Geotechnical Report
Hands on Children's Museum
Olympia, Washington

March 19, 2009

Prepared for

City of Olympia

LANDAU ASSOCIATES

950 Pacific Avenue, Suite 515
Tacoma, WA 98402
(253) 926-2493

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

This report summarizes the results of our geotechnical engineering services conducted to support preliminary design of the City of Olympia's (City) proposed Hands On Children's Museum (HOCM) project located in Olympia, Washington. The purpose of our services was to complete investigations to characterize subsurface soil and groundwater conditions at the site and to develop geotechnical conclusions and recommendations suitable for use by the design-build team to complete design of the proposed improvements. Additional subsurface explorations will likely be required in order to complete final design. The scope of the additional subsurface explorations will depend on the foundation alternative selected by the design-build team,

The general project area is shown on the Vicinity Map (Figure 1). The general configuration of the project area, the proposed improvements, some of the existing site features, and the location of the geotechnical explorations completed for this study and by others in the project vicinity are shown on the Site and Exploration Plan (Figure 2). Appendix A presents a description of the field explorations and summary logs of conditions observed in the explorations. Appendix B presents a description and results of the laboratory testing program. Logs of previous explorations are included in Appendix C.

This report has been prepared based on our discussions with representatives of the City, Miller Hull Partnership, LLC (project architect), and AHBL (project engineer); our review of readily available subsurface information in the project area; a base map of the site provided by the City; data collected during our field exploration program; our familiarity with geologic conditions within the vicinity of the project; and our experience on similar projects.

1.1 PROJECT UNDERSTANDING

Based on information provided to us, we understand the project consists of constructing the HOCM on Lot 5 on the Port of Olympia's East Bay property. As currently envisioned, the HOCM is anticipated to be approximately 175 ft long by 75 ft wide. Foundation alternatives being considered include pile foundations, mat foundations, and stone columns.

At-grade parking is planned in the portion of the property north of the proposed HOCM structure. In the area to the south and east of the HOCM structure, the improvements are anticipated to consist of several minor structures (Lookout Structure, Garden Shed, Tensile Shade Structures, and Suspension Bridge) amidst several architectural/educational features. These minor structures are expected to be supported by shallow-foundations.

We understand that the improvements will be designed and constructed by a design-build team. As the project evolves, the design-build team may need to complete additional subsurface explorations and complete additional geotechnical engineering analysis. We further understand that there is soil/groundwater contamination at the site which is being addressed by others.

1.2 SCOPE OF SERVICES

Landau Associates was contracted by the City of Olympia to provide geotechnical services to support the project. Our services were provided in accordance with the terms and conditions of our 2007 – 2008 On-Call General Services Agreement between the City and Landau Associates and our Revised Proposal for Geotechnical Services dated December 3, 2008. A task order signed by Mr. Rick Dougherty was received on January 21, 2009. Verbal authorization to proceed was provided earlier.

To support the proposed project, we provided the following specific services:

- Reviewed readily available geologic and geotechnical information in the project vicinity.
- Advanced three (3) geotechnical borings (B-1 through B-3) to depths of between 19 and 81½ ft below the existing ground surface (BGS), and three (3) cone penetrometer (CPT) soundings (CPT-1 through CPT-3) to depths of between 22.5 and 120 ft BGS to characterize subsurface soil and groundwater conditions at the site.
- Completed geotechnical laboratory testing consisting of natural moisture content determinations, fines content determinations, grain size analyses, and Atterberg limit determinations on selected soil samples recovered from our borings.
- Completed geotechnical engineering analyses and developed geotechnical engineering conclusions and recommendations to support preliminary design.
- Prepared and submitted this geotechnical report summarizing our field investigations and preliminary geotechnical engineering conclusions and recommendations for the project. The report includes:
    - a site plan showing the locations of the explorations completed for this investigation
    - descriptive summary logs of the conditions encountered in the explorations completed for this study
    - a summary of surface and subsurface conditions observed in the project area
    - a summary of the environmental issues at the site
    - recommendations for site earthwork including: wet weather construction considerations, site preparation activities, fill placement and compaction criteria, and subgrade preparation for slab-on-grade floors and pavement areas
    - seismic design criteria including an evaluation of the liquefaction and lateral spreading potential at the site and seismic design parameters per the 2006 IBC
    - an evaluation of the feasibility of supporting the HOCM building on a mat foundation with ground improvement. Recommendations are provided for the anticipated ground

improvement, mat foundation subgrade preparation, capillary break, mat foundation deflections, resistance to lateral loads, and foundation drainage considerations

- recommendations for deep foundations of the HOCM building including: pile type(s), recommended tip elevations, allowable axial capacity, downdrag loads, settlement, uplift, lateral pile capacity, impact of lateral spreading on the piles, and pile design construction considerations

- evaluation of slab-on-grade floors versus a structural slab including a recommended value for subgrade modulus for preliminary deflection analysis of slab-on-grade floors

- recommendations for helical anchor foundation to support minor structures

- recommendations for below-grade retaining walls

2.0 EXISTING CONDITIONS

This section provides a discussion of the general surface conditions, geologic setting, and subsurface conditions observed at the proposed Hands On Children's Museum site at the time of our investigation. Interpretations of the site conditions are based on the results of our review of available information, site reconnaissance, subsurface explorations, and laboratory testing.

2.1 SURFACE CONDITIONS

The proposed Hands On Children's Museum property (subject property) will be situated on Lot 5 of the Port of Olympia's East Bay property. Lot 5 is approximately 1.86 acres in size. The former East Bay Warehouse, which was pile supported, is partially located on Lot 5. A concrete slab-on-grade from the former warehouse covers much of the site. The floor slab has a significant amount of cracking and is uneven. Lot 5 is bounded by undeveloped port property to the north, west, and south and by Marine View Drive and East Bay to the east. Land use in the project area is primarily commercial/industrial. The property is fairly level, with elevations ranging from 8 to 11 ft (Thurston County 2009).

2.2 GEOLOGIC SETTING

Geologic information for the project area was obtained from the *Geologic Map of the Tumwater 7.5-Minute Quadrangle, Thurston County, Washington* (Walsh 2003) published by the Washington State Department of Natural Resources. According to the above-referenced geologic map, near-surface deposits in the project area are mapped as glacial recessional outwash. Soil defined as recessional outwash typically consists of stratified deposits of sand and gravel. Silt and fine-grained sand are common in portions of the unit, as are lacustrine deposits. Sorting, cross and horizontal stratification, and cut and fill structures are distinctive features of recessional outwash. Recessional outwash is transported by meltwater streams emanating from the face of the ablating glacier. At the project site, the recessional deposits are primarily composed of silt (i.e. lacustrine deposits) and fine-grained sand.

2.3 FIELD EXPLORATION PROGRAM

Soil and groundwater conditions at the proposed Hands On Children's Museum site were explored on January 8 and 9, 2009 by advancing three borings (B-1 through B-3) and three CPT soundings (CPT-1 through CPT-3). The borings were advanced with a truck-mounted drill rig. Borings B-1 and B-2 were advanced to a depth of approximately 19 ft BGS using hollow-stem auger drilling techniques. Boring B-3 was advanced to a depth of about 81½ ft BGS utilizing mud rotary drilling techniques. The CPT soundings were advanced utilizing truck-mounted CPT equipment. CPT-1, CPT-2, and CPT-3 soundings were advanced to depths of 22¾ ft, 120 ft, and 109 ft BGS, respectively. The approximate locations of the explorations completed by Landau Associates for this study are shown on the Site and Exploration Plan, Figure 2. A detailed discussion of the field exploration program, together with edited logs of the exploratory borings, is presented in Appendix A. A discussion of the geotechnical laboratory testing, together with the lab results, is presented in Appendix B.

2.4 SUBSURFACE SOIL CONDITIONS

Subsurface conditions across the site generally consist of fill overlying recessional deposits. For the purpose of this report, the recessional deposits were divided into three subunits (upper, intermediate, and lower) based on gradation characteristics, density, and other physical characteristics.

AMEC Earth and Environmental, Inc. (2007) advanced a series of borings for the proposed expansion of the LOTT treatment plant. Borings AB-8 and AB-9 were advanced immediately to the west of the proposed HOCM. GeoEngineers (2007) completed a series of monitoring wells on the HOCM property (MW01 through MW04) for the remedial investigation of the Port of Olympia's East Bay property. The conditions observed in the explorations completed by AMEC and GeoEngineers are similar to the conditions observed in our explorations. The location of the explorations advanced by others is presented on the Site and Exploration Plan, Figure 2. Summary logs of these explorations are included in Appendix C.

2.4.1 FILL

Fill was observed in all of the explorations completed for this study. The fill is estimated to be between 15½ and 18 ft thick. The upper 2 to 10 ft of fill observed in all of the explorations advanced for this study consists of loose to dense sand or gravel with variable silt content. Fill interpreted to consist of medium dense, silty sand and stiff to very stiff, sandy silt is present between 2 and 5 ft BGS in sounding CPT-1 and between 3 and 5½ ft BGS in sounding CPT-2. The lower portion of the fill consists of wood debris. The wood debris consists of relatively fine wood fragments and ranges in thickness from 8 to 10½ ft.

2.4.2 UPPER RECESSIONAL DEPOSITS

Soil classified as upper recessional deposits were encountered below the fill in all of the explorations completed for this project. In borings B-1 and B-2 and sounding CPT-1, the upper recessional deposits were encountered throughout the depths explored (19 to 22½ ft BGS). In the remaining explorations completed at the site (B-3, CPT-2, and CPT-3), the recessional deposits were encountered to depths of between 32½ and 33 ft BGS. The upper recessional deposits observed in the borings consist of loose to medium dense, sand or gravel with variable silt content. In the soundings, the upper recessional deposits were interpreted to consist of medium dense to dense sand to silty sand with scattered interbeds of stiff silt.

2.4.3 INTERMEDIATE RECESSIONAL DEPOSITS

Soil interpreted to be intermediate recessional deposits were encountered in boring B-3 from 32½ to 73½ ft BGS, in sounding CPT-2 from 33 to 66 ft BGS, and in sounding CPT-3 from 33 to 61 ft BGS. The intermediate recessional deposits encountered in boring B-3 were observed to consist of medium stiff to stiff, sandy to very sandy silt; silt with sand to trace sand; and silty clay. In the CPT soundings, the intermediate recessional deposits were interpreted to consist of medium stiff to stiff silt with interbeds of sandy silt.

2.4.4 LOWER RECESSIONAL DEPOSITS

The lower recessional deposits were encountered below the intermediate recessional deposits throughout the maximum depths explored in boring B-3 (81½ ft BGS) and in soundings CPT-2 (120 ft BGS) and CPT-3 (109 ft BGS). In boring B-3, the lower recessional deposits were observed to consist of medium dense to dense, sand with silt. In the soundings, the lower recessional deposits are interpreted to consist of medium dense to very dense sand with variable silt content. Interbeds of very stiff to hard, silt are interpreted to be present in sounding CPT-3.

2.5 GROUNDWATER

At the time of our field investigations in early-January 2009, groundwater was measured or interpreted to be between 4 and 6 ft BGS in the explorations completed for this study. As part of the remedial investigation completed on the East Bay Property for the Port of Olympia, GeoEngineers (2007) installed a series of monitoring wells. The monitoring wells were measured on August 28, 2007. In the wells installed near Lot 5, groundwater was reportedly encountered between elevation 5.3 and 6.6 ft NGVD29. The location of the monitoring wells installed by GeoEngineers is included on Figure 2.

It should be noted that the groundwater conditions reported above are for the specific locations and dates indicated, and therefore may not necessarily be indicative of other locations and/or times. Furthermore, it is anticipated that groundwater conditions will vary depending on local subsurface conditions, the weather, the tide, and other factors. Groundwater levels in the project area are expected to fluctuate seasonally, with maximum groundwater levels generally occurring during the winter and early spring months.

3.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

Based on the conditions observed in the explorations completed for this study, construction of the proposed Hands On Children's Museum (HOCM) is feasible using conventional construction techniques. A discussion of the environmental issues at the site, earthwork, and seismic design considerations are provided in Section 3.1 through Section 3.3 of this report.

In our opinion, there are two feasible methods to support the HOCM. The first alternative includes implementing ground improvement techniques and supporting the structure on a mat foundation. This alternative is discussed in Section 3.4 of this report. The second alternative is to support the HOCM on pile foundations with a pile-supported floor slab. With the pile foundation alternative, the first floor of the HOCM could be constructed as slab-on-grade, though there is a potential of differential settlement between the slab-on-grade floor and the pile-supported structure. Recommendations for pile foundations are provided in Section 3.5 of this report. A discussion of slab-on-grade floors is provided in Section 3.6. Recommendations for foundation support of minor structures are provided in Section 3.7.

As described in this report, once the final design alternative is selected, additional explorations and engineering analysis will be necessary.

3.1 ENVIRONMENTAL CONSIDERATIONS

Gasoline-range petroleum hydrocarbons, diesel- and motor oil-range petroleum hydrocarbons, cPAHs, SVOCs, Metals, and Dioxins/Furans were reportedly encountered within the site soil at concentrations above the MTCA Method A or B clean-up levels. Petroleum hydrocarbons and arsenic contamination were also reportedly encountered in the groundwater at the site (GeoEngineers 2007).

Depending on the clean-up action taken by the Port of Olympia, soil and groundwater removed during site construction may be contaminated. The Contractor should be prepared to monitor excavated soil and groundwater removed from the site during construction. If the contaminant levels are determined to be above the site cleanup action level, the soil and groundwater will need to be managed in accordance with the provisions of the site clean up action plan.

The method of clean up activities selected by the Port of Olympia for the site may change the ground conditions at the site. Landau Associates should review documentation related to the clean-up action selected by the Port of Olympia to determine if the proposed actions effect the geotechnical recommendations of this report and, if necessary, modify the recommendations contained in this report to account for changes in site conditions.

3.2 EARTHWORK

Earthwork to accommodate the proposed improvements is expected to consist of the removal of existing improvements, subgrade preparation for floor slabs and pavement, installation of onsite utilities, and the placement and compaction of structural fill.

3.2.1 WET WEATHER CONSTRUCTION CONSIDERATIONS

Earthwork-related construction will be influenced by weather conditions. Much of the existing soil at the site contains a significant amount of fine sand and silt and will be sensitive to moisture. Site grading activities using moisture-sensitive soil should normally occur during the relatively warmer and drier period between about mid-summer to early fall (typically about July through mid-October). Completing these activities outside of this normal construction window could lead to a significant increase in construction costs due to weather-related delays, repair of disturbed areas, and the increased use of “all-weather” import fill materials.

Because of the moisture sensitivity, unprotected site soil, in either a compacted or uncompacted state, can degrade quickly to a slurry-like consistency in the presence of water and construction traffic. If the subgrade or fill soil becomes loosened or disturbed, additional excavation to expose undisturbed soil and replacement with properly compacted structural fill will be required. For wet weather construction, the contractor may reduce the potential for disturbance of subgrades by the following:

- Protecting exposed subgrades from disturbance by construction activities by constructing gravel working mats
- Using a trackhoe with a smooth-bladed bucket to limit disturbance of the subgrade during excavation
- Suspending earthwork and other construction activities that may damage subgrades during rainy days
- Limiting and/or prohibiting construction traffic over unprotected soil
- Sloping excavated surfaces to promote runoff
- Sealing the exposed surface by rolling with a smooth drum compactor or rubber-tire roller at the end of each working day and removing wet surface soil prior to commencing filling each day.

3.2.2 SITE PREPARATION ACTIVITIES

Site preparation activities are expected to consist mainly of the removal of the slab-on-grade floor and piles associated with the former East Bay Warehouse. The existing slab-on-grade floor may be pulverized, stockpiled, and recycled for use as structural fill. If this alternate is chosen, the Portland cement concrete should be processed to meet the requirements for structural fill described in Section 3.2.4 of this report. If rebar or reinforcing mesh is present in the slab, the rebar or reinforcing mesh should be removed if the Portland cement concrete will be utilized as structural fill. Disposal of the Portland concrete rubble at an approved offsite location is also a viable alternative.

The former East Bay Warehouse piles should either be extracted or cut off at least 3 ft below the lowest proposed foundations level. If the piles are left in place, the location of the existing pile foundations should be surveyed and recorded. If the piles are extracted, we recommend that the hole from the extracted pile be filled with Controlled Density Fill (CDF). CDF should be placed using a tremie pipe. The pipe should be forced as deep as possible (minimum 10 ft below the surface). A sufficient head of CDF should be maintained in the hole as the tremie pipe is extracted. CDF should have an unconfined compressive strength of 50 to 300 psi.

We have not been informed of any utilities that are present across the site. If they exist, water supply lines, storm lines, and other buried lines should be abandoned or relocated prior to construction. Utilities that will be abandoned that are less than 3 ft below the lowest foundations level should be removed and disposed of at an approved off-site location. Deeper pipelines left in place should be grouted full with controlled density fill (CDF) to reduce the potential for differential settlement resulting from collapsed pipes or erosion. It should be noted that large-diameter utility lines that are abandoned in place could create obstructions during pile installation.

All incidental excavations associated with the removal of the existing improvements should be backfilled in accordance with the recommendations in Section 3.2.5 of this report.

3.2.3 SUBGRADE PREPARATION

Following site preparation activities and before placement of any structural fill, the upper 12 inches of exposed soil should be scarified, moisture-conditioned, and compacted as described in Section 3.2.5 of this report. The prepared subgrade should be proof-rolled with a loaded dump truck, large self-propelled vibrating roller, or equivalent piece of equipment in the presence of a qualified geotechnical or civil engineer to check for the presence of soft, loose, and/or disturbed areas. If any soft, loose, and/or disturbed areas are revealed during proof-rolling, these areas should either be moisture-conditioned and recompacted to the required density, or removed and replaced with import structural fill as described in Section 3.2.4 and compacted as described in Section 3.2.5 of this report.

3.2.4 STRUCTURAL FILL

Structural fill is defined as fill placed to support foundations, floors slabs, and pavement areas and used for utility trench backfill. The suitability of excavated soil or imported soil for use as structural fill will depend on the gradation and moisture content of the soil when it is placed. As the amount of fines increases, the soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. Soil containing more than about 5 percent fines cannot consistently be compacted to a dense, non-yielding condition when the water content is greater than about 2 to 3 percent above optimum moisture content. Optimum moisture content is the moisture content at which the greatest compacted dry density can be achieved.

Based on the results of our explorations, most of the near-surface soil across the site (encountered between 2 and 10 ft BGS) consists of granular fill composed of sand with silt and variable gravel or sandy gravel. The granular fill has a moderate amount of fines (between 5 and 15 percent) which indicates that the granular fill is moderately moisture sensitive. The near-surface granular fill located above the groundwater table is near its optimum moisture, however some limited moisture conditioning (i.e. drying or wetting) may be required to use this material for structural fill. If the near-surface soil is used, we recommend that its use be limited to extended periods of dry weather in the summer and early fall months (about July through mid-October) where the moisture can be more easily controlled. Fine-grained fill (interpreted to be present between 2 and 5 ft BGS in sounding CPT-1 and between 3 and 5½ ft BGS in sounding CPT-2) and wood debris are not suitable for use as structural fill and should be wasted at an approved off site location.

If the onsite soil cannot be properly moisture conditioned or the quantity of onsite soil is insufficient, import structural fill will be required. For warm, dry weather conditions (generally July through late September), import structural fill could consist of a well-graded, granular material with less than 15 percent fines (material passing a U.S. No. 200 sieve) and a maximum particle size of 4 inches. The moisture content should be within minus 2 percent to plus 1 percent of the optimum moisture content. If wet weather construction is anticipated, the amount of fines should not exceed 5 percent based on the minus ¾-inch fraction.

3.2.5 BACKFILL AND COMPACTION REQUIREMENTS

Prior to placing structural fill, the exposed subgrade should be prepared in accordance with the recommendations contained in Section 3.2.3 of this report. The initial lifts of fill over the prepared subgrade should be placed in relatively uniform horizontal lifts, not exceeding 6 inches thick, loose measure. The initial lifts of fill should be compacted with multiple passes of a large, steel drum roller without the use of vibration. Fill placed at least 2 ft above the prepared subgrade elevation should be placed in a maximum 10-inch thick, loose measure, lift. Each lift should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D1557 test procedure.

3.2.6 SETTLEMENT

The site is currently situated between elevation 8 and 11 ft (Thurston County 2009). According to drawings provided by Miller-Hull, the finished grade of the proposed HOCM is planned at elevation 13 ft. Moderately compressible soil and significant deposits of wood debris are present below the site. The placement of fill to raise site grades will lead to settlement of these soil layers.

We estimate that between 1 and 2 inches of primary consolidation and immediate settlement may occur if the site is raised by 2 ft. It is anticipated that 25 percent of the settlement described above will occur as the fill is placed. The remaining 75 percent of settlement described above will occur within 2 to 4 months after completion of fill placement.

In addition to the settlement described above, secondary consolidation of the wood chips due to decomposition and reorientation will occur during the design life of the improvements. Based on the conditions observed in the explorations, we estimate that between 8 and 10½ ft wood debris underlies all or portions of the site. The amount of secondary settlement is expected to vary according to the thickness of the wood debris, and the effects of any preloading due either to past site activities or as part of initial site construction activities. Given the uncertainty involved in the assessment of secondary consolidation settlement, we estimate that between 3 and 12 inches or more of post-construction secondary settlement of the wood debris could occur during the design life of the improvements.

The placement of permanent fill to raise site grades or temporary preloading by stockpiles of fill would reduce the amount of post-construction secondary consolidation, however, it would not completely eliminate the potential settlement of the ground surface due to secondary consolidation.

Our scope of services did not include consolidation testing of the wood debris. If the first-floor is supported by a slab-on-grade floor and no ground improvement is implemented, we recommend that the design build team consider completing consolidation testing to further characterize the compressibility characteristics of the wood waste.

3.3 SEISMIC DESIGN CONSIDERATIONS

The Pacific Northwest is seismically active and the site could be subject to ground shaking from a moderate to major earthquake. Consequently, earthquake shaking should be anticipated during the design life of the proposed HOCM and the HOCM should be designed to resist earthquake loading using appropriate design methodology.

According to Palmer et al. (1999), the proposed HOCM site is considered to have a high susceptibility for liquefaction during a major seismic event. During the 1949 Olympia Earthquake (measured peak ground acceleration of 0.30g in downtown Olympia), up to 5 inches of liquefaction induced settlement occurred in the Port of Olympia area (Palmer et al. 1999).

3.3.1 CODE BASED SEISMIC DESIGN

Seismic design of the proposed HOCM will be in accordance with the 2006 IBC (ICC 2006). The near-surface soil at the site is liquefiable. According to the 2006 IBC, the liquefiable soil profile that underlies the site classifies as Site Class F, per Table 1613.5.2. For Site Class F soil profiles, the 2006 IBC requires that the Site Coefficients, F_a and F_v, be determined in accordance with the *Minimum Design Loads of Buildings and Other Structures* (ASCE 7-05) published by the American Society of Civil Engineers (ASCE 2005).

According to Section 11.4.7 of ASCE 7-05, a dynamic site response analysis is required to determine the site coefficients, F_a and F_v, for Site Class F and for structures with periods of vibration greater than 0.5 seconds. At the time this report was prepared, the period of vibration for the HOCM building had not been determined. If the HOCM building has a period of vibration less than or equal to 0.5 seconds, Section 20.3.1 of ASCE 7-05 allows for the site coefficients to be equal to the site coefficients if the project site were not susceptible to liquefaction. In the absence of liquefaction, the site would classify as Site Class E (Soft Soil Profile).

Assuming a period of vibration of the structures of less than or equal to 0.5 second, the following spectral accelerations for a 2 percent probability of exceedance in 50 years should be used to determine the maximum considered earthquake spectral acceleration (USGS 2008):

Spectral Acceleration for short periods (S_S):	116.0% of gravity (1.16g)
Spectral Acceleration for a 1-second period (S_1):	43.5% of gravity (0.44g)

For a Site Class E, a value of 0.90 should be used for site coefficient F_a, and 2.4 for site coefficient F_v. The design spectral response accelerations can be taken as 2/3 of the values determined above.

3.3.2 LIQUEFACTION AND LATERAL SPREADING ANALYSIS AND RESULTS

Liquefaction is defined as a significant rise in pore water pressure within a soil mass caused by earthquake-induced cyclic shaking. The shear strength of liquefiable soil is reduced during large and/or long-duration earthquakes as the soil consistency approaches that of a semi-solid slurry, which can result in significant and widespread structural damage if not properly mitigated. Deposits of loose, granular soil below the water table are most susceptible to liquefaction, although non-plastic and low-plasticity silts and clays are also considered to be susceptible to liquefaction.

Geotechnical data from the deep explorations (B-2, CPT-1, and CPT-3) was analyzed to estimate the factor of safety against liquefaction occurring at the site. The potential for liquefaction was assessed using the "modified simplified procedure" presented by Youd et al. (2001). Where appropriate, recommendations for liquefaction susceptibility evaluations proposed by Idriss and Boulanger (2008) were incorporated into our analysis.

The liquefaction susceptibility of the cohesive soils was assessed using the method proposed by Bray and Sancio (2006). According to the criteria described by Bray and Sancio, any fine-grained deposit with a plasticity index less than 12 and a water content greater than 85 percent of the liquid limit is potentially susceptible to liquefaction. In the CPT soundings, fine-grained soil with an interpreted "Soil Behavior Type Index," I_c, of 2.6 or less is assumed to be susceptible to liquefaction. The depths where the interpreted "Soil Behavior Type Index" is greater than 2.6 generally coincide with the depths in the borings where the soil has a plasticity index greater than 12.

For this study, the maximum depth of liquefaction was assumed to be 70 ft. This depth is slightly less than the depth established by the WSDOT Geotechnical Design Manual (WSDOT 2008a).

The potential for, and magnitude of, lateral spreading was assessed using an empirical approach developed by Youd et al. (2002).

3.3.2.1 Peak Ground Acceleration and Magnitude for Analysis

According to the 2006 IBC, neglecting liquefaction, the soil that underlies the HOCM site classified as Site Class E (Soft Soil Profile) per Table 1613.5.2 (assuming a structure period of vibration equal to or less than 0.5 seconds). For Type E soils, the maximum considered peak horizontal ground acceleration for liquefaction and lateral spread analysis is equal to $0.40S_{DS}$, or 0.28g. An approximate magnitude 7.0 earthquake producing a peak horizontal ground acceleration of 0.28g was used in the liquefaction and lateral spread analyses at the HOCM site.

3.3.2.2 Liquefaction Analysis Results

The computed factor of safety against liquefaction at the proposed HOCM site indicated that much of the coarse-grained soil fill and upper recessional outwash below the groundwater table is less than 1.0. Therefore, it is anticipated that much of the coarse-grained fill and upper recessional outwash deposits are susceptible to liquefaction during the design event.

Based on the results of the laboratory testing of the samples obtained from the borings, most of the fine-grained soil encountered within the intermediate recessional deposits meet the criteria defined by Bray and Sancio and are potentially susceptible to liquefaction. Most of the soil located within the intermediate recessional outwash has a factor of safety of less than 1.0, and may liquefy during the design seismic event. The lower recessional outwash unit is located below the maximum considered depth of liquefaction.

3.3.2.3 Liquefaction Induced Settlement

The amount of post-liquefaction ground subsidence during the design seismic event was estimated using an empirical method developed by Zhang et al. (2002) which is based on areas that had undergone liquefaction. The magnitude of post-liquefaction ground subsidence may be about 7 to 8 inches. The actual magnitude and extent of liquefaction-induced settlement will depend on many factors, including the duration and intensity of the ground shaking during the seismic event, and local soil and groundwater conditions. Therefore, the magnitude of liquefaction-induced settlement may vary from that estimated above.

3.3.2.4 Lateral Spreading

Lateral spreading is a phenomenon where lateral ground displacements occur as a result of soil liquefaction. Lateral spreading is typically observed on very gently sloping ground or on virtually level ground adjacent to slopes or shorelines. Lateral spreading tends to break the upper soil layers into blocks that progressively move downslope during an earthquake. Large fissures at the head of the lateral spread are common, as are compressed or buckled soil at the toe of the soil mass. Lateral spreading has been shown to occur if there are liquefiable soil layers located within a depth equal to twice the depth of the adjacent channel, which for this project is East Bay (Idriss and Boulanger 2008).

The potential for lateral spreading of the banks along the East Bay was analyzed using an empirical procedure presented by Youd et al. (2002), which is based on case histories of observed lateral spreading at sites with liquefiable soil. Studies at sites where lateral spreading has occurred have shown that the magnitude of lateral spreading is typically within a range of one-half to twice the predicted movement.

According to Finlayson (2005), East Bay is less than 6 ft deep (i.e. elevation -6 ft) immediately to the northeast of the project site. For the purpose of this analysis, the slope leading into East Bay was assumed to be inclined at 1¾H:1V (horizontal to vertical). At the east boundary of the proposed HOCM building, the empirical method predicts horizontal movements during the design earthquake of about 1½ ft. At the west edge of the proposed building, the empirical method predicts lateral movement of about 1 ft during the design earthquake. The actual magnitude and extent of lateral spreading will depend on many factors, including the duration and intensity of the ground shaking, and local soil and groundwater conditions. Therefore, actual ground movements may vary from those estimated above.

The empirical approach can be conservative, depending on soil properties and site conditions. Other advanced methods can be used to better estimate lateral spreading, but require significant effort that is not in our scope.

3.3.2.5 Interpretation of Results and Mitigation

At the proposed HOCM, there is a high potential for widespread liquefaction during the design earthquake which will lead to ground subsidence and lateral spreading. Based on the explorations and engineering analysis completed for this project, liquefiable soil could extend to depths of about 70 ft below the existing ground surface (i.e. about elevation -60 ft).

If deep foundations are utilized to support the HOCM building, the deep foundations would need to extend below the liquefiable soil. The liquefaction of soil above the pile tip elevation will lead to the creation of downdrag loads on the building foundation elements. Downdrag loads could lead to pile damage (due to possibly exceeding the structural capacity of the pile) and increased foundation settlement. Ground subsidence also had the potential of damaging slab-on-grade foundations and damaging connections between the pile-supported building and utilities extending outside of the building.

Potential methods to mitigate against liquefaction at the site include: improving the soils such that liquefaction does not occur or allowing liquefaction to occur and designing the improvements to tolerate the consequences of liquefaction and, if necessary, repairing areas of damage caused by liquefaction induced settlement.

Given the anticipated depth of piles needed to support the proposed HOCM, improving the soils such that liquefaction does not occur is a feasible alternative for this project. Recommendations for ground improvement are provided in the following section of this report.

3.4 MAT FOUNDATION ALTERNATIVE

As previously described in this report, the near-surface soil at the site is liquefiable and there is a significant amount of wood debris which could lead to intolerable settlement over the design life of the proposed HOCM. These factors effectively eliminate the alternative of constructing the proposed HOCM on shallow or mat foundations without ground improvement.

3.4.1 GROUND IMPROVEMENT

Ground improvement is typically designed by the contractor. Based on our conversations with Hayward Baker (Koelling 2009), a local ground improvement contractor, ground improvement at the site would likely consist of vibro-replacement stone columns.[1] In order to reduce the amount of spoils brought to the surface, it is anticipated that the vibro-replacement stone columns utilized at the project site would be installed using the dry, bottom feed method. In order to bridge over the wood debris, the upper portion of the stone columns (i.e. the portion of the stone column passing through the wood debris) would be amended with Portland cement to create a vibro concrete columns (VCC).[2] The placement of several feet of import structural fill over the top of the VCC would allow for the HOCM to be supported by a mat foundation.

Installation of stone columns through the entire liquefiable zone (70 ft depth) is likely not cost effective. We recommend that, at a minimum, stone columns be installed through the sandy to very sandy silt layer which is present between 37½ to 45½ ft BGS. If stone columns are installed to 45 ft BGS, approximately 25 ft of liquefiable soil zone would be present below the improved zone. According to data collected by Ishihara (1985), ground surface manifestation of a 25-ft liquefied zone is not evident if the thickness of non-liquefiable near-surface soil (i.e. improved ground) is greater than about 45 ft.

In order to mitigate against the liquefaction risk, the replacement ratio (percentage of total area composed of stone columns) of the stone columns should increase the density/consistency of the improved soil such that the soil has a minimum factor of safety of at least 1.2 for a magnitude 7.0 earthquake generating a 0.28g peak ground acceleration.

At the time this report was prepared, the proposed clean-up methodology of the contaminated soil was unknown to use. If the contaminated soil is capped and left in place, the construction of vibro-replacement stone columns installed utilizing the dry, bottom-feed method will bring a limited amount of contaminated soil/groundwater to the surface. Spoils with contaminant concentrations above the MTCA site clean-up standards will need to be removed and disposed of at an approved location.

---

[1] Detailed information regarding vibro-replacement stone columns, including diagrams describing the installation techniques, are provided on Hayward Baker's webpage. http://www.haywardbaker.com/

[2] The cement content of the VCC is dependent on the thickness and composition of the wood debris. The ground improvement contractor would need to complete additional explorations at the site to further characterize the wood waste and to collect samples for laboratory testing to determine the optimum cement content.

3.4.2 MAT FOUNDATION BEARING PRESSURE AND SUBGRADE PREPARATION

At the time this report was prepared, the load of the proposed HOCM was not available to us (the loads are anticipated to be low). The ground improvement subcontractor should be notified of the anticipated loads and settlement criteria, and the ground improvement should be designed to meet the criteria established by the structural engineer. For frost protection purposes, the exterior edge of the mat foundation should be situated at least 1½ ft below the lowest adjacent grade.

After completion of the ground improvement, we recommend that a minimum of 12 inches of import structural fill be placed over the improved subgrade to provide a working surface. The ground improvement contractor may specify a larger thickness of fill over the improved ground in order to bridge the load between the VCC. The import structural fill should consist of a well-graded, granular material with less than 10 percent fines (material passing a U.S. No. 200 sieve) with a maximum particle size of 3 inches. The moisture content should be within minus 2 percent to plus 1 percent of the optimum moisture content. If wet weather construction is anticipated, the amount of fines should not exceed 5 percent based on the minus ¾-inch fraction.

Prior to placing the working surface/bridging layer, the exposed subgrade should be prepared in accordance with the recommendations contained in Section 3.2.3 of this report. The initial lifts of fill over the prepared subgrade should be placed in relatively uniform horizontal lifts, not exceeding 6 inches thick, loose measure. The initial lifts of fill should be compacted with multiple passes of a large, steel drum roller without the use of vibration. Fill placed at least 2 ft above the prepared subgrade elevation should be placed in maximum 10-inch thick, loose measure, lifts. Each lift should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D1557 test procedure. Trench backfill should be placed in relatively uniform horizontal lifts, not exceeding 10 inches thick, loose measure. Each lift should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D1557 test procedure.

3.4.3 CAPILLARY BREAK

A minimum of 4 inches of clean, free-draining material, such as washed gravel, should be placed beneath the mat foundation to act as a capillary break. Washed gravel should consist of clean, durable rock that is fairly well-graded between coarse and fine, has a maximum particle size of ¾ inches, and has less than 5 percent sand (material passing a U.S. No. 4 sieve). A condensation barrier should be placed beneath interior slab-on-grade floors to prevent condensation of water vapor on the bottom of the floor slab from wicking up through the floor slab. The condensation barrier should consist of reinforced, minimum 10-mil membrane with tape sealed joints. We recommend an inspection of the condensation barrier to verify that all openings have been properly sealed.

If the structural engineer determines that a layer of granular material to facilitate concrete curing is necessary, the layer of granular material should consist of a compacted, 4-inch-thick layer of clean, crushed rock material, such as ¾-inch minus crushed rock. Care should be taken during construction to prevent water penetration into this layer that could become trapped between the slab and vapor retarder. Trapped water beneath the slab may lead to problems with interior flooring materials.

3.4.4 MAT FOUNDATION DEFLECTIONS

For deflection analysis of the mat foundation, we recommend using a vertical coefficient of subgrade reaction, K_{01} of 150 pounds per cubic inch (pci) for a one-foot square plate. This value assumes that at least 2 ft of compacted structural fill underlies the slab-on-grade. The coefficient of vertical subgrade reaction can be proportioned for the diameter slab's reaction area by the expression:

$$K_0 = [(B+1)/(2B)]^2 * K_{01}$$

where B is the effective diameter of the slab's reaction area, in feet, which is a function of both the thickness and flexural strength of the slab, and K_0 itself. Calculation of the value of B is an iterative process. We suggest using an initial value of 15 times the slab thickness.

3.4.5 RESISTANCE TO LATERAL LOADS

Resistance to lateral loads may be assumed to be provided by friction acting on the base of mat foundation and by passive lateral earth pressures acting against the sides of mat foundation. An ultimate coefficient of sliding resistance of 0.50, applied to the vertical dead loads only, may be used to compute frictional resistance. For design purposes, the passive resistance of undisturbed, medium dense to very dense recessional outwash or properly compacted structural fill placed against the sides of foundations may be considered equivalent to a fluid with a density of 250 pounds per cubic foot. The upper 1 ft of passive resistance should be neglected in design if not covered by pavement or floor slabs. The value for coefficient of sliding resistance does not include a factor of safety, and the value for the foundation passive earth pressure has been reduced by a factor of 2.0 to limit deflections to less than 1 percent of the embedded depth.

3.4.6 FOUNDATION DRAINAGE CONSIDERATIONS

Drainage should be provided for all foundations and floor slabs lower than the adjacent exterior grade. The drainage system should consist of a minimum 4-inch-diameter, smooth-walled, heavy-duty, minimum Schedule 40 PVC perforated pipe (with the perforations placed downward) in a minimum 12-inch diameter envelope of 1-inch minus drain gravel. The drain gravel should completely surround the perforated drain pipe and be completely surrounded by a non-woven geotextile such as Mirafi 140N, or equivalent. The top of the perforated pipe should be no higher than the top of the adjacent footing. Foundation drains should discharge into the storm system or an approved location. Roof downspouts should not be introduced into the footing drain, but discharged directly into the site stormwater system or other appropriate outlet by means of a tightline-type system. To reduce the possibility of water ponding and infiltrating into the subsurface near foundations, exterior grades should slope to promote runoff away from the structures.

3.5 DEEP FOUNDATION ALTERNATIVE

The site is underlain by compressible and liquefiable fill and recessional deposits. Unless ground improvement is utilized to reduce the liquefaction potential and to reduce undesirable foundation settlement, the HOCM building will need to be supported on deep foundations bearing below the lowest liquefiable layer. To provide proper support of the building, we recommend that the deep foundations be installed into the medium dense to very dense, recessional outwash deposits (i.e. lower recessional deposits) encountered about 73 ft BGS (i.e. at about elevation -62 ft).

Augercast and driven piles are pile types typically used to support buildings in geologic conditions similar to what is observed at the site. At the HOCM site, augercast piles have the following limitations:

- Augercast piles are typically not feasible if the length to diameter ratio of the pile is greater than 30. Augercast piles have a typical maximum diameter of about 2 ft, consequently, the maximum length of augercast piles is 60 ft (less than the minimum length of piles needed at the site. Longer augercast piles can be constructed, however, an experienced augercast pile contractor and extensive continuity testing will be required.
- At the time this report was prepared, the proposed clean-up methodology of the contaminated soil was unknown to use. If the contaminated soil is capped and left in place, the construction of augercast piles will likely bring contaminated soil/groundwater to the surface. Spoils with contaminant concentrations above the MTCA site clean-up standards will need to be removed and disposed of at an approved location.
- The wood debris encountered in our soil borings is loose, fibrous, and has an open structure. The open structure of the fibrous material could lead to the interconnectivity of the augercast piles and the exposure of the steel reinforcing cage as the grout seeps away from the pile. If this were to occur, the augercast pile capacity could be severely compromised. To mitigate

against this, it is anticipated that the upper 10 to 15 ft of the augercast pile would need to be cased.

3.5.1 DRIVEN PILES

Because of the limitations described above, it is our opinion that driven pipe or driven, prestressed concrete piles may be more feasible pile types for this project. Axial and uplift capacities for 18- and 24-inch diameter piles installed to elevation -70 ft are provided in the following section of this report.

3.5.1.1 Vertical Axial Capacity

Piles to support the proposed HOCM building should be designed for both service and seismic loading conditions. Applied loads for the service loading condition include dead and live loads. During service loading, the axial capacity of a single pile is equal to the tip capacity and the skin capacity developed along the entire length of the pile. Applied loads for the seismic loading case typically include dead loads and inertial loading caused by the earthquake shaking. The axial capacity of a single pile is assumed to be equal to the tip capacity and the portion of the skin capacity developed below the lowest liquefiable layer. For both the service and seismic loading condition, downdrag loads (see Section 3.5.1.2) should also be included if the structure is not able to settle after the application of liquefaction-induced downdrag loads.

The allowable axial capacities for 18- and 24-inch diameter, driven pipe piles installed to elevation -70 ft are presented in the following table. The allowable pile capacities assume that the steel pipe piles are driven closed-ended. For the service loading condition, the allowable pile capacity presented in the table below include a factor of safety of at least 2.5 on the calculated ultimate capacities. For seismic loading conditions, the allowable pile capacities include a factor of safety of at least 1.5 on the calculated ultimate capacity. Interbeds of very stiff to hard, silt or clay are present within the lower recessional deposits. Piles bearing in the interbeds will have a reduced pile capacity. The allowable pile capacities presented in the table below account for the presence of these interbeds.

ALLOWABLE AXIAL PILE CAPACITY (TONS)

Pile Type	Pile Diameter (inches)	Loading Condition	
		Service	Seismic
Steel Pipe Pile	18	97	60
	24	140	100
Prestressed Concrete	18	120	66
	24	170	106

Provided the piles have a center-to-center pile spacing of at least 3D, where D is the diameter of the piles, the pile group capacity can be taken as the sum of the individual pile capacities. If the center-to-center pile spacing is less than 3D, the individual pile capacities should be reduced accordingly.

If a WEAP™ analysis is used to calculate the ultimate pile capacity during pile driving, the ultimate pile capacity should be at least two times the allowable pile capacity during service loading summarized in the table above. Regardless of the results of the WEAP™ analysis, the piles should be driven to at least elevation -70 ft.

3.5.1.2 Downdrag Loads

After a significant earthquake event where liquefaction has occurred, post-liquefaction ground subsidence accumulates as downdrag along the length of the pile. Downdrag will lead to either increased foundation settlement or additional axial loads (i.e. downdrag loads) applied to the pile and the bearing soil. If the foundation is allowed to settle as the downdrag loads are applied (i.e., if the HOCM can tolerate additional total and differential settlement), the pile will shed the downdrag load. If the structure is unable to tolerate the additional total or differential settlement, the piles will need to be installed deeper than elevation -70 ft and the allowable pile capacity may need to be reduced. The piles should be structurally designed to accommodate the downdrag loads developed along the length of the pile if additional total or differential settlement is unacceptable. The table below summarizes the upper bound estimate of downdrag loads of piles driven to elevation -70 ft.

ALLOWABLE AXIAL PILE CAPACITY (TONS)

Pile Type	Pile Diameter (inches)	Downdrag Load (tons)
Steel Pipe Pile	18	120
	24	160
Prestressed Concrete	18	140
	24	185

Downdrag can also occur if structural fill is utilized to raise site grades after pile installation. Consequently, pile foundations should not be installed until most of the settlement created by raising site grades has occurred.

3.5.1.3 Deep Foundation Settlement

Based on the subsurface conditions encountered at the site, settlement of the piles would typically be due to a combination of elastic compression of the pile and elastic settlement of granular soils and very stiff to hard low plasticity silts encountered below a depth of two-thirds the pile length.

Pile settlement is dependent on a number of factors including: the size of the piles, the number of piles in the pile group, the pile spacing, the pile tip elevation, the axial load per pile, and other factors. For the purpose of developing an estimate of pile group settlement, the following assumptions were made:

- 24-inch diameter steel pipe piles
- Pile tip located at approximately elevation -70 ft
- 2 by 2 pile group
- Pile spacing of 3 times the pile diameter (pile group width of 5.3 ft)
- Axial load per pile of 140 tons (pile group load of 560 tons).

We estimate that ½ to 1 inch of settlement will occur during service loading conditions. After a seismic event which leads to downdrag loading, an additional 1 to 1½ inches of settlement may occur. Differential settlement between adjacent pile caps may be as high as 50 percent of the total settlement as described above. It is anticipated that a majority of the settlement described above will occur as the building and downdrag loads are applied to the pile. Pile group settlement should be reevaluated by the design build team once the pile size, pile layout, pile tip elevation, and pile load have been determined.

3.5.1.4 Uplift Capacity

The uplift capacity of a pile group should be taken as the minimum of the uplift resistance of the pile group considered as a block or the sum of the individual pile uplift capacities. The uplift capacity of a pile group can be taken as the weight of the soil located within the pile group and the weight of the piles. We recommend assuming that the effective unit weight of the soil is 53 pcf.

For service loading conditions, the uplift capacity of a single pile can be taken as the skin friction capacity taken along the entire length of the pile and the weight of the piles. For seismic loading conditions, the uplift capacity of a single pile can be taken as the skin friction capacity developed below the lowest liquefiable layer and the weight of the pile. The table below summarizes the allowable uplift capacities for piles driven to elevation -70 ft.

ALLOWABLE UPLIFT CAPACITY OF A SINGLE PILE (TONS)

Pile Type	Pile Diameter (inches)	Loading Condition	
		Service	Seismic
Steel Pipe Pile	18	60	15
	24	80	20
Prestressed Concrete	18	83	18
	24	110	20

For the service loading condition, the allowable uplift capacity includes a factor of safety of 3.0. For the seismic loading condition, the allowable uplift capacity presented above includes a factor of safety of at least 2. The weight of the pile is not included in the uplift capacities presented in the table above.

3.5.1.5 Lateral Capacity

The lateral capacity of piles is predominately governed by soil located within 10 D_p, where D_p is the pile diameter in ft, of the ground surface. The following table provides preliminary input soil parameters for an LPILE™, or other similar computer program, analysis of lateral load behavior during static loading conditions.

RECOMMENDED INPUT SOIL PARAMETERS FOR LATERAL LOAD ANALYSIS DURING STATIC LOADING

Depth (ft BGS)(a	Depth (in BGS)(a)	Soil Type/(Soil Profile Number)	Soil Unit Weight (pci)	Soil Internal Angle of Friction (degrees)	Soil Cohesion (psi)	k-value (pci)	E_{50}
0 to 4	0 to 48	Sand/(4)	0.069	34	0	90	---
4 to 7½	48 to 90	Sand/(4)	0.031(b)	31	---	20	---
7½ to 16(c)	90 to 192(c)	---(c)	0.011(b)(c)	---(c)	---(c)	---(c)	---(c)
16 to 33	216 to 396	Sand/(4)	0.034(b)	34	---	60	---
33 to 70	396 to 840	Clay/(1)	0.031(b)	---	6.94	---	0.010
70 to 100	840 to 1,200	Sand/(4)	0.036(b)	35	---	125	---

Notes:
(a) Depth of soil layers should be taken from the top of the former East Bay Warehouse slab-on-grade floor.
(b) Bouyant Unit Weight
(c) Wood Waste from 7½ to 16 ft BGS. No contribution to lateral capacity from wood waste should be accounted for in design.

As discussed in Section 3.3.2.2 of this report, portions of the near-surface soil may liquefy during the design earthquake. To model this behavior, a second set of input soil parameters should be assigned to the liquefiable soil layers. The input soil parameters for LPILE™ analysis of lateral load behavior during seismic loading is provided in the following table. The seismic loading soil parameters were developed using estimated residual soil strength parameters for the liquefied layers and the saturated soft clay soil model used by LPILE™. This approach is considered an approximation of actual conditions. The validity of this technical approach was confirmed by one of the developers of the LPILE™ computer program (Wang and Reese 1998). The residual soil strength parameters were determined using an approach developed by Idriss and Boulanger (2007).

RECOMMENDED INPUT SOIL PARAMETERS FOR LATERAL LOAD ANALYSIS DURING SEISMIC LOADING

Depth (ft BGS)(a	Depth (in BGS)(a)	Soil Type/(Soil Profile Number)	Soil Unit Weight (pci)	Soil Internal Angle of Friction (degrees)	Soil Cohesion (psi)	k-value (pci)	E_{50}
0 to 4	0 to 48	Sand/(4)	0.069	34	---	90	---
4 to 7½	48 to 90	Clay/(1)	0.031(b)	---	0.5	---	0.020
7½ to 16(c)	90 to 192(c)	---(c)	0.011(b)(c)	---(c)	---(c)	---(c)	---(c)
16 to 27	192 to 324	Sand/(4)	0.034(b)	34	---	60	---
27 to 33	324 to 396	Clay/(1)	0.034(b)	---	8.33	---	0.005
33 to 37	396 to 444	Clay/(1)	0.031(b)	---	6.94	---	0.010
37 to 56	444 to 672	Clay/(1)	0.031(b)	---	3.13	---	0.020
56 to 60	672 to 720	Clay/(1)	0.031(b)	---	6.94	---	0.010
60 to 70	720 to 840	Clay/(1)	0.031(b)	---	2.5	---	0.020
70 to 100	840 to 1,200	Sand/(4)	0.036(b)	35	---	125	---

Notes:
(a) Depth of soil layers should be taken from the top of the former East Bay Warehouse slab-on-grade floor.
(b) Bouyant Unit Weight
(c) Wood Waste from 7½ to 16 ft BGS. No contribution to lateral capacity from wood waste should be accounted for in design.

To account for group effects, the soil resistance, p, shall be reduced as specified in the following table (WSDOT 2008a). The multipliers (Pm) are a function of the center-to-center spacing of the piles (expressed in multiples of pile diameter, D) in the group in the direction of the loading. Values for intermediate pile spacing values may be interpolated from the values provided in the table below.

PILE LOAD MODIFIERS, Pm

Pile Center to Center Spacing	Pile Load Modifiers, Pm		
	Row 1	Row 2	Row 3
2D	0.45	0.33	0.25
3D	0.70	0.50	0.35
5D	1.00	0.85	0.70

It should be noted that the input soil parameters for lateral load analysis during seismic loading are based on the conditions observed in boring B-3 and sounding CPT-2, which were completed in the footprint of the proposed HOCM. The conditions observed in the explorations completed outside of the HOCM building indicated the potential for more extensive liquefaction and a different set of input soil parameters for lateral load analysis during seismic loading may be appropriate. We recommend that the design-build team consider completing another deep exploration in the eastern third of the proposed HOCM to evaluate the liquefaction potential at this location.

3.5.1.6 Impact of Lateral Spreading on Pile Capacity

Lateral spreading will impose lateral pressures on the piles. We recommend that a force based approach be utilized to evaluate the lateral pressures on the piles due to lateral spreading. In the force based approach, the lateral pressures exerted on the piles are as follows (WSDOT 2008a):

- Non-liquefied crustal layers (i.e. layers located above the lowest liquefiable layer with the potential to lead to lateral spreading) exert full passive pressure on each pile.
- Liquefied soil exerts a pressure equal to 30 percent of the total overburden pressure.

Based on the conditions observed in boring B-3 and sounding CPT-2, the only liquefiable soil with the potential to lead to lateral spreading is between about 4 and 7½ ft BGS. Figure 3 presents the lateral pressure imposed on a deep foundation due to lateral spreading. The lateral pressures described above should be assumed to act over one pile diameter.

The conditions observed in the explorations completed outside of the HOCM building footprint (B-1, B-2, and CPT-3) indicated the potential for more extensive liquefaction and lateral spreading. Sounding CPT-1 was advanced in the eastern third of the proposed HOCM structure. This exploration was terminated at a shallow depth due to hitting an obstruction and deep subsurface information was not obtained. If the conditions observed in the eastern third of the HOCM are similar to the conditions observed outside of the footprint of the HOCM structure, additional liquefaction and lateral spreading could occur in this portion of the building. This could lead to increased lateral pressures on the piles located in this area. We recommend that the design-build team consider advancing another deep (i.e. 40 to 50 ft) exploration in this portion of the structure to further evaluate the liquefaction and lateral spread potential and its impact on deep foundations.

3.5.1.7 Pile Construction Considerations

Driven piles to support the HOCM are expected to consist of closed-ended, steel pipe or prestressed concrete piles. The piles will be predominately driven through loose to dense sand with variable silt content and medium stiff to very stiff silt.

Pile driving is a dynamic process and it is not uncommon for pile tip depths, as determined by driving resistance, to differ from the pile tip depths estimated from static methods of analysis. We recommend driving several test piles, evenly spaced throughout the footprint of the HOCM, to evaluate the driving criteria and required pile length prior to installation of production piles. The test piles should be driven utilizing the same methods and equipment as the production piles. Test piles should be sized at least 20 ft longer than the pile length recommended in the preceding sections. Test piles may be incorporated in the final structure if they meet the required driving resistance. Once the driving criteria and tip elevation is established, production piles may be driven to the minimum tip elevation established by the test pile program.

Between about 5 and 15 ft BGS, wood debris was encountered in our explorations. The wood debris encountered in our explorations was observed to consist of relatively fine wood fragments (sawdust); however, larger pieces of wood or logs may be present in the debris. If encountered, logs could obstruct pile driving and possibly result in damage to the piles. In a boring advanced by AMEC Earth and Environmental (2007) immediately to the west of the HOCM site, refusal of continued drilling was encountered within the wood debris. If an obstruction is encountered and the pile cannot be advanced, or the pile becomes damaged, the pile may need to be abandoned and relocated. Abandoned piles should either be extracted or cut off at least 3 ft below the bottom of the pile cap. The use of a conical driving pile tip may be considered to aid in penetrating through obstructions, if encountered.

Pile driving should be accomplished in accordance with Section 6-05 of the 2008 *Standard Specifications for Roadway, Bridge, and Municipal Construction* (WSDOT *Standard Specifications*; WSDOT 2008b). The hammer chosen to drive the pile should have a rated energy meeting the requirements in Section 6-05.3(9)B of the 2008 WSDOT *Standard Specif*ications. Because of the expected small number of piles, the required driving criteria may be determined in accordance with Section 6-05.3(12) of the 2008 WSDOT *Standard Specifications*. Alternatively, a WEAP™ analysis could be completed to evaluate pile driving criteria.

A WEAP™ analysis could also be utilized to evaluate the pile driving equipment required to drive the pile to the selected depth. The ultimate pile capacity calculated using static pile capacity estimation techniques is an input in WEAP™ analysis. It should be noted that the ultimate axial pile capacity presented in Section 3.5.1.1 of this report have been reduced to account for weaker soils present below the estimated pile tip elevation. Additionally, dense to very dense sand and gravel layers are present above the anticipated pile tip elevation, including immediately below the wood debris. The WEAP™ analysis should account for these factors.

After pile driving has begun, the pile should be driven continuously until the required ultimate bearing capacity is achieved. The only allowed pauses during pile driving are for splicing, mechanical problems, or other unforeseen events. It has been our experience that when driving in saturated, granular deposits, which are present at the proposed pile tip elevation, piles often do not achieve the required ultimate capacity as calculated from dynamic pile driving equations because of reduced side friction due to "liquefaction" of the soil around the pile during driving. This situation is usually evident when the driving resistance (blows per foot) remains fairly constant with depth. If this is observed to occur during driving of piles, the pile could be redriven after a waiting period of at least 24 hours to evaluate the effect of soil setup. The purpose of the waiting period is to allow dissipation of excess pore water pressure in the soil around the pile. After the waiting period, the pile should be redriven no more than 3 inches. If the pile fails to achieve the required capacity after the redrive, then the pile would need to be lengthened and the driving resumed. Alternatively, a dynamic analysis, such as CAPWAP™, could be performed to evaluate the pile capacity.

Given the expected range of pile capacities, a hammer with a relatively high-rated energy will be necessary to drive the piles to the required depth. Significant levels of vibration and soil densification can occur within about 50 to 100 ft of pile driving. Nearby utilities and structures could be negatively impacted as a result of pile driving. The contractor should evaluate the impact of pile driving on nearby settlement sensitive structures.

Since the effect of pile driving can be felt several hundred feet away from the pile, we recommend that the City complete a preconstruction survey of all non-port owned structures within a distance of at least 250 ft of pile driving operations. The purpose of the preconstruction survey is to document existing conditions, such as significant cracks and indications of pre-existing settlement. The preconstruction survey should be completed in the presence of the effected parties and consist of photographs and/or video tape, notes, and sketches of preexisting conditions. In addition, the City should consider vibration monitoring during pile driving to document vibration levels at nearby structures.

We recommend that the installation of driven piles be observed by a geotechnical or civil engineer with pile driving experience to evaluate the contractor's construction procedure, collect and interpret the installation data, monitor variations in subsurface conditions, and confirm the required penetration depths.

3.5.2 DEWITT DRIVEN GROUT PILES

DeWitt Driven Grout piles are a proprietary pile type developed by DeWitt Construction, Inc. The DeWitt Driven Grout piles are cast-in-place concrete piles that bear on a steel boot that is driven to the desired tip elevation. The steel boot is driven into the ground with a mandrel. The mandrel maintains the shaft diameter during pile driving. Once the tip elevation is reached, cement grout is placed under pressure from the tip as the mandrel is removed. The mandrel reportedly maintains the diameter of the pile and keeps soil from entering the grout column. Once the mandrel is completely removed, pile installation is completed by installing a rebar cage. DeWitt Driven Grout piles generally can achieve very similar axial capacities as compared to other driven pile types. For the HOCM project, DeWitt Driven Grout Piles have the advantage of generating minimal soil spoils. Like augercast piles, steel casing may be required to prevent the loss of grout into the fibrous wood waste that is present at the site.

According to representatives of DeWitt Construction, Inc. (Ullakko 2009), DeWitt Driven Grout Piles have been installed in the vicinity of the HOCM site. For the Columbia Place project, 18-inch diameter, DeWitt Driven Grout piles were installed to a depth of 95 ft BGS. These piles reportedly achieved an ultimate axial capacity of about 300 tons. DeWitt Driven Grout piles may also be utilized for the LOTT expansion immediately to the west of the HOCM property. Assuming similar soil conditions, DeWitt Construction, Inc. estimates that 16-inch diameter DeWitt Driven Grout Piles driven to about 90 ft BGS would have an allowable axial capacity (service loading) of about 100 tons. The allowable axial capacity includes a factor of safety of at least 2 on the ultimate pile capacity. The allowable axial capacity during seismic loading would be less. The allowable axial capacity of DeWitt Driven Grout Piles is confirmed with pile load tests.

3.6 SLAB-ON-GRADE FLOOR

As described in Section 3.2.6 of this report, if no ground improvement is implemented, between 3 and 12 inches of post-construction secondary settlement could occur during the design life of the structure. This would be gradual and manifest itself through shallow surface depressions, the loss of support and the potential cracking of slab-on-grade floors. In addition, up to 6 to 7 inches of liquefaction-induced settlement may occur across the site after the design earthquake. This could lead to an increased loss of support and further cracking/faulting of the slab-on-grade floor. Due to the variability of the near-surface site soils, differential settlement between two points over a horizontal distance of 50 ft could be as high as the total settlement. For these reasons, it may be desirable to utilize a pile supported grade-beam to support the structures’ first floor.

Assuming that the building pad was constructed in accordance with the recommendations provided in Section 3.2 of this report and the owner is willing to accept the risks (as described in the preceding paragraph of this report) associated with slab-on-grade construction at the site, the floors for the proposed structure could be constructed as slab-on-grade. Prior to placing the drainage material, the slab subgrade should be scarified as necessary, moisture-conditioned to near optimum moisture content, and compacted to at least 95 percent of the maximum dry density as determined by the ASTM D1557 test procedure. The resulting slab subgrade should be firm and unyielding. The prepared surface should be checked by a qualified geotechnical engineer for any loose and/or disturbed areas. If detected, these areas should be further compacted as recommended above, or overexcavated and replaced with structural fill in accordance with Sections 3.2.4 and 3.2.5 of this report. Recommendations for underslab drainage and for deflection analysis of the slab-on-grade floor are provided in Section 3.4.3 of this report.

3.7 MINOR STRUCTURE FOUNDATION SUPPORT

Based on the conditions observed in the borings completed for the project, between 8 and 10½ ft of wood debris is present at the site. The wood debris could lead to undesirable long-term settlement during the design life of the proposed improvements. We recommend that the design-build team consider utilizing helical anchors, such as those manufactured by AB Chance©, to transmit the loads through the wood debris into the underlying medium dense to dense recessional outwash deposits anticipated to be located between 15 to 18 ft BGS. Alternatively, minor structures could be supported on shallow foundations, provided the owner is willing to accept the risk of long-term total and differential settlement.

Liquefiable soil will be present below the base of the helical anchors or shallow foundation. It is anticipated that after the design earthquake, minor structures may settle and may need to be leveled/rebuilt.

3.7.1 HELICAL ANCHORS

Helical anchors are typically contractor designed using established installation torque to pile capacity relationships. For this project, the soil profile in the following table may be assumed for the purpose of designing helical anchors. For design, the groundwater level should be assumed to be about 4 ft below the existing ground surface. When considering the size of helical anchor for the project, the contractor should evaluate the ability of the anchor to penetrate through medium dense to dense sandy gravel deposits encountered in the upper recessional deposits. Downward axial design of helical piles should include a factor of safety of at least 2.0 on the calculated ultimate capacity. The top-most helical pile should extend at least three times the helix diameter upper recessional deposits. If helical anchors are installed to 12 ft BGS, wood debris is not encountered, and the required loads are achieved the helical anchors need not be installed any deeper.

Soil Type	Depth Range (ft BGS`)	Soil Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Medium Dense to Dense Sand	0 to 4	120	34	0
Loose Sand	4 to 7½	53(a)	31	0
Wood Debris	7½ to 18	18(a)	0	0
Upper Recessional Deposits	18 to 33	58(a)	34	0

Note:
(a) Buoyant unit weight

It should be recognized that the actual subsurface conditions at a specific location may be different from those described above (i.e. wood debris may not be present at all locations). As a result, the actual lengths of the helical piles may vary depending on the actual subsurface conditions encountered during construction. We, therefore, recommend structuring the contract documents to allow for pile length adjustments.

Liquefiable soil will be present below the base of the helical anchors. It is anticipated that after the design earthquake, minor structures supported by helical anchors may settle and the minor structures may need to be leveled/rebuilt.

It is anticipated that helical anchors will not be able to penetrate through large pieces of wood or other debris which may be present in the fill layer. If an obstruction is encountered, the helical anchor will likely need to be abandoned and relocated.

3.7.2 SHALLOW FOUNDATIONS

Shallow foundations could also be utilized to support minor structures. The allowable bearing pressure of shallow foundations is dependent on the specific location of the structure, the size of the foundation, the thickness of the wood waste, the distance from the base of the foundation to the top of the wood waste, and other factors. If shallow foundations will be utilized to support minor structures, we recommend that the design-build team complete site specific geotechnical investigations and additional engineering analysis.

3.8 BELOW-GRADE RETAINING WALLS

The magnitude of lateral earth pressures that develops against subsurface building and retaining walls will depend on the inclination of adjacent slopes, type of backfill, method of backfill placement, degree of backfill compaction, magnitude and location of adjacent surcharge loads, and the degree to which the wall can yield laterally during or after placement of backfill. When a subsurface wall is restrained against lateral movement or tilting, the soil pressure exerted is the at-rest soil pressure. Such wall restraint may develop if a rigid structural network is constructed prior to backfilling or if the wall is inherently stiff or otherwise restrained from rotation. In contrast, active soil pressure will be exerted on a subsurface wall if its top is allowed to rotate or yield a distance of at least 0.002 times its height during placement of backfill.

We recommend that yielding walls with level backfill under drained conditions be designed for an equivalent fluid density of 35 pcf for active soil conditions. If the wall is restrained from rotation during backfilling, an equivalent fluid density of 55 pcf should be used for design assuming level backfill and drained conditions. Design of any subsurface walls should include appropriate lateral earth pressures caused by any adjacent surcharge loads. For uniform surcharge pressures, uniformly distributed loads of 0.26 and 0.41 times the surcharge pressure should be added for yielding and non-yielding walls, respectively.

Dynamic lateral earth pressures due to a 1-in-2,475-year seismic event (2 percent probability of exceedance in a 50-year period) should be included in design of all retaining walls. A peak horizontal ground acceleration of 28 percent of gravity (S_{DS}/2.5) was assumed in computing dynamic lateral earth pressures. For retaining walls with level backfill able to translate laterally at least 2 inches during a seismic event should be designed to withstand a dynamic uniform lateral earth pressure of 6H psf (H is the vertical height of the wall). Walls unable to translate laterally (level backfill) should be designed to withstand a dynamic uniform lateral pressure of 22H psf. The resultant can be assumed to act at a point of 0.6H above the base of the wall. The dynamic lateral pressure should be added to the static lateral earth pressures.

To provide drainage behind the wall, backfill within 3 ft from the face of the wall should consist of free-draining, well-graded sand and gravel material with less than 5 percent fines and a maximum particle size of 2 inches. Because of its potential to run, we do not recommend the use of pea gravel as wall backfill. To avoid overstressing of the wall during placement and compaction, backfill placed within 3 ft of the wall should be compacted to between 90 and 92 percent of the maximum dry density determined by the ASTM D1557 test procedure. Additional recommendations for wall drainage are provided in Section 3.4.6 of this report.

Recommendations for foundation support of below-grade retaining walls are provided in Section 3.7 of this report. Recommendations for resistance to lateral loads are provided in Section 3.4.5 of this report.

4.0 USE OF THIS REPORT

Landau Associates prepared this geotechnical report for the exclusive use of the City of Olympia and the City of Olympia's HOCM Design-Build team for use by the Design-Build team to complete preliminary design of the new HOCM project in Olympia, Washington. This report is not intended to be sufficient for final design; however, the information in this report could be supplemented with additional explorations and engineering analyses that could form the basis of the design phase geotechnical recommendations. Any use of this report by others, or for purposes other than intended, is at the user's sole risk. Within the limitations of scope, schedule, and budget, our services have been conducted in accordance with generally accepted practices of the geotechnical engineering profession; no other warranty, express or implied, is made as the professional advice included in this report.

The findings, conclusions, and recommendations presented herein are based on our understanding of the project, our review of available geotechnical and geologic information in the project vicinity, and on subsurface conditions observed during explorations completed for this project. Some variations in subsurface soil and groundwater may not become evident until additional explorations are conducted, or even until construction. An appropriate contingency should be included in the final design phase and construction to accommodate potential variability of ground conditions. As the final loads, wall locations, and final grades are defined, we should be advised so that we can review our preliminary recommendation to see if they are consistent with project specific plans.

We appreciate the opportunity to provide geotechnical services on this project and look forward to assisting you during the design and construction phases. If you have any questions or comments regarding the information contained in this report, or if we may be of further service, please call.

LANDAU ASSOCIATES, INC.

BRIAN A. BENNETTS
43153
B A Bennetts
3/19/09

Brian A. Bennetts, P.E.
Senior Project Engineer

Edward J. Heavey, P.E.
Principal

BAB/EJH/jas

5.0 REFERENCES

AMEC Earth and Environmental, Inc. 2007. *Supplement No. 2: LOTT Administration Building, Thurston Avenue and Adams Street, Olympia, Washington.* Report prepared for the LOTT Alliance. April 5.

ASCE. 2005. *Minimum Design Loads for Buildings and Other Structures.* Publication 7-05. American Society of Civil Engineers.

Bray, J. D., and R.B. Sancio. 2006. *Assessment of the Liquefaction Susceptibility of Fine Grained Soils.* ASCE Journal of Geotechnical and Geoenvironmental Engineering. Vol. 132. No. 9. Pp. 1165-1177.

Finlayson, D. 2005. *Combined Bathymetry and Topography of the Puget Lowlands, Washington State* URL: http://www.ocean.washington.edu/data/pugetsound/.

GeoEngineers. 2007. *Draft Remedial Investigation/Feasibility Study and Conceptual Cleanup Action Plan, East Bay Redevelopment, Port of Olympia, Olympia, Washington, Ecology Facility/Site No. 5785176, VCP No. SW0827.* Report prepared for the Port of Olympia. December 20.

Idriss, I.M. and R.W. Boulanger. 2007. *SPT- and CPT-Based Relationships for the Residual Strength of Liquefied Soils.* Earthquake Geotechnical Engineering, 4th International Conference on Earthquake Engineering – Invited Lectures. K.D. Pitilakis, ed., Springer, Netherlands. Pp. 1 – 22.

Idriss, I.M., and R.W. Boulanger. 2008. *Soil Liquefaction During Earthquakes.* Earthquake Engineering Research Institute (EERI), MNO-12, 226 pp.

Ishihara, K. 1985. *Stability of Natural Deposits During Earthquakes.* Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering. Vol. 1. pp. 321-376.

International Code Council (ICC). 2006. *2006 International Building Code*.

Koelling, M. 2009 *Personal Communication between Mark Koelling (Hayward Baker Geotechnical Construction) and Edward Heavey and Brian Bennetts (Landau Associates) regarding Ground Improvement Techniques for HOCM Building.* January 22.

Palmer, S.P., T.J. Walsh, and W.J. Gerstel. 1999. *Geologic Folio of the Olympia-Lacy-Tumwater Urban Area, Washington: Liquefaction Susceptibility Map*. Washington State Department of Natural Resources. Geologic Map GM-47.

Thurston County. 2009. *Thurston Geodata Center*. URL: http://www.geodata.org/. Accessed January 12.

Ullakko, G. 2009. *Personal Communication between Garth Ullakko (DeWitt Construction, Inc.) and Brian Bennetts (Landau Associates) regarding DeWitt Driven Grout Piles.* March 11.

USGS. 2008. *Earthquake Ground Motion Parameters Software Program.* Version 5.0.9. U.S. Geological Survey.

Youd, T.L., I.M. Idriss, R.D. Andrus, I. Arango, G. Castro, J.T. Christian, R. Dobry, W.D.L. Finn, L.F. Harder, M.E. Hynes, K. Ishihara, J.P. Koester, S.S.C. Liao, W.F. Marcuson, G.R. Martin, J.K Mitchell, Y. Moriwaki, M.S. Power, P.K. Robertson, R.B. Seed, and K.H. Stockoe. 2001. *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*. Journal of Geotechnical and Geoenvironmental Engineering. 127(10): 817-833.

Youd, T.L., Hansen, C.M. and Bartlett, S.F. 2002. *Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement.* ASCE Journal of Geotechnical and Geoenvironmental Engineering. Vol. 128. No. 12. pp. 1007-1017.

Walsh, T.J., R. L. Logan, H. W. Schasse, and M. Polenz. 2003. *Geologic Map of the Tumwater 7.5-minute Quadrangle, Thurston County, Washington.* Washington Division of Geology and Earth Resources. Open File Report 2003-25.

Wang, S. and Reese, LC. 1998. *Design of Pile Foundation in Liquefied Soils.* Geotechnical Earthquake Engineering and Soil Dynamics III. Vol. 1. Proceedings sponsored by Geo-Institute of the American Society of Civil Engineers. Geotechnical Special Publication No. 75. August 3-6.

WSDOT. 2008a. *Geotechnical Design Manual.* Washington State Department of Transportation. Publication M46-03. November.

WSDOT. 2008b. *Standard Specifications for Road, Bridge, and Municipal Construction*. Washington State Department of Transportation.

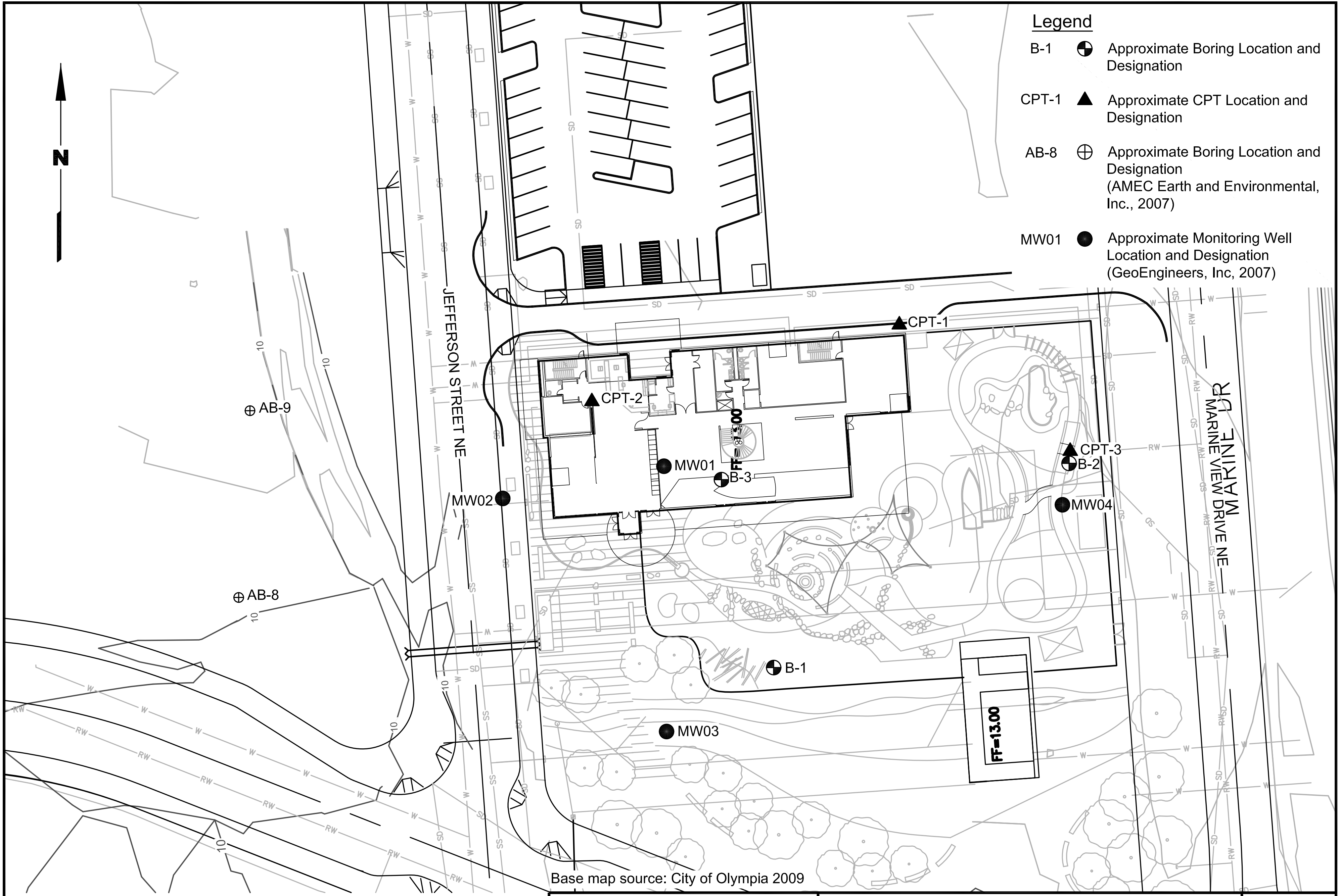
Zhang, G., Robertson, P.K. and R.W.I. Brachman. 2002. *Estimating Liquefaction-Induced Ground Settlements from CPT for Level Ground*. Canadian Geotechnical Journal. 1169-1180.

Data Source: ESRI 2006

LANDAU ASSOCIATES

Hands On Children's Museum Olympia, Washington	**Vicinity Map**	Figure 1

City of Olympia | Y:\258\017.010\T\Drawings\Fig 2.dwg (A) "Figure 2" 1/16/2009



Base map source: City of Olympia 2009

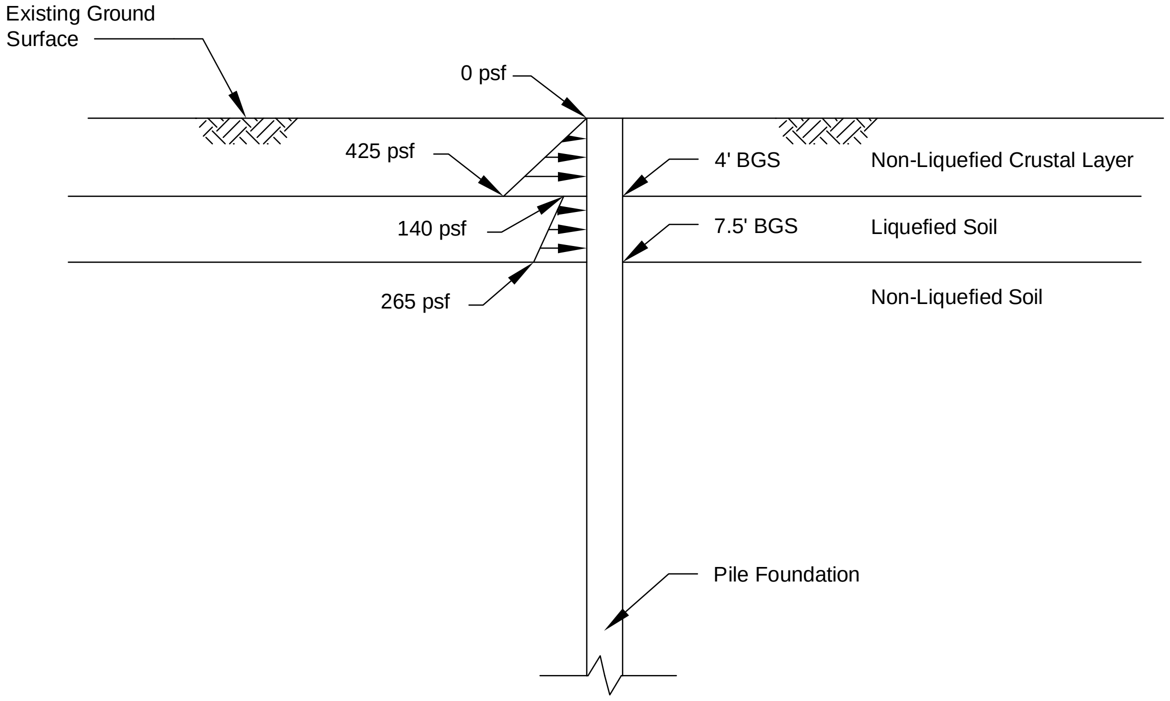
0 60 120
Scale in Feet

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Site and Exploration Plan

Figure
2



Not to Scale

Notes:

1. Lateral pressure exerted on pile during lateral spreading is based on soil conditions observed in boring B-3 and sounding CPT-2. If more liquefiable soil is present in eastern portion of building, lateral pressure due to lateral spreading could be greater than what is shown above.

2. The lateral pressure is assumed to act over one pile diameter.

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Lateral Pressure Exerted On Pile During Lateral Spreading

Figure
3

APPENDIX A

Field Explorations

APPENDIX A
FIELD EXPLORATIONS

Subsurface conditions at the Hands On Children's Museum Site were explored on January 8 and 9, 2009. The exploration program consisted of advancing three (3) exploratory borings (B-1 through B-3) and three (3) cone penetrometer test (CPT) soundings (CPT-1 through CPT-3) at the approximate locations illustrated on the Site and Exploration Plan (Figure 2). The explorations were located in the field with a hand-held GPS unit.

EXPLORATORY BORINGS

Borings B-1 and B-2 were completed to a depth of 19 ft BGS using truck-mounted drill rig and the hollow stem auger method. Boring B-3 was completed to a depth of 81½ ft BGS using truck-mounted drill rig and the mud-rotary drilling method. Holocene Drilling, Inc., of Edgewood, Washington advanced the borings under subcontract to Landau Associates.

The geotechnical exploratory program was coordinated and monitored by a Landau Associates geologist who also obtained representative soil samples, maintained a detailed record of the observed subsurface soil and groundwater conditions, and described the soil encountered by visual and textural examination. Each representative soil type observed in our exploratory borings was described using the soil classification system shown on Figure A-1, in general accordance with ASTM D2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure).* Logs of the exploratory borings are presented on Figures A-2 through A-4. These logs represent our interpretation of subsurface conditions identified during the field exploration program. The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific date and locations reported, and therefore, are not necessarily representative of other locations and times.

Disturbed samples of the soil encountered from the borings were obtained at frequent intervals using either a 1.5-inch inside-diameter (ID) Standard Penetration Test split-spoon sampler or a 2.43-inch ID Dames & Moore sampler. The sampler was driven up to 18 inches (or a portion thereof) into the undisturbed soil ahead of the auger bit with a 140-lb automatic hammer falling a distance of approximately 30 inches. The number of blows required to drive the sampler for the final 12 inches (or portion thereof) of soil penetration, is noted on the boring logs adjacent to the appropriate sample notation.

Upon completion of drilling and sampling, the boreholes were abandoned in general accordance with the requirements of Washington Administrative Code (WAC) 173-160.

CONE PENETROMETER TEST PROGRAM

The field exploration program included a CPT program to provide an overview of the subsurface soil conditions at the project site. The CPT program was completed on January 8, 2009. The CPT soundings were advanced to depths ranging from between 22½ to 120 ft BGS using truck-mounted CPT equipment. In Situ Engineering, Inc. of Snohomish, Washington completed the CPT soundings under subcontract to Landau Associates.

At each CPT sound location, a four-channel electronic cone was pushed at a rate of about 1 to 2 cm/sec. The cone was used to simultaneously record tip resistance, sleeve friction, pore pressure, and inclination every 5 cm. Data was transmitted electronically from the cone to a receiver located at the ground surface. Upon completion of testing, the CPT soundings were abandoned in general accordance with the requirements of WAC 173-160.

In Situ Engineering, Inc. reduced the collected CPT data and plotted tip resistance and friction ration (sleeve friction divided by tip resistance) as a function of sounding depth. They then used published correlations (Robertson and Campanella 1983) to estimate soil behavior types and equivalent Standard Penetration Test (SPT) values at each interval where data was recorded (i.e., every 5 cm). In Situ Engineering's detailed interpretation of soil behavior types and equivalent SPT values are presented on Figures A-5 through A-7.

It should be noted that the published correlations used by In Situ Engineering, Inc. to develop their detailed logs are generally regarded as predictions of soil behavior rather than actual soil type. Factors such as changes in stress history, sensitivity, stiffness, and void ratio will influence the soil classifications when using the published correlations. Accordingly, actual soil types at some or all of the CPT locations may vary from the soil types shown on In Situ Engineering's interpreted logs. Furthermore, the soil and groundwater conditions depicted are only for the specific dates and locations reported and, therefore, are not necessarily representative of other locations and times.

During the advancement of CPT sounding CPT-3, In Situ Engineering performed seismic testing. The shear wave velocity tests were performed at 1 meter intervals [(3.28 feet (ft)] along the entire length of the CPT soundings. The data from the shear wave velocity tests are presented on Figure A-8.

REFERENCE

Robertson, P.K. and R. G. Campanella. 1983. *Interpretation of Cone Penetration Tests, Part I: Sand.* Canadian Geotechnical Journal. Volume 20. pp. 718-733.

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL(1)	TYPICAL DESCRIPTIONS (2)(3)
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)	▨	**GW**	Well-graded gravel; gravel/sand mixture(s); little or no fines
			▨	**GP**	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)	▨	**GM**	Silty gravel; gravel/sand/silt mixture(s)
			▨	**GC**	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)	▨	**SW**	Well-graded sand; gravelly sand; little or no fines
			▨	**SP**	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)	▨	**SM**	Silty sand; sand/silt mixture(s)
			▨	**SC**	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)		▨	**ML**	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
			▨	**CL**	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
			▨	**OL**	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)		▨	**MH**	Inorganic silt; micaceous or diatomaceous fine sand
			▨	**CH**	Inorganic clay of high plasticity; fat clay
			▨	**OH**	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL		▨	**PT**	Peat; humus; swamp soil with high organic content

OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT	■	**AC** or **PC**	Asphalt concrete pavement or Portland cement pavement
ROCK	▨	**RK**	Rock (See Rock Classification)
WOOD	▨	**WD**	Wood, lumber, wood chips
DEBRIS	▨	**DB**	Construction debris, garbage

NOTES:

1. USCS letter symbols correspond to symbols used by the Unified Soil Classification System and ASTM classification methods. Dual letter symbols (e.g., SP-SM for sand or gravel) indicate soil with an estimated 5-15% fines. Multiple letter symbols (e.g., ML/CL) indicate borderline or multiple soil classifications.
2. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test Method for Classification of Soils for Engineering Purposes*, as outlined in ASTM D 2487.
3. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

Primary Constituent: > 50% - "GRAVEL," "SAND," "SILT," "CLAY," etc.
Secondary Constituents: > 30% and ≤ 50% - "very gravelly," "very sandy," "very silty," etc.
> 15% and ≤ 30% - "gravelly," "sandy," "silty," etc.
Additional Constituents: > 5% and ≤ 15% - "with gravel," "with sand," "with silt," etc.
≤ 5% - "trace gravel," "trace sand," "trace silt," etc., or not noted.

2/3/09 Y:\258\017.010\T\258017.GPJ SCS WSDOT 1OF2

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Soil Classification System and Key

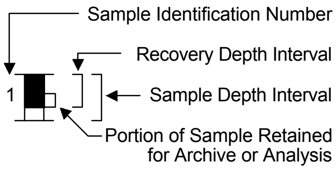
Figure
A-1
(1 of 2)

Drilling and Sampling Key

SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Single-Tube Core Barrel
f	Double-Tube Core Barrel
g	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Rotosonic
5	Air Rotary (Rock)
6	Wash Rotary (Rock)
7	Other - See text if applicable

SAMPLE NUMBER & INTERVAL



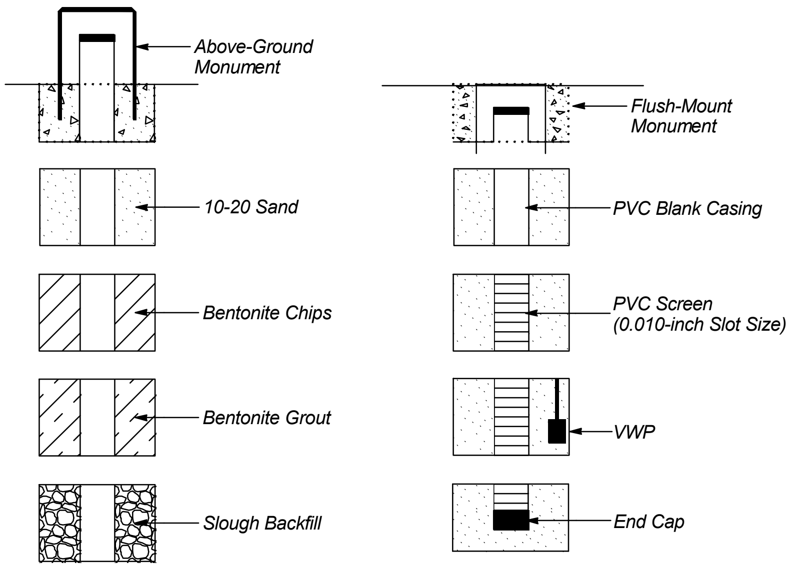
Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
VST	Vane Shear Test
GT	Other Geotechnical Testing
CA	Chemical Analysis

Groundwater

▽	Approximate water elevation at time of drilling (ATD).
▼	Approximate water elevation at other time(s). When multiple water levels are obtained other than ATD, only a representative range is shown. See text for additional information.
Note:	Groundwater levels can fluctuate due to precipitation, seasonal conditions, and other factors.

Well Log Graphics



2/3/09 Y:\258\017.010\T\258017.GPJ SCS WSDOT 2OF2

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Soil Classification System and Key

Figure
A-1
(2 of 2)

B-1

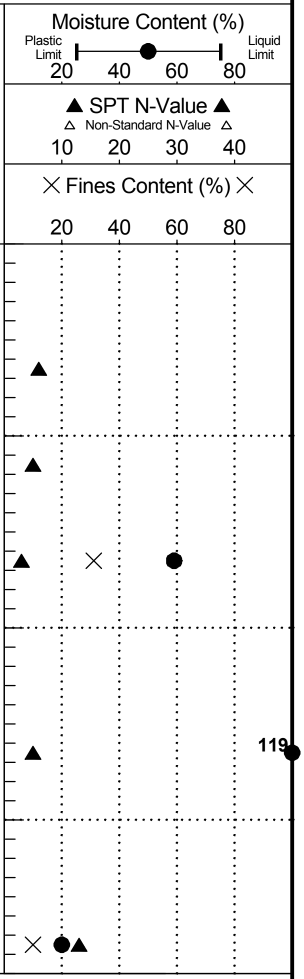
LAI Project No: 258017.010

SAMPLE DATA

SOIL PROFILE

Drilling Method: Hollow-Stem Auger
Ground Elevation (ft): Not Measured
Drilled By: Holocene Drilling Inc.
Logged By: KMH Date: 01/09/09

Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Description	Groundwater
					SP	Brown, fine to coarse SAND with gravel and trace shells (loose, moist) **(FILL)**	
	S-1	b2	6				
	S-2	b2	5				▽ ATD
					SM/ WD	Gray, very silty, fine to medium SAND with gravel and abundant wood debris (very loose, wet)	
	S-3	b2	3	-200=31 W = 59 GS			
					WD	Wood debris	
	S-4	b2	5	W = 119			
					SW- SM	Dark gray, gravelly, fine to coarse SAND with silt (medium dense, wet) **(RECESSIONAL OUTWASH)**	
	S-5	b2	13	-200=10 W = 20 GS			



Depth (ft): 0, 5, 10, 15, 20, 25, 30, 35

Boring Completed 01/09/09. Total Depth of Boring = 19.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Log of Boring B-1

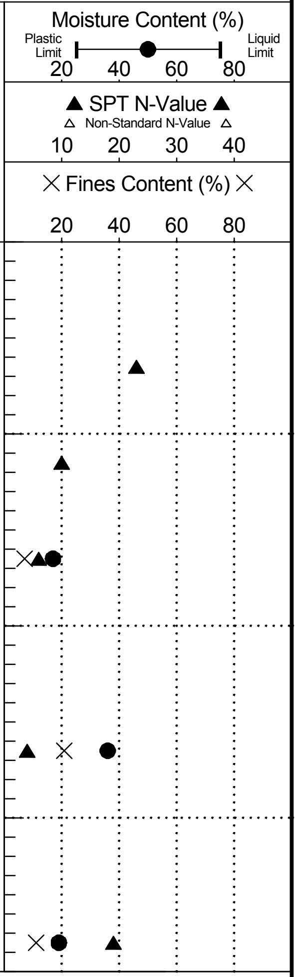
Figure
A-2

B-2

LAI Project No: 258017.010

Drilling Method: Hollow-Stem Auger
Ground Elevation (ft): Not Measured
Drilled By: Holocene Drilling Inc.
Logged By: KMH Date: 01/09/09

SAMPLE DATA						SOIL PROFILE			
Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol		Groundwater
0		S-1	b2	23			SP	Brown, very gravelly, fine to coarse SAND (medium dense, damp) **(FILL)**	
5		S-2	b2	10			GP	Brown, sandy GRAVEL (medium dense, moist)	∇ ATD
		S-3	b2	6	-200=7 W = 17 GS		SP-SM	Dark gray, gravelly, fine to medium SAND with silt (loose, wet)	
10		S-4	b2	4	-200=21 W = 36 GS		SM/WD	Dark gray, silty, gravelly, fine to coarse SAND with abundant wood (very loose, wet)	
15		S-5	b2	19	-200=11 W = 19 GS		SP-SM	Dark gray, gravelly, fine to coarse SAND with silt and wood (medium dense, wet) **(RECESSIONAL OUTWASH)**	



Boring Completed 01/09/09. Total Depth of Boring = 19.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Log of Boring B-2

Figure
A-3

B-3

LAI Project No: 258017.010

SAMPLE DATA / SOIL PROFILE

Drilling Method: Mud Rotary

Ground Elevation (ft): Not Measured

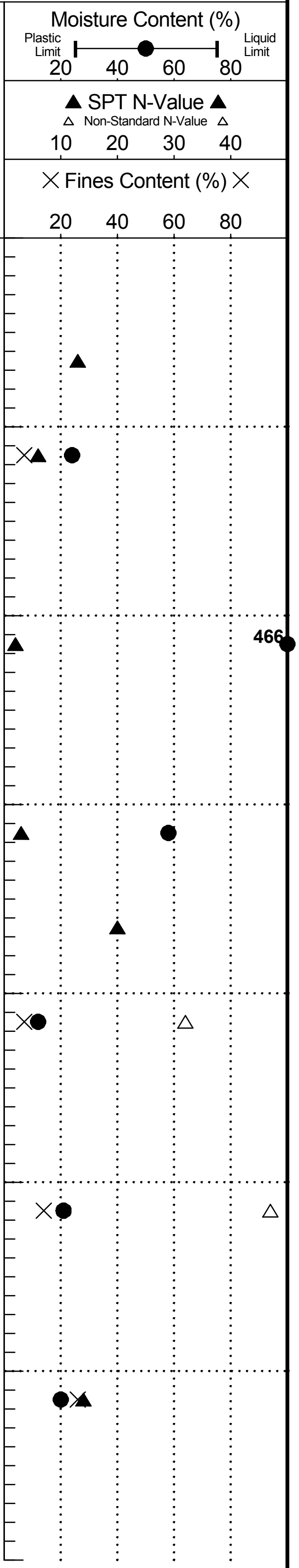
Drilled By: Holocene Drilling Inc.

Logged By: KMH Date: 01/08/09

Groundwater: Groundwater Not Encountered

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
0–5		S-1	b2	13	
5–10		S-2	b2	6	-200=7 W = 24 GS
10–15		S-3	b2	2	W = 466
15–20		S-4	b2	3	W = 58
		S-5	b2	20	
20–25		S-6	a2	32	-200=7 W = 12 GS
25–30		S-7	a2	47	-200=14 W = 21 GS
30–35		S-8	b2	14	-200=26 W = 20 GS

USCS Symbol	Description
GP	Brown GRAVEL with sand (medium dense, wet) **(FILL)**
SP-SM	Gray, fine to medium SAND with silt (loose, wet)
WD	Brown WOOD (very soft, wet)
GP-GM	Gray, very sandy GRAVEL with silt (medium dense, wet) **(RECESSIONAL OUTWASH)**
SP-SM	Gray, fine to medium SAND with silt and gravel (medium dense, wet)
SM	Gray, silty, fine to medium SAND with trace gravel (medium dense, wet)
CL	Gray, silty CLAY (medium stiff, wet)



Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

258017.01 2/3/09 Y:\258017.010\T\258017.GPJ SOIL BORING LOG WITH GRAPH

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Log of Boring B-3

Figure A-4 (1 of 3)

B-3

LAI Project No: 258017.010

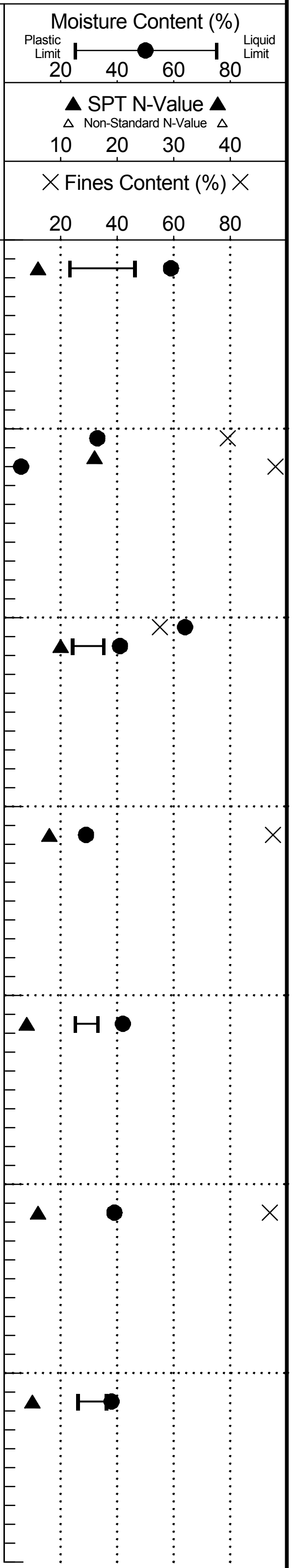
Drilling Method: Mud Rotary

Ground Elevation (ft): Not Measured

Drilled By: Holocene Drilling Inc.

Logged By: KMH Date: 01/08/09

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	SOIL PROFILE	Groundwater
35		S-9	b2	6	W = 59 AL	CL	Gray, silty CLAY (medium stiff, wet)	
						ML	Gray, sandy SILT; non-plastic (very stiff, wet)	Groundwater Not Encountered
40		S-10A S-10B	b2	16	W = 33 -200 = 79 W = 6 -200 = 96		SILT interbed	
45		S-11A S-11B	b2	10	W = 64 -200 = 55 W = 41 AL		becomes very sandy	
						ML	Gray SILT with sand to trace sand (medium stiff to stiff, wet)	
50		S-12	b2	8	W = 29 -200 = 95			
55		S-13	b2	4	W = 42 AL			
60		S-14	b2	6	W = 39 -200 = 94			
65		S-15	b2	5	W = 38 AL			
70								



Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

258017.01 2/3/09 Y:\258017.010\T\258017.GPJ SOIL BORING LOG WITH GRAPH

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Log of Boring B-3

Figure

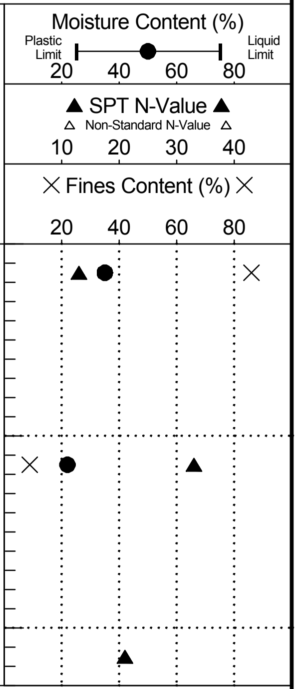
A-4
(2 of 3)

B-3

LAI Project No: 258017.010

Drilling Method: Mud Rotary
Ground Elevation (ft): Not Measured
Drilled By: Holocene Drilling Inc.
Logged By: KMH Date: 01/08/09

SAMPLE DATA						SOIL PROFILE			
Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol		Groundwater
70		S-16	b2	13	W = 35 -200 = 86		ML	Gray SILT with sand to trace sand (medium stiff to stiff, wet)	
							SP-SM	Blue-gray, fine to medium SAND with silt (dense, wet)	Groundwater Not Encountered
75		S-17	b2	33	W = 22 -200 = 9				
80		S-18	b2	21				becomes medium dense	



Boring Completed 01/08/09. Total Depth of Boring = 81.5 ft.

85

90

95

100

105

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

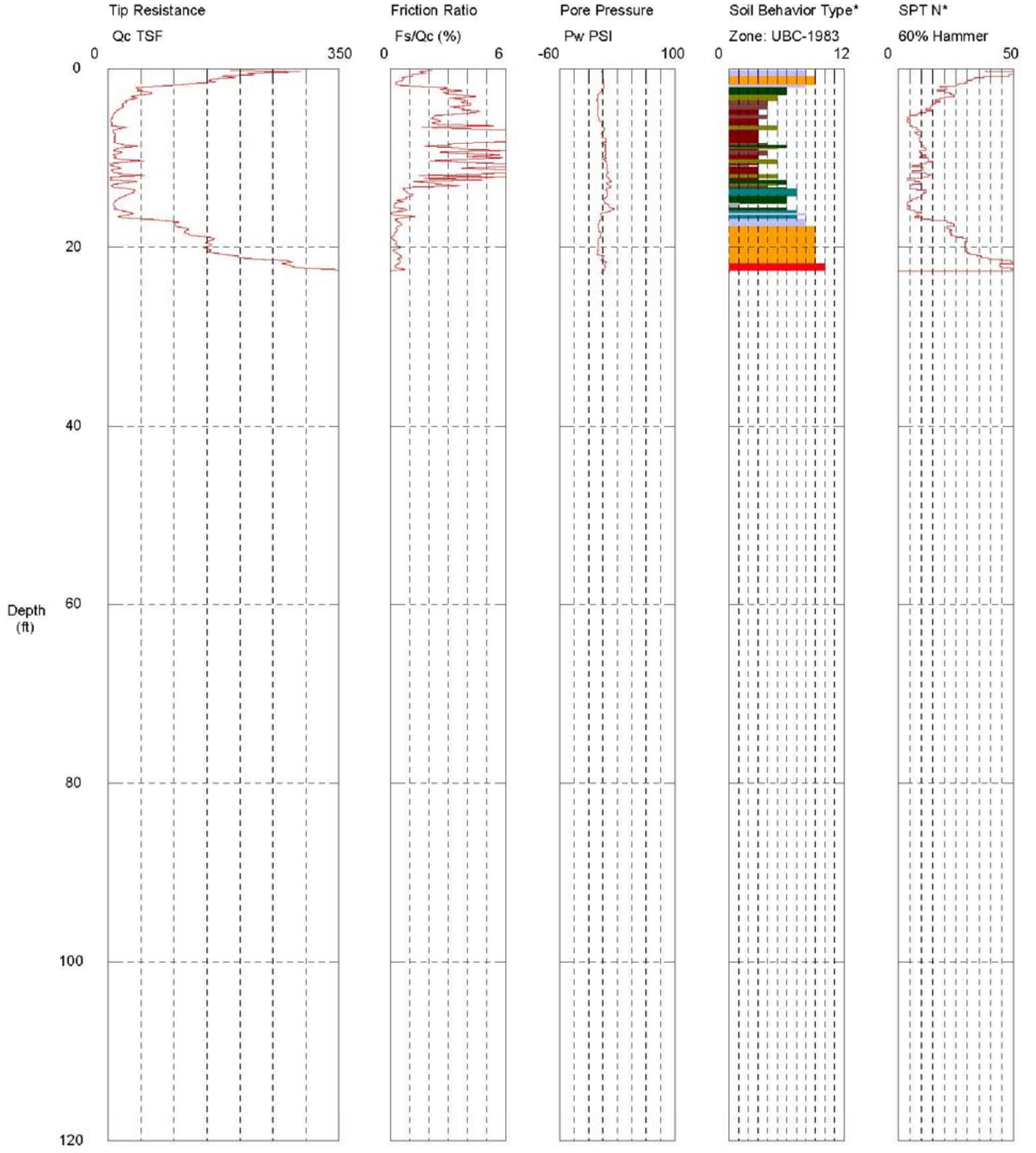
Log of Boring B-3

Figure
A-4
(3 of 3)

Landau Assoiciates

Operator: Brown
Sounding: CPT-01
Cone Used: DSG1079

CPT Date/Time: 1/8/2009 11:02:02 AM
Location: Olympia Hands On Children's Museum
Job Number: 258017.010.011



Maximum Depth = 22.80 feet

Depth Increment = 0.164 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

Pushed through cored hole on east side of slab. Refused on gravels.

In Situ Engineering

*Soil behavior type and SPT based on data from UBC-1983

Hands On Children's Museum
Olympia, Washington

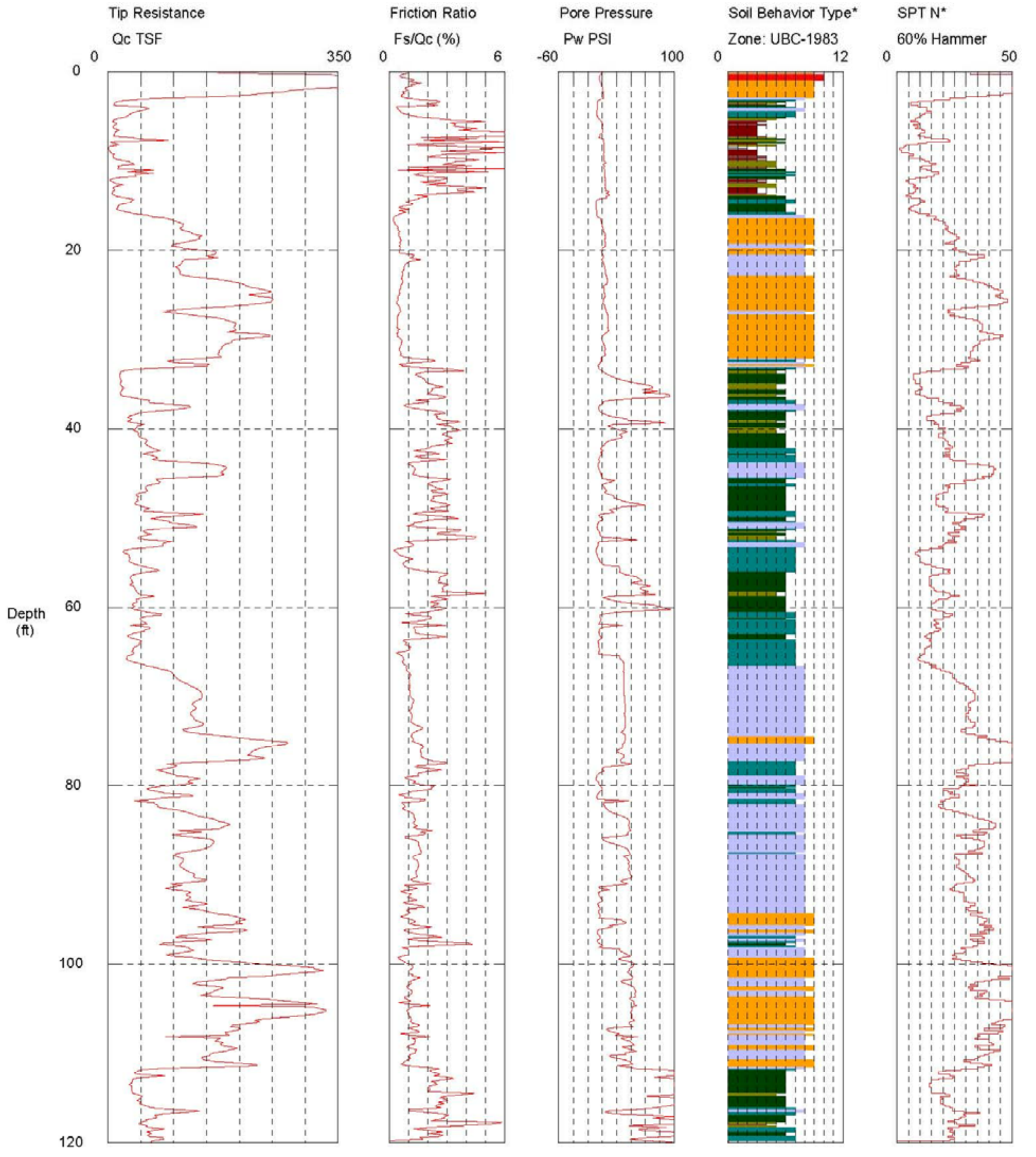
Log of Sounding
CPT-01

Figure
A-5

Landau Associates

Operator: Brown
Sounding: CPT-02
Cone Used: DSG1079

CPT Date/Time: 1/8/2009 11:44:44 AM
Location: Olympia Hands On Children's Museum
Job Number: 258017.010.011



Maximum Depth = 119.91 feet

Depth Increment = 0.164 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

Pushed through cored hole on west side of slab.

In Situ Engineering

*Soil behavior type and SPT based on data from UBC-1983

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Log of Sounding
CPT-02

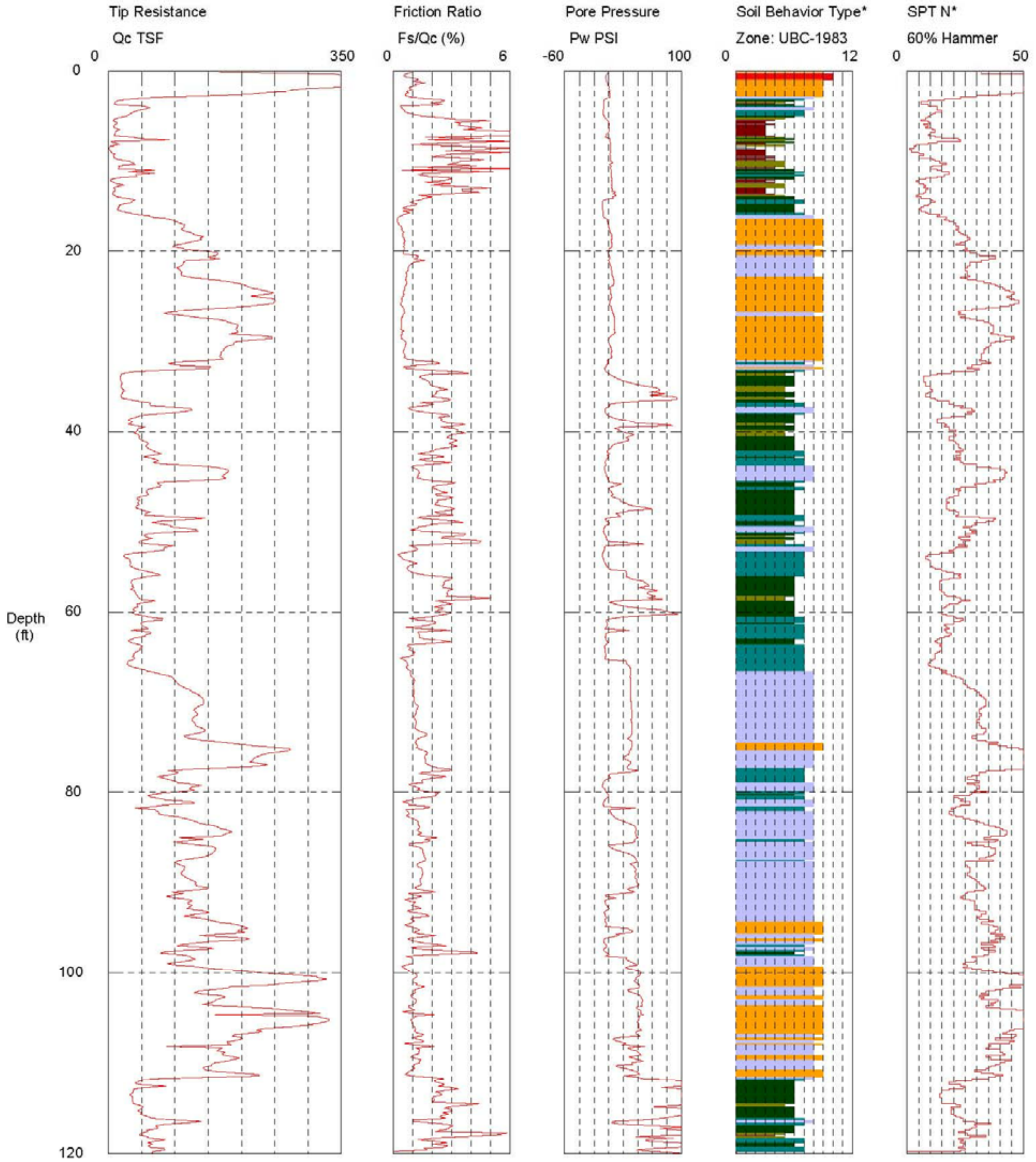
Figure
A-6

City of Olympia | Y:\258\017.010\T\Figs A5-A8.dwg (A) "Figure A-6" 1/23/2009

Landau Associates

Operator: Brown
Sounding: CPT-02
Cone Used: DSG1079

CPT Date/Time: 1/8/2009 11:44:44 AM
Location: Olympia Hands On Children's Museum
Job Number: 258017.010.011



Maximum Depth = 119.91 feet

Depth Increment = 0.164 feet

- 1 sensitive fine grained
- 2 organic material
- 3 clay
- 4 silty clay to clay
- 5 clayey silt to silty clay
- 6 sandy silt to clayey silt
- 7 silty sand to sandy silt
- 8 sand to silty sand
- 9 sand
- 10 gravelly sand to sand
- 11 very stiff fine grained (*)
- 12 sand to clayey sand (*)

Pushed through cored hole on west side of slab.

In Situ Engineering

*Soil behavior type and SPT based on data from UBC-1983

City of Olympia | Y:\258017.010\T\Figs A5-A8.dwg (A) "Figure A-7" 1/23/2009

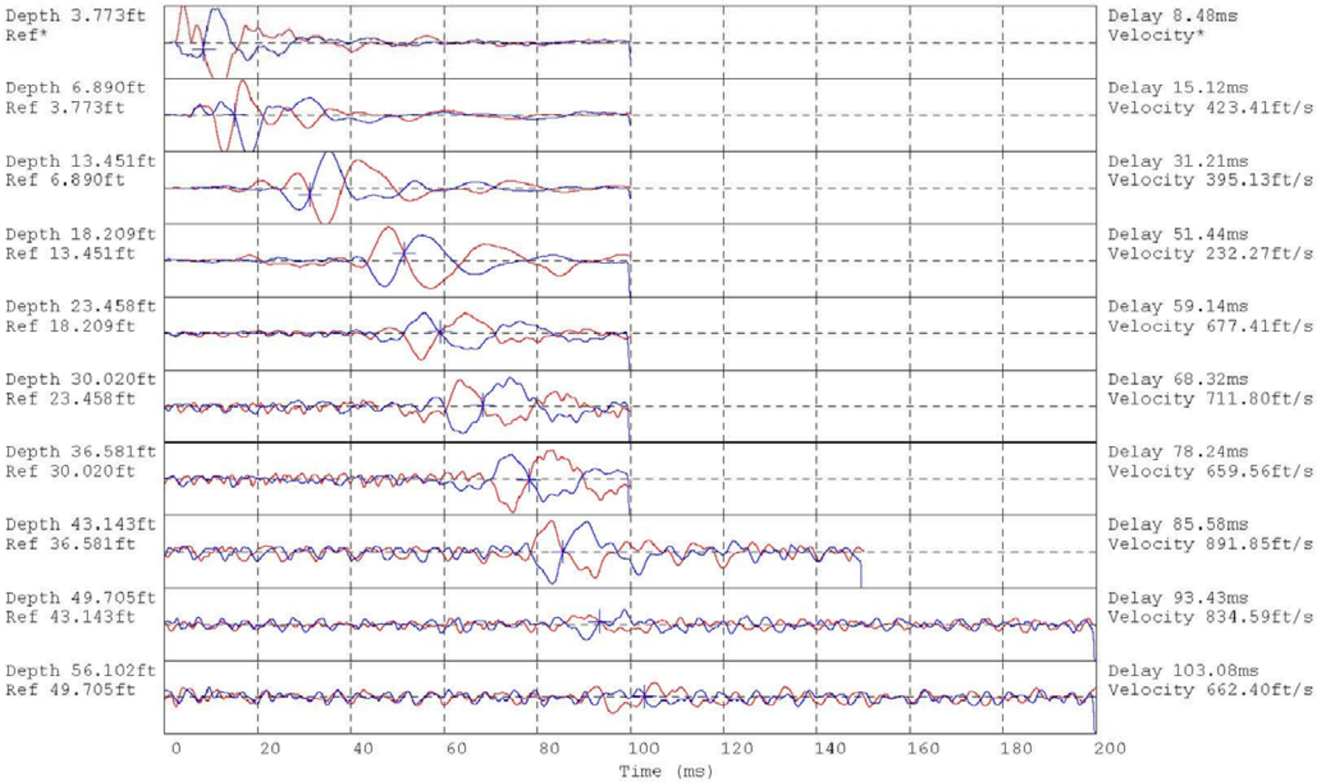
LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Log of Sounding
CPT-03

Figure
A-7

Shear Wave Velocity Plots CPT-03
Olympia Hands on Children's Museum



Hammer to Rod String Distance 0.75 (m)
* = Not Determined

City of Olympia | Y:\258\017.010\T\Figs A5-A8.dwg (A) "Figure A-8" 1/23/2009

In Situ Engineering

LANDAU ASSOCIATES

Hands On Children's Museum Olympia, Washington	**Shear Wave Velocity** **CPT-03**	Figure A-8

APPENDIX B

Laboratory Testing

APPENDIX B
LABORATORY TESTING

Natural moisture content determinations, fines content determinations, grain size analyses, and Atterberg limit determinations tests were performed on selected samples to aid in soil classification. Laboratory testing was performed in general accordance with the ASTM standard test procedures, which are described below. The samples were checked against the field log descriptions, which were updated where appropriate in general accordance with ASTM D2487, *Standard Test Method for Classification of Soils for Engineering Purposes*.

NATURAL MOISTURE CONTENT

Natural moisture content determinations were performed on selected soil samples recovered from the borings in general accordance with ASTM D2216. The natural moisture content is shown as W=xx (percent of dry weight) at the respective sample depth in the column labeled “Test Data” on the summary boring logs in Appendix A.

SIEVE ANALYSIS

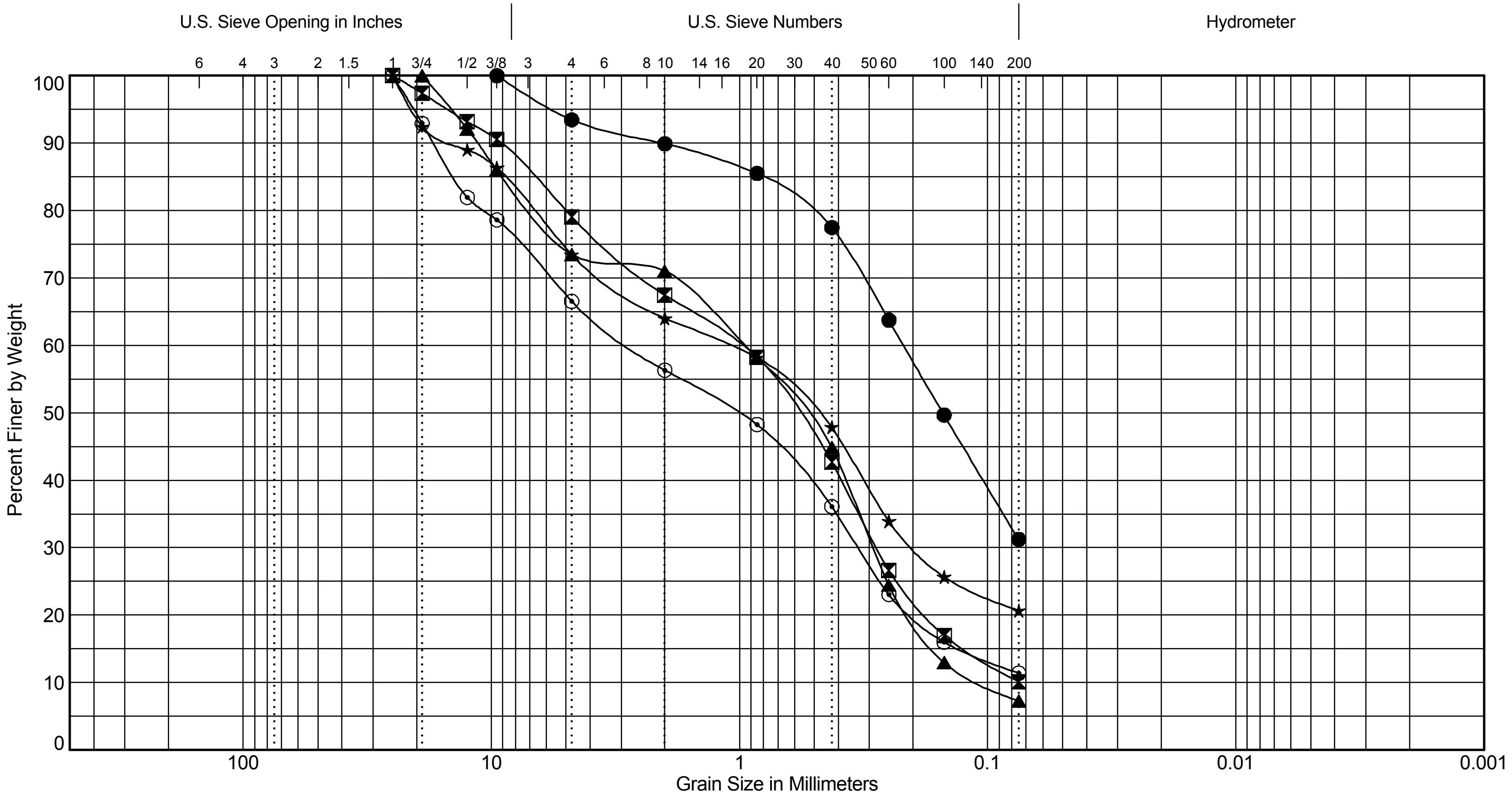
Sieve analyses were performed on representative soil samples obtained from the borings in accordance with ASTM D422, to provide an indication of the grain size distribution. Samples selected for sieve analysis are designated with a “GS” in the column labeled “Test Data” on the summary boring logs in Appendix A. The results of the sieve analyses are presented in the form of grain size distribution curves on Figures B-1 through B-2 in this appendix.

FINES CONTENT

The fines content (the percentage of material passing the U.S. Standard No. 200 sieve) of selected soil samples obtained from our exploratory borings were determined in general accordance with ASTM D1140 test procedures. The fines content is shown as -200=xx (percent of dry weight) at the respective sample depth in the column labeled “Test Data” on the summary boring logs in Appendix A.

ATTERBERG LIMITS

Atterberg limit determinations were performed on representative soil samples obtained from the borings in general accordance with ASTM D4318, to determine the liquid limit (LL), plastic limit (PL), and plasticity index (PI). Samples on which Atterberg limit determinations were completed are designated by “AL” in the column labeled “Test Data” on the summary logs. The results of the Atterberg limit determinations are presented on Figure B-3 in this appendix.



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	S-3	7.5	59	Very silty, fine to medium SAND with gravel	SM
⧗	B-1	S-5	17.5	20	Gravelly, fine to coarse SAND with silt	SW-SM
▲	B-2	S-3	7.5	17	Gravelly, fine to medium SAND with silt	SP-SM
★	B-2	S-4	12.5	36	Silty, gravelly, fine to coarse SAND	SM
⊙	B-2	S-5	17.5	19	Very gravelly, fine to coarse SAND with silt	SP-SM

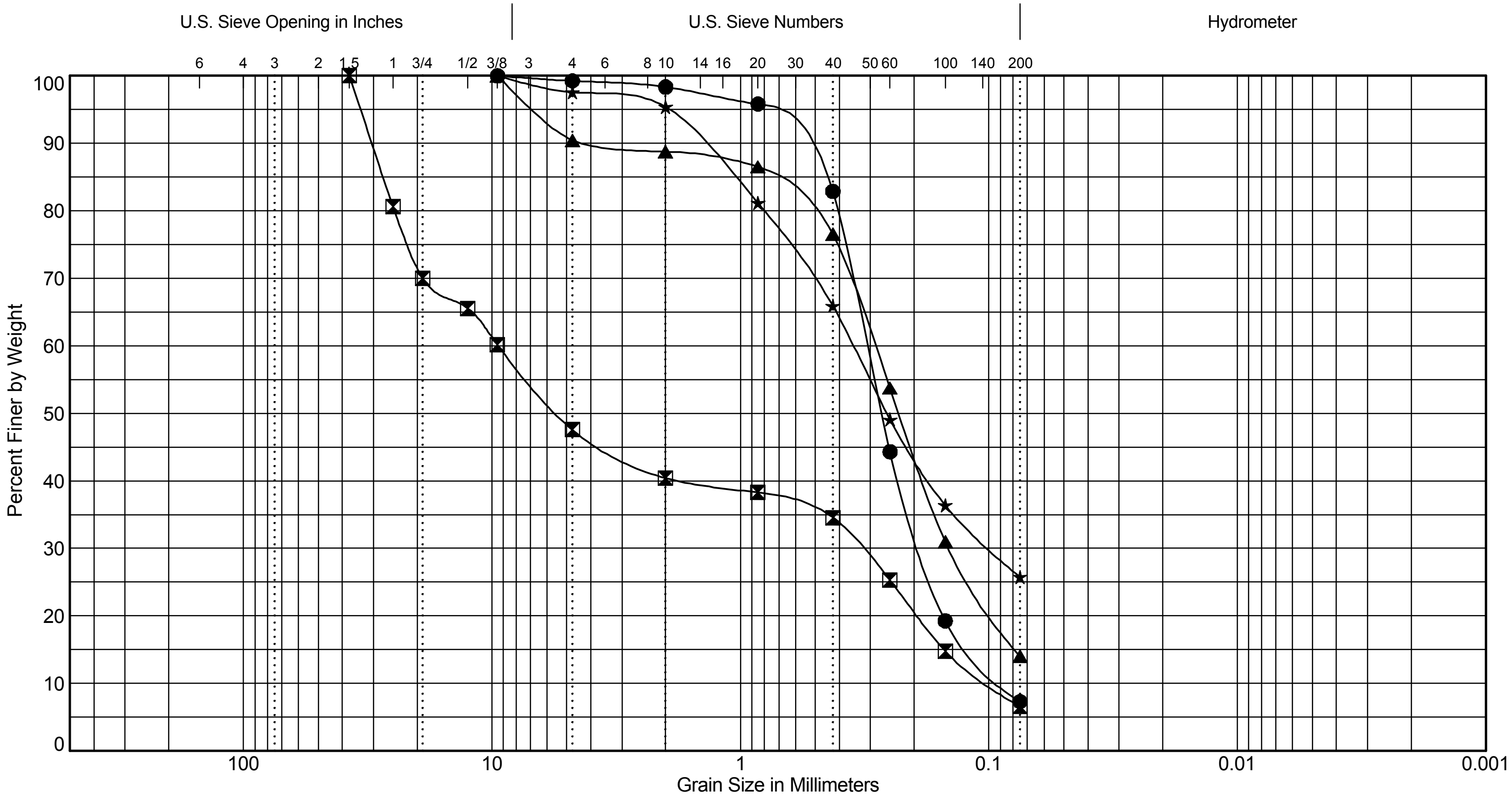
LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Grain Size Distribution

Figure
B-1

258017.01 2/3/09 Y:\258\017.010\T\258017.GPJ GRAIN SIZE FIGURE



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

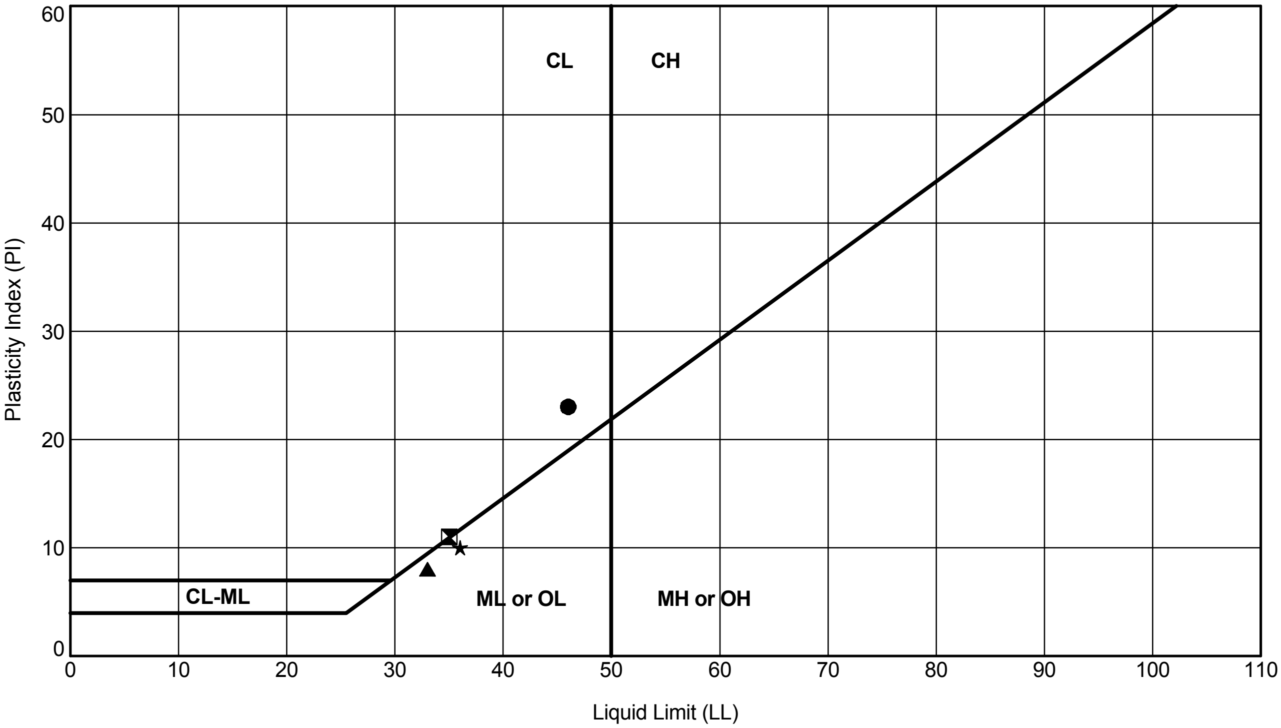
Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-3	S-2	5.0	24	Fine to medium SAND with silt	SP-SM
⧗	B-3	S-6	20.0	12	Very sandy GRAVEL with silt	GP-GM
▲	B-3	S-7	25.0	21	Fine to medium SAND with silt and gravel	SP-SM
★	B-3	S-8	30.0	20	Silty, fine to medium SAND	SM

LANDAU ASSOCIATES

Hands On Children's Museum
Olympia, Washington

Grain Size Distribution

Figure
B-2



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-3	S-9	35.0	46	23	23	59	Silty CLAY	CL
☒	B-3	S-11B	45.5	35	24	11	41	SILT with sand to trace sand	ML
▲	B-3	S-13	55.0	33	25	8	42	SILT with sand to trace sand	ML
★	B-3	S-15	65.0	36	26	10	38	SILT with sand to trace sand	ML

ASTM D 4318 Test Method

258017.01 2/3/09 Y:\258017.010\T\258017.GPJ ATTERBERG LIMITS FIGURE

Landau Associates

Hands On Children's Museum
Olympia, Washington

Plasticity Chart

Figure
B-3

APPENDIX C

Logs By Others

SUPPLEMENT NO. 2:
LOTT ADMINISTRATION BUILDING
THURSTON AVENUE AND ADAMS STREET
OLYMPIA, WASHINGTON

Submitted to:

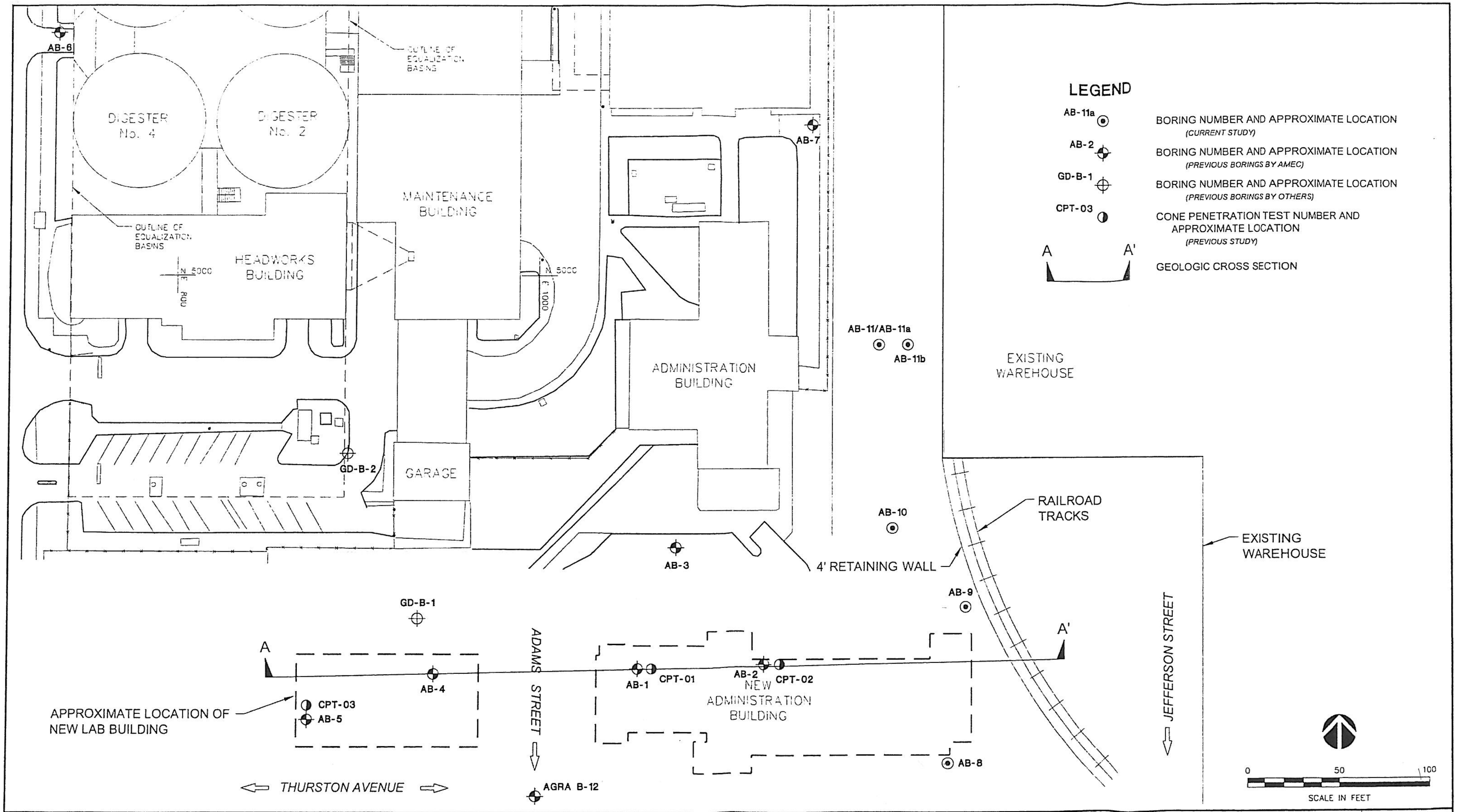
LOTT Alliance
111 Market St N.E., Suite 250
Olympia, Washington 98501

Submitted by:

AMEC Earth & Environmental, Inc.
11335 N.E. 122nd Way, Suite 100
Kirkland, Washington 98034-6918

April 5, 2007

7-917-15513-B



NOTE: DRAWING THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE AMEC EARTH & ENVIRONMENTAL REPORT No. 7-917-15513-B DATED 3/20/07.

CLIENT LOGO

CLIENT:
LOTT ALLIANCE

AMEC Earth & Environmental
11335 N.E. 122nd Way, Suite 100
Kirkland, WA, U.S.A. 98034-6918

amec

DWN BY:	JRS
CHK'D BY.	WJL
DATUM.	NONE
PROJECTION.	NONE
SCALE	1"=50'

PROJECT
LOTT ADMINISTRATION BUILDING

TITLE
SITE AND EXPLORATION PLAN

Olympia, Washington

DATE.	MARCH 2007
PROJECT NO:	7-917-15513B
REV NO.:	A
FIGURE No	FIGURE 2

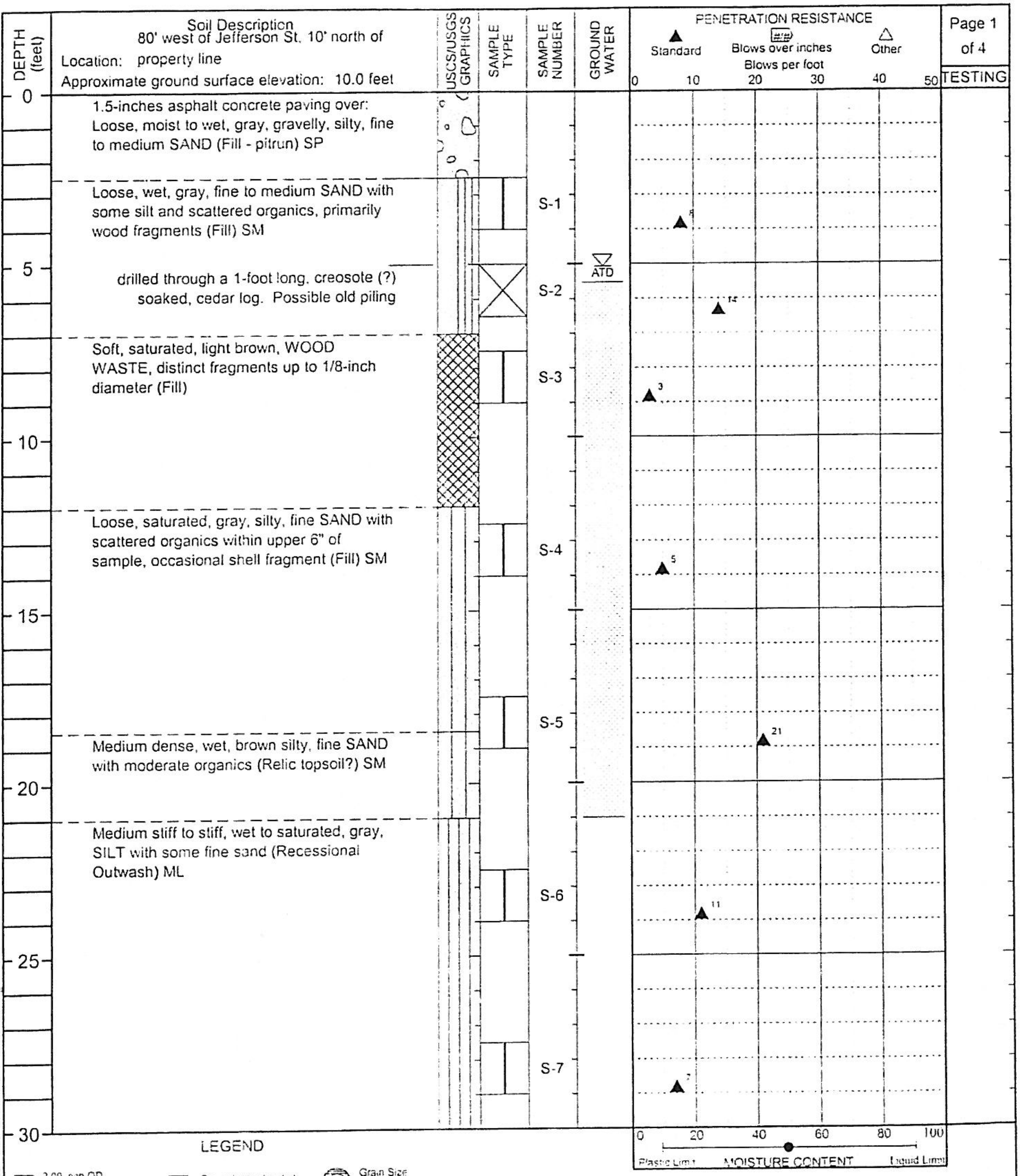
G: 91 15000 15513-B – Lott Admin Bldg 15513-B-02.dwg – Layout1 – Apr. 05, 2007 10:58am – jeffrey sanders

PROJECT: *LOTT Administration Building* JOB No. *7-917-15513-B* BORING No. *AB-8*

Location: 80' west of Jefferson St. 10' north of property line

Approximate ground surface elevation: 10.0 feet

Page 1 of 4



Soil Description	SAMPLE NUMBER	Standard (Blows per foot)
1.5-inches asphalt concrete paving over: Loose, moist to wet, gray, gravelly, silty, fine to medium SAND (Fill - pitrun) SP		
Loose, wet, gray, fine to medium SAND with some silt and scattered organics, primarily wood fragments (Fill) SM	S-1	8
drilled through a 1-foot long, creosote (?) soaked, cedar log. Possible old piling	S-2	14
Soft, saturated, light brown, WOOD WASTE, distinct fragments up to 1/8-inch diameter (Fill)	S-3	3
Loose, saturated, gray, silty, fine SAND with scattered organics within upper 6" of sample, occasional shell fragment (Fill) SM	S-4	5
	S-5	21
Medium dense, wet, brown silty, fine SAND with moderate organics (Relic topsoil?) SM		
Medium stiff to stiff, wet to saturated, gray, SILT with some fine sand (Recessional Outwash) ML	S-6	11
	S-7	7

LEGEND

- 2.00-inch OD split-spoon sampler
- No Recovery
- ATD Groundwater level at time of drilling
- Grain Size Analysis (% fines shown)

amec

11335 N.E. 122nd Way Suite 100
Kirkland, Washington 98034-6913

Drilling Method: HSA Hammer Type: Automatic Date drilled: *March 05, 2007* Logged By: *WJL*

Drilled by: *Enviromental Drilling*

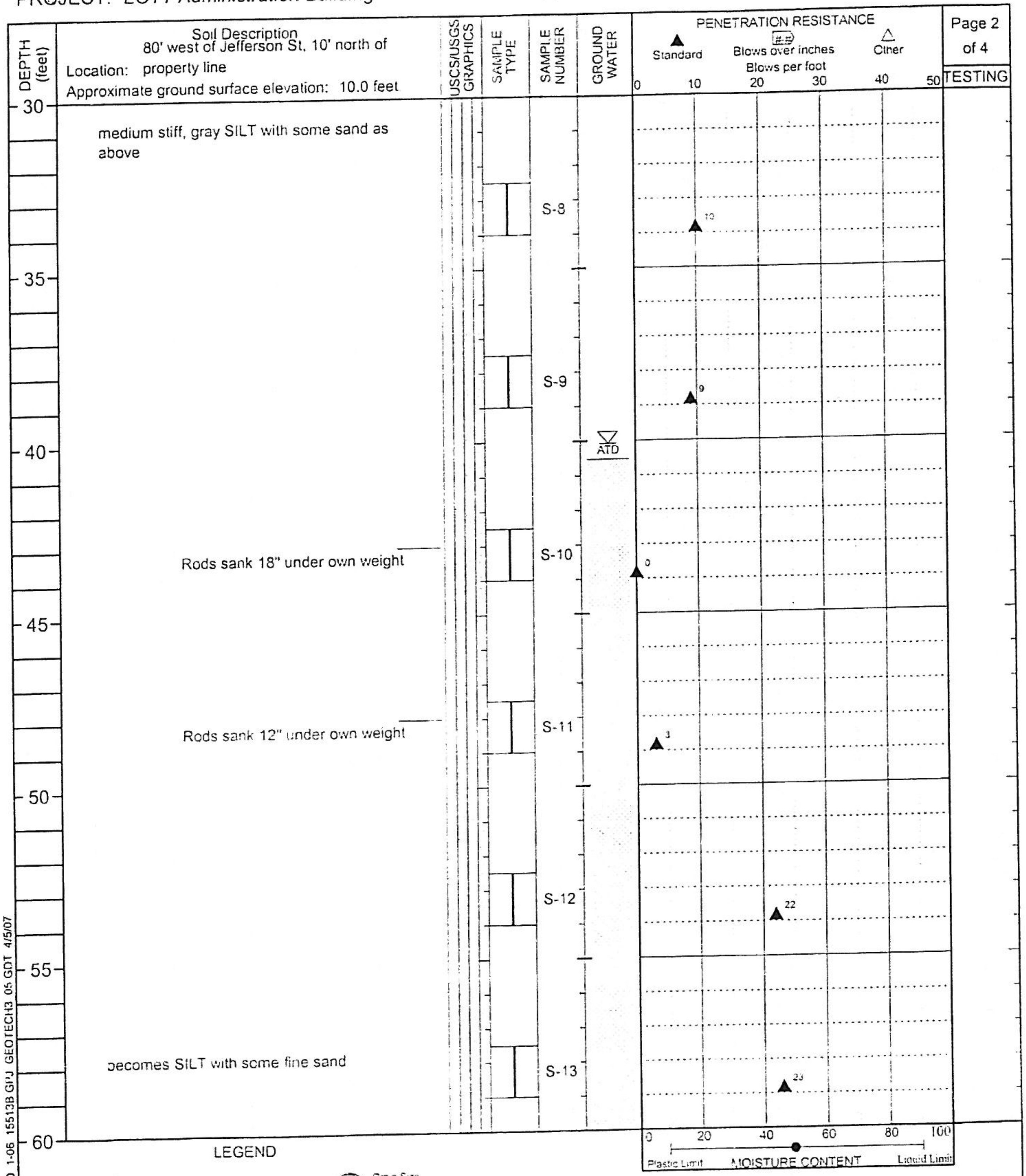
PROJECT: *LOTT Administration Building* JOB No. *7-917-15513-B* BORING No. *AB-8*

Soil Description

Location: 80' west of Jefferson St, 10' north of property line

Approximate ground surface elevation: 10.0 feet

Page 2 of 4



medium stiff, gray SILT with some sand as above

Rods sank 18" under own weight

Rods sank 12" under own weight

becomes SILT with some fine sand

LEGEND

2.00-inch OD split-spoon sampler

No Recovery

Groundwater level at time of drilling

Grain Size Analysis (F= fines shown)

amec

11335 N.E. 122nd Way Suite 100
Kirkland, Washington 98034-6913

SOIL ROCK COMBO 1-06 15513B.GPJ GEOTECH3_05.GDT 4/5/07

Drilling Method: HSA Hammer Type: Automatic Date drilled: *March 05, 2007* Logged By: *WJL*

Drilled by: *Enviromental Drilling*

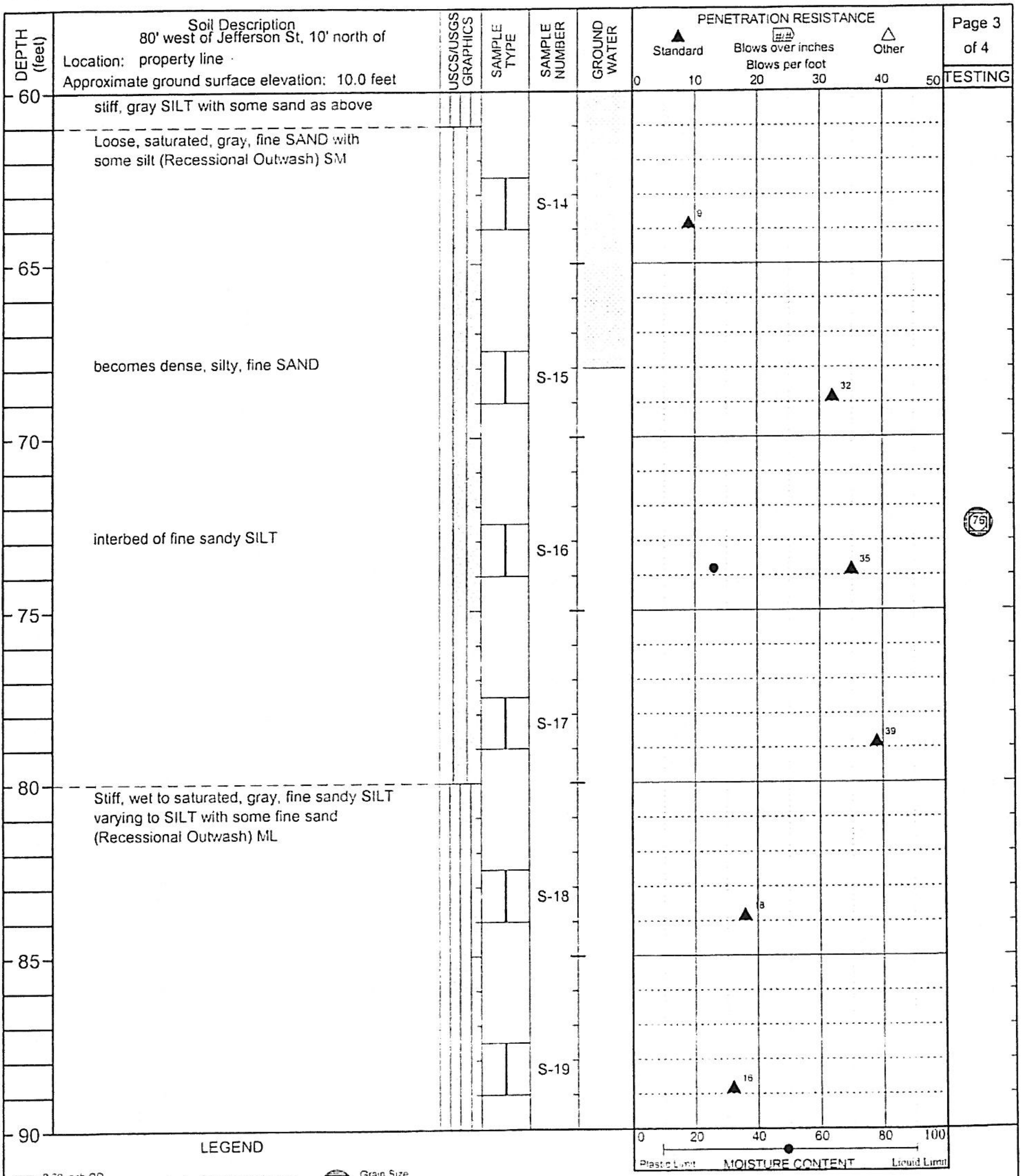
PROJECT: *LOTT Administration Building* JOB No. *7-917-15513-B* BORING No. *AB-8*

Page 3 of 4

Soil Description

Location: 80' west of Jefferson St, 10' north of property line

Approximate ground surface elevation: 10.0 feet



stiff, gray SILT with some sand as above

Loose, saturated, gray, fine SAND with some silt (Recessional Outwash) SM

becomes dense, silty, fine SAND

interbed of fine sandy SILT

Stiff, wet to saturated, gray, fine sandy SILT varying to SILT with some fine sand (Recessional Outwash) ML

SOIL ROCK COMBO 1-06 15513B.GPJ GEOTECH3 05.GDT 4/5/07

LEGEND

2.00-inch OD split-spoon sampler

☒ No Recovery

ATD Groundwater level at time of drilling

Grain Size Analysis (% fines shown)

amec

11335 N.E. 122nd Way Suite 100
Kirkland, Washington 98034-6913

Drilling Method: HSA Hammer Type: Automatic Date drilled: *March 05, 2007* Logged By: *WJL*

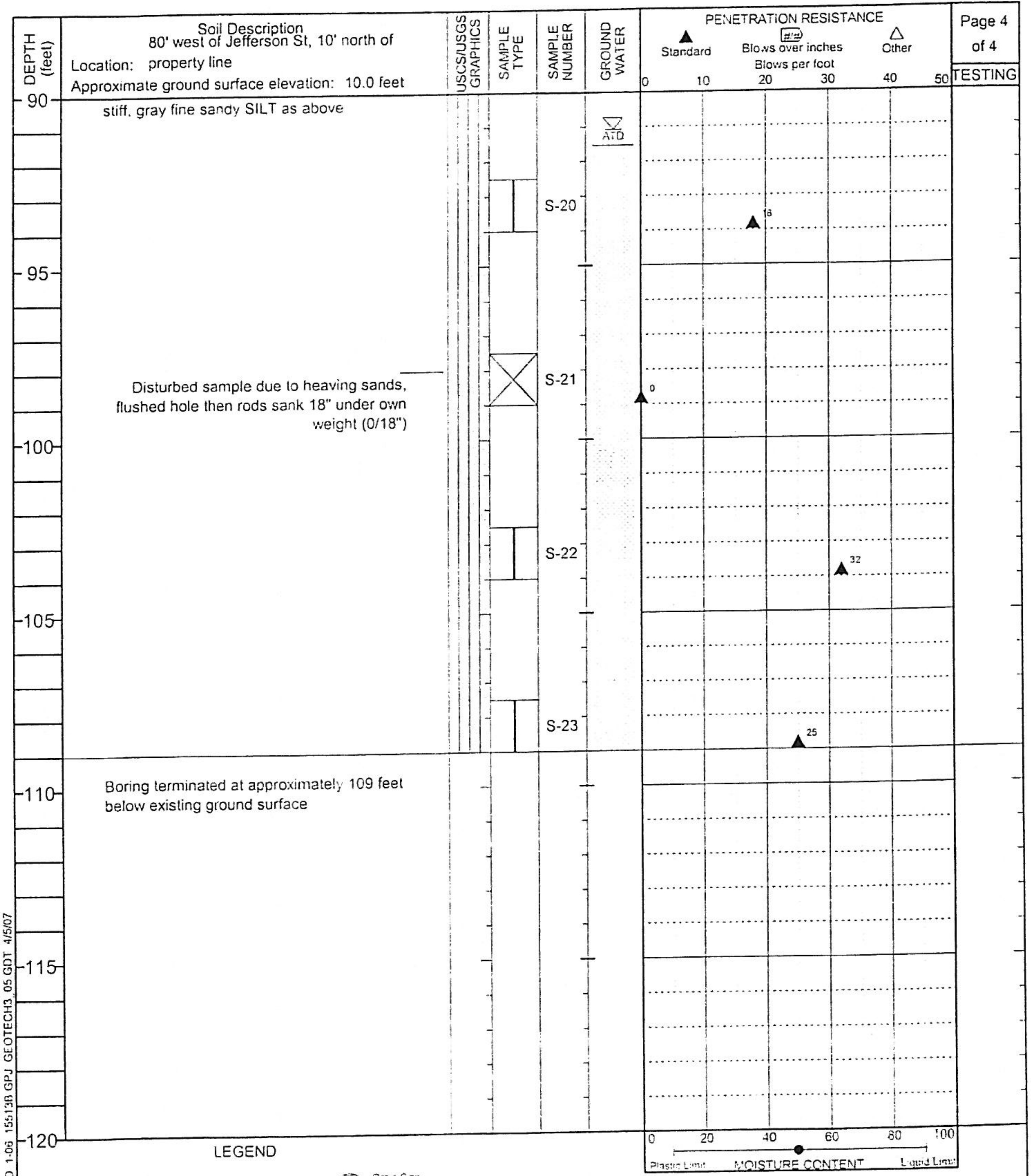
Drilled by: *Enviromental Drilling*

PROJECT: *LOTT Administration Building* JOB No. *7-917-15513-B* BORING No. *AB-8*

Location: 80' west of Jefferson St, 10' north of property line

Approximate ground surface elevation: 10.0 feet

Page 4 of 4



stiff, gray fine sandy SILT as above

Disturbed sample due to heaving sands, flushed hole then rods sank 18" under own weight (0/18")

Boring terminated at approximately 109 feet below existing ground surface

LEGEND

◫ 2.00-inch OD split-spoon sampler

☒ No Recovery

▽ ATD Groundwater level at time of drilling

Grain Size Analysis (% fines shown)

amec

11335 N.E. 122nd Way Suite 100
Kirkland, Washington 98034-6913

Drilling Method: HSA Hammer Type: Automatic Date drilled: *March 05, 2007* Logged By: *WJL*

Drilled by: *Enviromental Drilling*

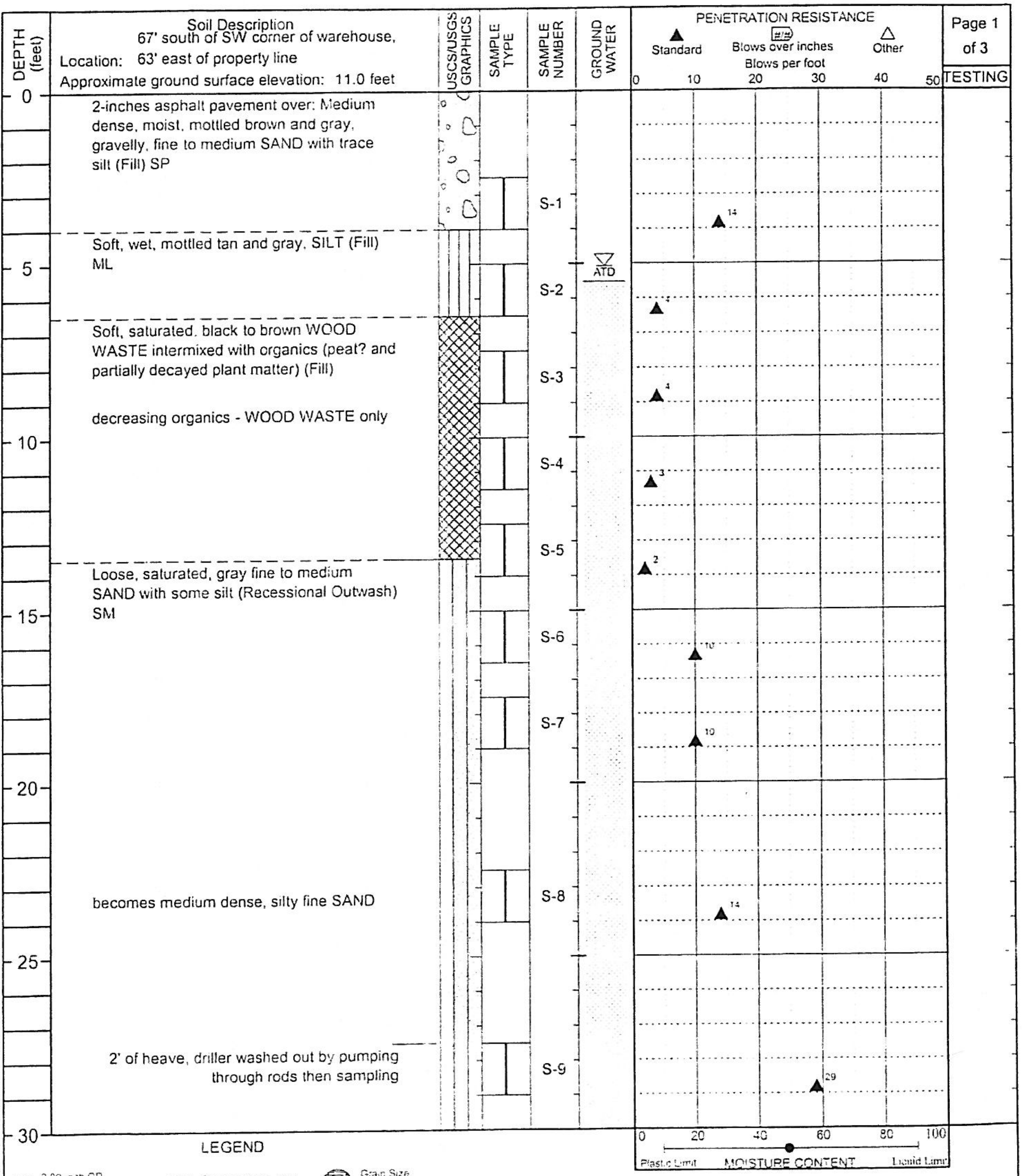
PROJECT: *LOTT Administration Building* JOB No. *7-917-15513-B* BORING No. *AB-9*

Page 1 of 3

Soil Description

Location: 67' south of SW corner of warehouse, 63' east of property line

Approximate ground surface elevation: 11.0 feet



2-inches asphalt pavement over: Medium dense, moist, mottled brown and gray, gravelly, fine to medium SAND with trace silt (Fill) SP

Soft, wet, mottled tan and gray, SILT (Fill) ML

Soft, saturated, black to brown WOOD WASTE intermixed with organics (peat? and partially decayed plant matter) (Fill)

decreasing organics - WOOD WASTE only

Loose, saturated, gray fine to medium SAND with some silt (Recessional Outwash) SM

becomes medium dense, silty fine SAND

2' of heave, driller washed out by pumping through rods then sampling

LEGEND

2.00-inch OD split-spoon sampler

ATD Groundwater level at time of drilling

Grain Size Analysis (% fines shown)

200 Wash (% fines shown)

amec

11335 N.E. 122nd Way Suite 100
Kirkland, Washington 98034-6913

Drilling Method: HSA Hammer Type: Automatic Date drilled: *March 06, 2007* Logged By: *WJL*

Drilled by: *Enviromental Drilling*

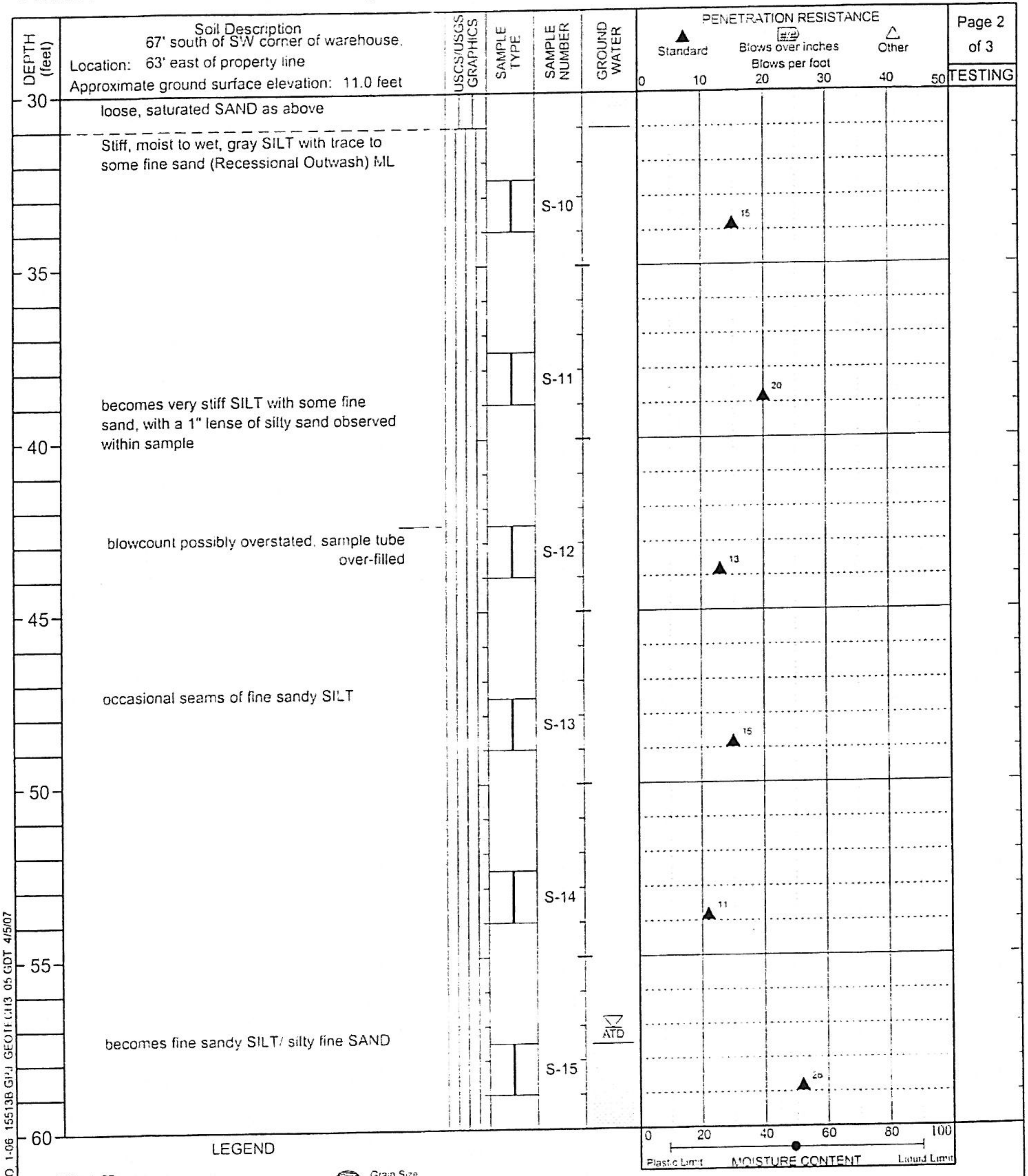
PROJECT: *LOTT Administration Building* JOB No. *7-917-15513-B* BORING No. *AB-9*

Page 2 of 3

Soil Description

Location: 67' south of SW corner of warehouse, 63' east of property line

Approximate ground surface elevation: 11.0 feet



loose, saturated SAND as above

Stiff, moist to wet, gray SILT with trace to some fine sand (Recessional Outwash) ML

becomes very stiff SILT with some fine sand, with a 1" lense of silty sand observed within sample

blowcount possibly overstated, sample tube over-filled

occasional seams of fine sandy SILT

becomes fine sandy SILT/ silty fine SAND

LEGEND

◫	2.00-inch OD split-spoon sampler
▽ ATD	Groundwater level at time of drilling
◎	Grain Size Analysis (% fines shown)
▢	200 Wash (% fines shown)

SOIL ROCK COMBO 1-06 15513B.GPJ GEOTECH3 05.GDT 4/5/07

amec

11335 N.E. 122nd Way Suite 100
Kirkland, Washington 98034-6913

Drilling Method: HSA Hammer Type: Automatic Date drilled: *March 06, 2007* Logged By: *WJL*

Drilled by: *Enviromental Drilling*

PROJECT: *LOTT Administration Building* JOB No. *7-917-15513-B* BORING No. *AB-9*

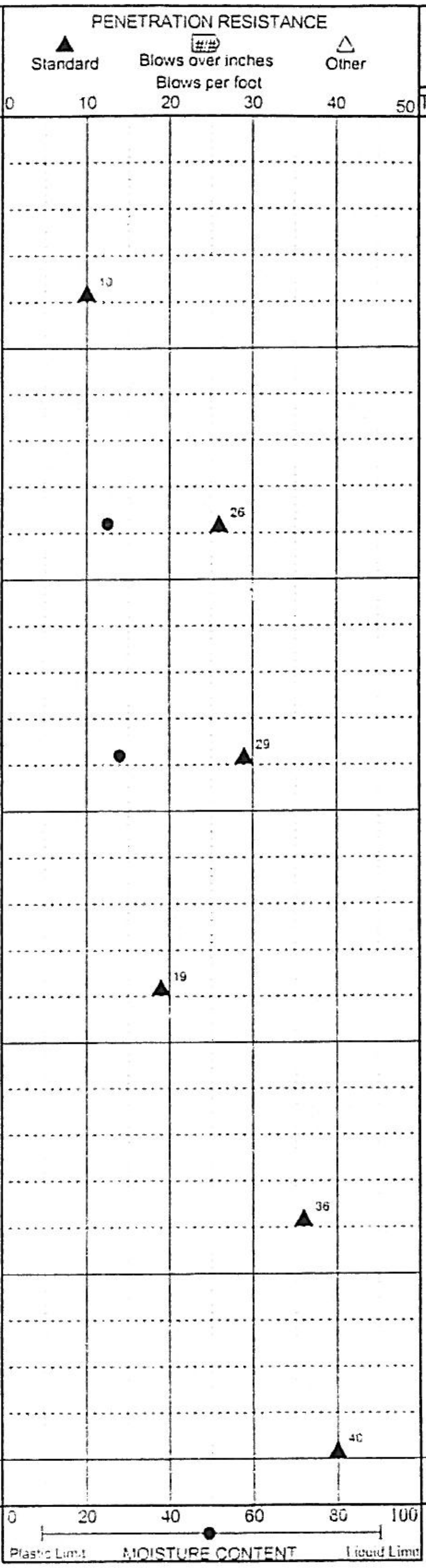
Soil Description

Location: 67' south of SW corner of warehouse, 63' east of property line

Approximate ground surface elevation: 11.0 feet

Page 3 of 3

DEPTH (feet)	Soil Description	SAMPLE NUMBER	TESTING
60	stiff, wet fine sandy SILT as above		
	becomes SILT with some (?) clay, moderate plasticity, easily rolled to 1/8" thread	S-16	
65			
	Medium dense, saturated, gray, silty, fine SAND with occasional stringers of SILT (Recessional Outwash) SM		34
	blowcount possibly overstated, sample tube over-filled	S-17	
70			
	driller noted 6" of slough within hole	S-18	41
75			
	interbedded layer of: very stiff, wet, gray, fine, sandy SILT ML driller pumped water through rods into hole prior to sampling	S-19	
80			
	becomes dense, gray, fine to medium SAND with some silt 2' of heave noted, pumped water through rods to wash out prior to sampling	S-20	
85			
	becomes dense, gray, silty, fine SAND	S-21	
	Boring terminated at approximately 89 feet		
90			



LEGEND

2.00-inch OD split-spoon sampler

ATD Groundwater level at time of drilling

Grain Size Analysis (% fines shown)

200 Wash (% fines shown)

11335 N.E. 122nd Way Suite 100
Kirkland, Washington 98034-6913

Drilling Method: HSA Hammer Type: Automatic Date drilled: *March 06, 2007* Logged By: *WJL*

Drilled by: *Enviromental Drilling*

DRAFT
REMEDIAL INVESTIGATION/FEASIBILITY STUDY AND CONCEPTUAL CLEANUP ACTION PLAN
EAST BAY REDEVELOPMENT
PORT OF OLYMPIA
OLYMPIA, WASHINGTON
ECOLOGY FACILITY/SITE NO. 5785176
VCP NO. SW0827

DECEMBER 20, 2007

FOR
PORT OF OLYMPIA

■	**Test Pit (GeoEngineers, Inc. - Oct. 2007)**
⊕	**Monitoring Well (GeoEngineers, Inc. - Jan. & July 2007)**
●	**Direct-Push Boring (GeoEngineers, Inc. - Sept. 2006, Jan. & July 2007)**
◉	**Direct-Push Boring (Brown and Caldwell - Nov. 2006, Jan. & Feb. 2007)**
▬	**East Bay Redevelopment Proposed Short Plat Parcel Boundaries**
▭	**East Bay Redevelopment Project Area**
▬ ▬	**LOTT Parcel (Pending Sale from Port)**
▲	**Artesian Well (Maintained by Port)**
⊕	**Monitoring Well (Delta Environmental - June 2003)**
◐	**Direct-Push Boring (Northwest Testing Company, Oct. 2006)**
◎	**Cone Penotrometer Test (Landau - May 2007)**
◑	**Boring (Landau - May 2007)**
▬	**Proposed Right-of-Way**
▬	**Storm Drainage System**
⬟	**Outfall Locations**

N
W E
S

150 0 150
Feet

Reference: United States Geologic Survey (obtained January 2007). Parcel boundaries, Artesian Well locations and Storm Drainage data are based on information provided by the Port of Olympia. (Proposed Right-of-Way per Skillings Connolly Drawing).

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for infomation purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. can not guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record oth this communication.

Site Plan and Exploration Locations

East Bay Redevelopment Project
Olympia, Washington

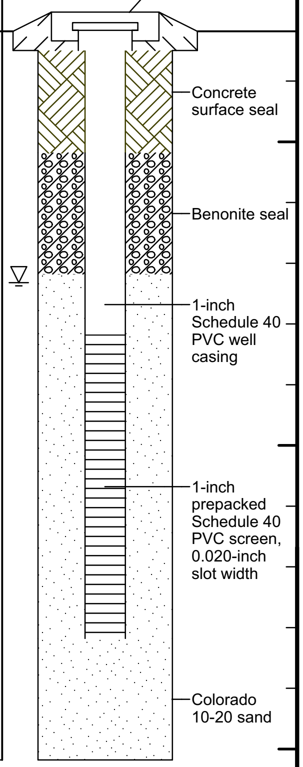
GEOENGINEERS

Draft
Figure 3

Date(s) Drilled	01/02/07	Logged By	TSG	Checked By	KMB
Drilling Contractor	ESN-NW	Drilling Method	Direct Push	Sampling Methods	Grab; 5035A for VOCs
Auger Data	NA	Hammer Data	Pneumatic	Drilling Equipment	Stratoprobe
Total Exploration Depth (ft)	12	Ground Surface Elevation (ft)	101.82	Groundwater Elevation (ft)	97.68
Vertical Datum	Assumed (100')	Datum/ System	GCS - North American - 1983	Easting(x): Northing(y):	47.0483000631 -122.896724939

Elevation feet	Depth feet	SAMPLES Recovered (in)	Blows/foot	Sub-Sample Sample Number	Testing	Group Symbol	MATERIAL DESCRIPTION	Sheen	Headspace Vapor TLV (ppm)
	0	12				CC	Concrete slab		
						SP	Brown fine to coarse sand with gravel (loose to medium dense, moist)	NS	
				020	CA			NS	
		12							
	5			040			Grades to loose, moist	NS	
								NS	
		36				GP	Brown gravel with sand (loose, wet)		
				080				NS	
	10								
				100	CA				
	15								
	20								

WELL CONSTRUCTION



Note: See Figure B-1 for explanation of symbols.

LOG OF MONITORING WELL MW01 (AKA 425)

GEOENGINEERS

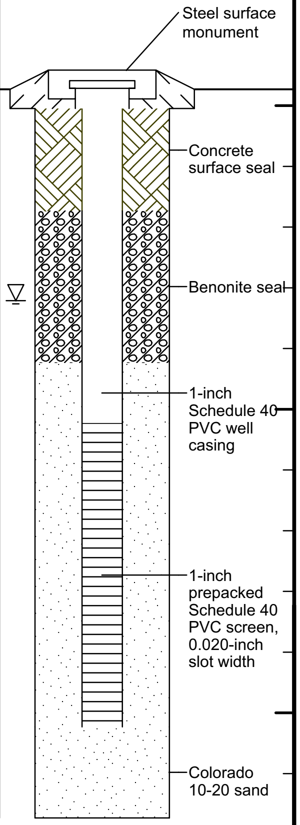
Project: Phase II ESA/Hands on Children's Museum
Project Location: Olympia, Washington
Project Number: 0615-033-00

Figure B-4
Sheet 1 of 1

Date(s) Drilled	01/02/07	Logged By	TSG	Checked By	KMB
Drilling Contractor	ESN-NW	Drilling Method	Direct Push	Sampling Methods	Grab; 5035A for VOCs
Auger Data	NA	Hammer Data	Pneumatic	Drilling Equipment	Stratoprobe
Total Exploration Depth (ft)	12	Ground Surface Elevation (ft)	100.27	Groundwater Elevation (ft)	96.79
Vertical Datum	Assumed (100')	Datum/ System	GCS - North American - 1983	Easting(x): Northing(y):	47.0481962236 -122.897072584

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Testing	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Sheen	Headspace Vapor TLV (ppm)
	0		24					SP	Brown fine to medium sand with occasional fine gravel (loose, moist)		No PID
					000					NS	
					020	CA		SM	Brown fine gravel with sand and trace silt	NS	
	5				040					NS	
					060					NS	
					080	CA				NS	
	10										
	15										
	20										

WELL CONSTRUCTION



Note: See Figure B-1 for explanation of symbols.

LOG OF MONITORING WELL MW02 (AGT 115)

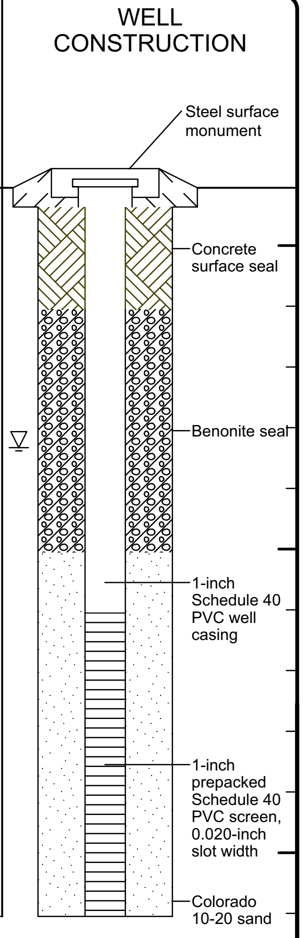
GEOENGINEERS

Project: Phase II ESA/Hands on Children's Museum
Project Location: Olympia, Washington
Project Number: 0615-033-00

Figure B-5
Sheet 1 of 1

Date(s) Drilled	01/02/07	Logged By	TSG	Checked By	KMB
Drilling Contractor	ESN-NW	Drilling Method	Direct Push	Sampling Methods	Grab; 5035A for VOCs
Auger Data	NA	Hammer Data	Pneumatic	Drilling Equipment	Stratoprobe
Total Exploration Depth (ft)	12	Ground Surface Elevation (ft)	100.95	Groundwater Elevation (ft)	96.67
Vertical Datum	Assumed (100')	Datum/ System	GCS - North American - 1983	Easting(x): Northing(y):	47.04784838 -122.896712081

Elevation feet	Depth feet	SAMPLES Interval	SAMPLES Recovered (in)	SAMPLES Blows/foot	SAMPLES Sub-Sample Sample Number	SAMPLES Testing	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Sheen	Headspace Vapor TLV (ppm)
	0		12					CC	Concrete slab		
								SP	Brown fine to coarse sand with gravel (medium dense, moist)		
					000					NS	0.0
					020					NS	0.0
			48								
	5				040	CA				NS	0.0
										NS	0.0
			24					GP	Gray/brown gravel with sand (medium dense, wet)		
					080	CA				NS	0.0
	10										
	15										
	20										



Note: See Figure B-1 for explanation of symbols.

LOG OF MONITORING WELL MW03 (AKA 427)

GeoEngineers

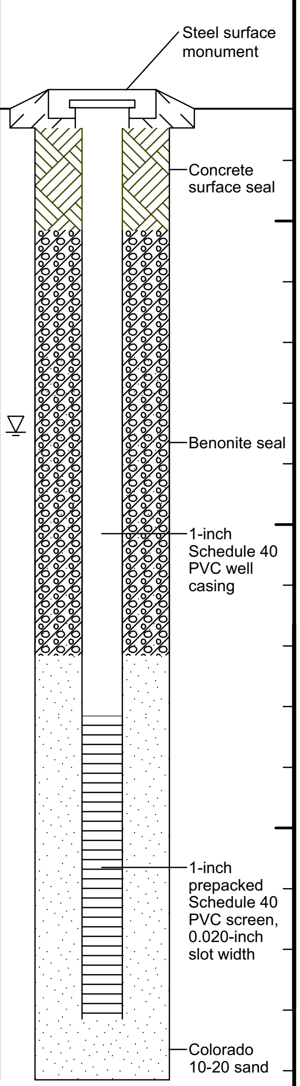
Project:	Phase II ESA/Hands on Children's Museum
Project Location:	Olympia, Washington
Project Number:	0615-033-00

Figure B-6
Sheet 1 of 1

Date(s) Drilled	01/02/07	Logged By	TSG	Checked By	KMB
Drilling Contractor	ESN-NW	Drilling Method	Direct Push	Sampling Methods	Grab; 5035A for VOCs
Auger Data	NA	Hammer Data	Pneumatic	Drilling Equipment	Stratoprobe
Total Exploration Depth (ft)	16	Ground Surface Elevation (ft)	101.85	Groundwater Elevation (ft)	96.52
Vertical Datum	Assumed (100')	Datum/ System	GCS - North American - 1983	Easting(x): Northing(y):	47.048155726 -122.896040672

Elevation feet	Depth feet	SAMPLES					Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Sheen	Headspace Vapor TLV (ppm)
		Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Testing					
	0		40					GP	Medium brown gravel with sand (medium dense, moist to dry)		
					000					NS	0.0
					020	CA		SP	Dark brown sand with gravel and brick (fill)	NS	0.0
			24					OL	Brown organic silt		
	5				040			ML/SP	Gray silt with layers of fine to coarse graded sand with shells (dense/loose, wet)	NS	0.0
										NS	0.0
			40								
					080					NS	0.0
	10										
					100			SP	Black sand with decomposing wood material (moist)	NS	0.0
			48						Dark brown coarse sand with occasional gravel (loose, wet) Preservative odor		
					120				Sewage odor	NS	0.0
	15				140	CA			Dark brown/black coarse sand with gravel and wood debris (loose, wet) Intense preservative odor	NS	0.0
	20										

WELL CONSTRUCTION



Note: See Figure B-1 for explanation of symbols.

LOG OF MONITORING WELL MW04 (AKA 424)

GEOENGINEERS

Project: Phase II ESA/Hands on Children's Museum
Project Location: Olympia, Washington
Project Number: 0615-033-00

Figure B-7
Sheet 1 of 1