and occasional cobbles to 6 inches
10 ft	S5.2 bag							41.7	
11 ft									
12 ft									
13 ft									increasing moisture, occasional gray sandy clay pockets
14 ft									
15 ft									
16 ft									Bottom of test pit excavation at 16 feet. No ground water encountered.
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.
Highland Terrace	05135	JGH	TP-06

PROJECT LOCATION	SURFACE ELEVATION	DATE	START TIME	FINISH TIME
NW Pacific Hwy at Larsen Dr, La Center, WA	265 feet amsl	09/13/05	1330	1445

Depth	Sample No. 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									10 to 12 inches topsoil with grass rootlets
1–2 ft					Cove Silty Clay Loam	CL	A-6	~15	Lean CLAY with sand, brown, moist, very stiff, low plasticity, fine-grained sand
3–5 ft	S6.1		3.2	1.6	Cove Silty Clay Loam	CH	A-7-6(45)	28.7	Fat CLAY, gray, moist, very stiff, high plasticity, trace sand nuclear density test results at 3 feet: wet density = 117.7 pcf, dry density = 90.1 pcf, moisture = 30.7%, void ratio = 0.871 sample S6.1 from 3 feet submitted for laboratory analysis: moisture content = 28.7%, silt and clay content = 91.3%, liquid limit = 63.4%, plasticity index = 45.2%
5–16 ft						CL	A-6	~30	Lean CLAY, orangish brown with gray mottling, moist, very stiff, low to moderate plasticity, some fine-grained sand color grades to light brown, decreasing plasticity
16–20 ft									Bottom of test pit excavation at 16 feet. No ground water encountered.

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.
Highland Terrace	05135	JGH	TP-07

PROJECT LOCATION	SURFACE ELEVATION	DATE	START TIME	FINISH TIME
NW Pacific Hwy at Larsen Dr, La Center, WA	270 feet amsl	09/13/05	1450	1540

Depth	Sample No. 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									10 to 12 inches topsoil with grass rootlets
1 ft					Hesson Clay Loam	CL	A-4	~15	Lean CLAY with sand, brown, moist, very stiff, low plasticity, fine- to medium-grained sand
2 ft									
3 ft			4.5+	1.5					decreasing sand
4 ft									
4.5 ft						CL	A-4	~20	Lean CLAY, orangish brown with light gray mottles, moist, very stiff, low plasticity, trace fine- to medium-grained sand
5 ft									
6 ft									
7 ft									
8 ft									
9 ft									
10 ft									
11 ft									
12 ft									Bottom of test pit excavation at 12 feet. No ground water encountered.
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.
Highland Terrace	05135	JGH	TP-08

PROJECT LOCATION	SURFACE ELEVATION	DATE	START TIME	FINISH TIME
NW Pacific Hwy at Larsen Dr, La Center, WA	263 feet amsl	09/13/05	1545	1610

Depth	Sample No. 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									8 to 10 inches topsoil with grass rootlets
1–3 ft			4.5+ tsf	1.0 tsf	Hesson Clay Loam	CL	A-4	~15	Lean CLAY with sand, brown, moist, very stiff, low plasticity, fine- to medium-grained sand
3–11 ft						CL	A-6	19.6	Sandy CLAY, gray with orangish brown mottling, moist, very stiff, low plasticity, fine- to coarse-grained sand, some fine gravel nuclear density test results at 4 feet: wet density = 130.0 pcf, dry density = 108.9 pcf, moisture = 19.6%, void ratio = 0.551
11–14 ft									Troutdale Formation Sedimentary Conglomerate, brown with gray mottling, severely to completely weathered, sandy soil matrix with subrounded to rounded gravel and occasional cobbles, fine- to coarse-grained sand
14–20 ft									Bottom of test pit excavation at 14 feet. No ground water encountered.

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.
Highland Terrace	05135	JGH	TP-09

PROJECT LOCATION	SURFACE ELEVATION	DATE	START TIME	FINISH TIME
NW Pacific Hwy at Larsen Dr, La Center, WA	265 feet amsl	09/13/05	1615	1645

Depth	Sample No. 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									8 to 10 inches topsoil with grass rootlets
1–8 ft			4.5+ tsf	1.0 tsf	Hesson Clay Loam	CL	A-4	~15	Lean CLAY with sand, brown, moist, very stiff, low plasticity, fine-grained sand
8–17 ft	S9.1 bag (9 ft)							26.2	Troutdale Formation Sedimentary Conglomerate, brown with gray mottling, severely to completely weathered, sandy soil matrix with subrounded to rounded gravel and occasional cobbles, fine- to coarse-grained sand sample S9.1 from 9 feet submitted for laboratory analysis: moisture content = 26.2%, silt and clay content = 40.7%, sand content = 41.8%, gravel content = 17.5%
	S9.2 bag (14 ft)							26.6	
17–20 ft									Bottom of test pit excavation at 17 feet. No ground water encountered.

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.

APPENDIX C
SOIL CLASSIFICATION INFORMATION

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.

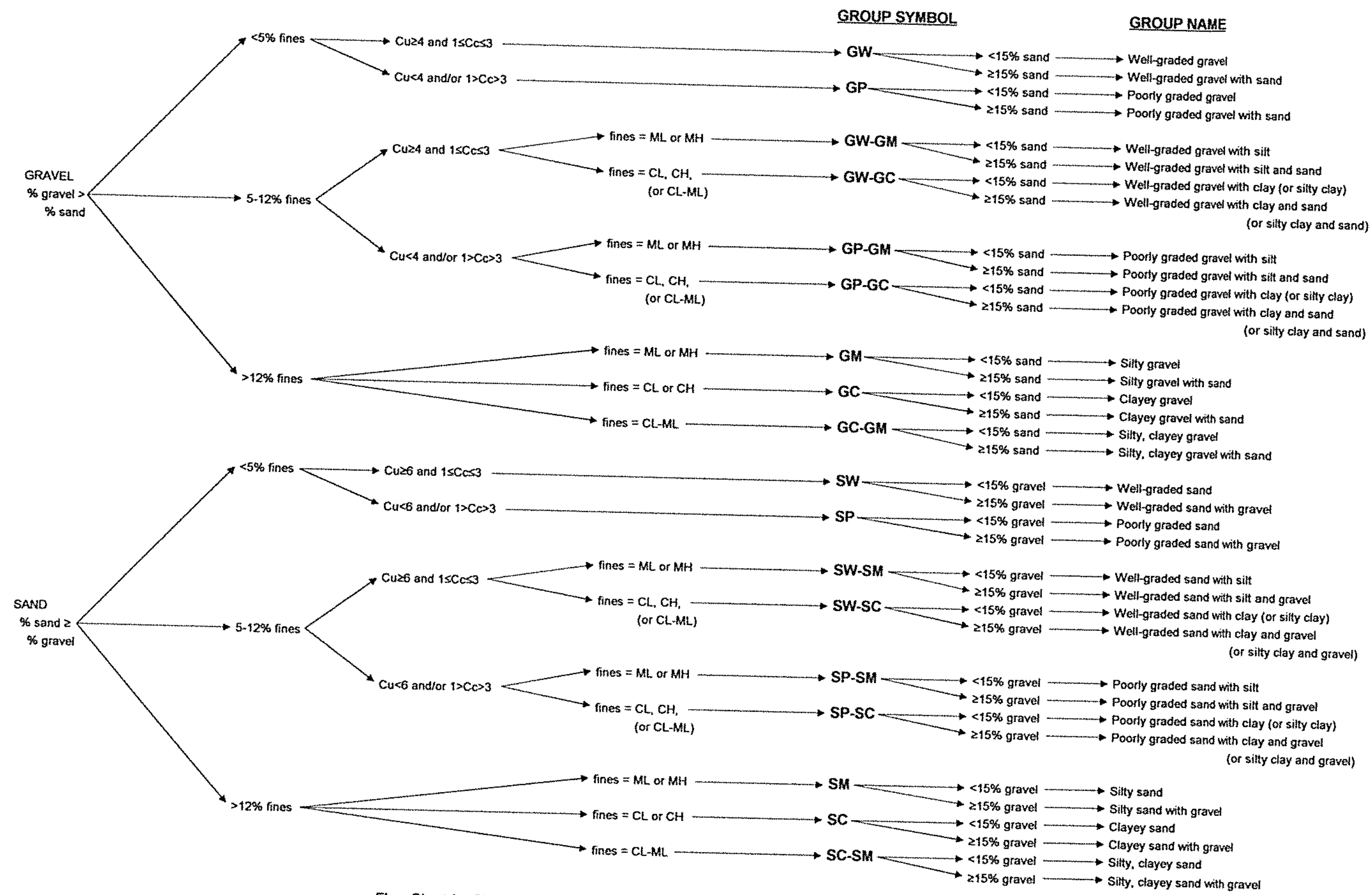
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

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			

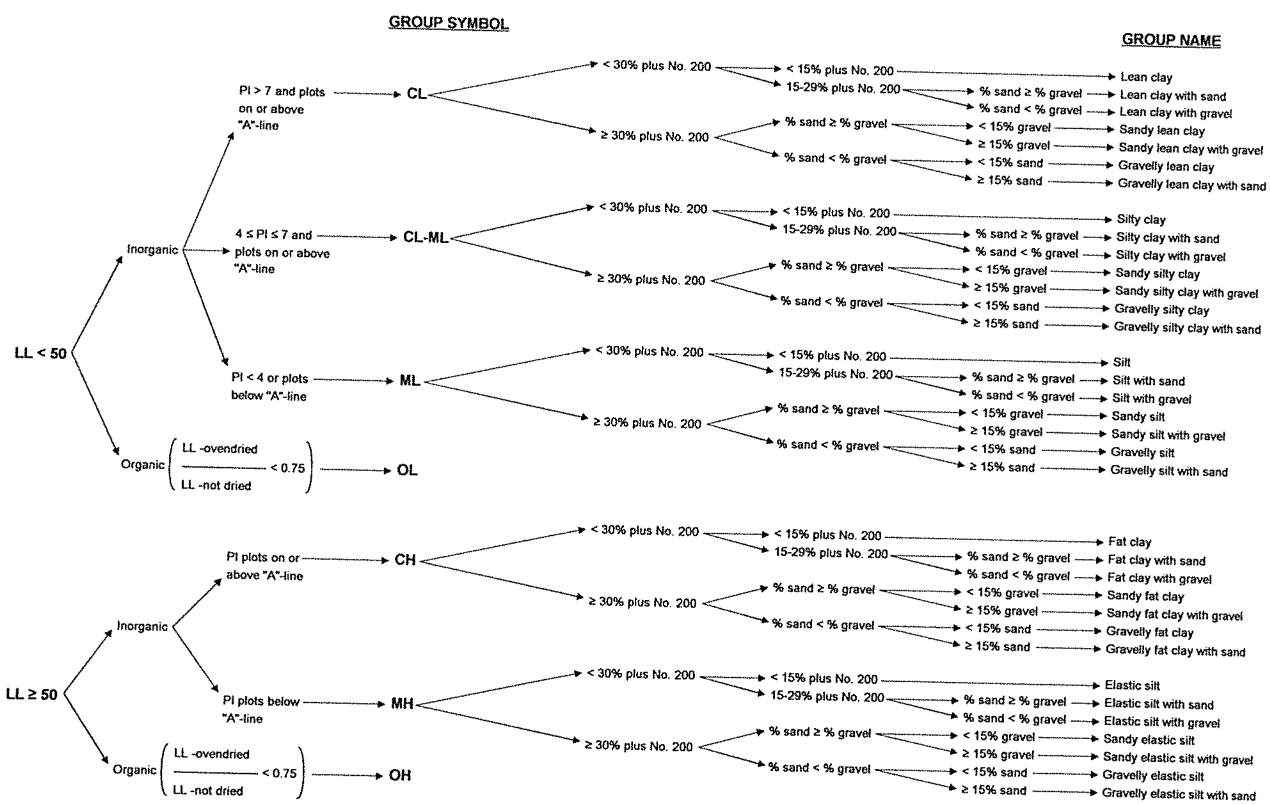
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

ASTM SOIL CLASSIFICATION SYSTEM



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)

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

BOULDERS	COBBLES	GRAVEL	SAND	SILT	CLAY
12 to 36 inches	3 to 12 inches	¾ inch to 3 inch (coarse gravel) ¼ inch to ¾ inch (fine gravel)	# 10 to # 4 sieve (coarse sand) # 40 to # 10 sieve (medium sand) #200 to # 40 sieve (fine sand)	0.001 mm to # 200 sieve (AASHTO system)	Less than 0.002 mm (AASHTO system)

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	more than 4.0
Very Hard	more than 60	N/A

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.

APPENDIX D
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Columbia West Engineering, Inc.

Date: September 30, 2005
Project: Highland Terrace, 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.

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Consultant Responsibility

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